NZS 4203:1992

Corrigenda appended

New Zealand Standard

Code of practice for general structural design and design loadings for buildings

(Known as the Loadings Standard)

Volume 1: Code of practice

Superseding NZS 4203:1984

NZS 4203:1992

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COMMITTEE REPRESENTATION

This standard was prepared under the direction of the Building and Civil Engineering Board, (30/-) for the Standards Council, established under the Standards Act 1988 (1965).

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ACKNOWLEDGEMENT

Standards New Zealand gratefully acknowledges the contribution of time and expertise from all those involved in developing this standard.

Published by Standards New Zealand, the trading arm of the Standards Council, Private Bag 2439, Wellington 6140. Telephone: (04) 498 5990; Fax: (04) 498 5994; Website: www.standards.co.nz.

AMENDMENTS

No.	*Date of issue*	*Description*	*Entered by, and date*

CONTENTS PAGE

(Contents continued overleaf)

PART 4 EARTHQUAKE PROVISIONS

(Contents continued)

PART 5 PROVISIONS FOR WIND FORCES

Appendix

(Contents continued overleaf)

(Contents continued)

Figure

(Contents continued overleaf)

RELATED DOCUMENTS

Reference is made in this document to the following:

AUSTRALIAN STANDARDS

AS 1170:- - - -	Minimum design loads on structures
Part 1:1989	Dead and live loads and load combinations
Part 2:1989	Wind loads

BUILDING INDUSTRY AUTHORITY

New Zealand Building Code, 1992
Approved Document B1 Structure – General

FOREWORD

This revision of NZS 4203 is written in limit state format. It stipulates the level of loading which is required to be imposed on a building when assessing its ability to satisfy a particular limit state and stipulates a means for calculating the distribution of loading to the elements of the building. This revision is to be used with material standards presented in limit state format. Part 1 of this Standard presents the scope and definitions and Part 2 the general requirements and load combinations. Parts 3, 4 and 5 present provisions for dead and live loads, seismic actions and wind loads, respectively. Part 6 addresses other load effects such as snow, soil pressures, ponding and ground water.

Volume 2 (commentary) provides background details and explanatory guidelines to the provisions in Volume 1 and provides recommended sound practice details to satisfy performance related provisions from Volume 1. Commentary clauses are not mandatory.

Limit state design

A structure, or part of it, may fail to fulfil basic functions required of it. When this occurs it is said to have reached a limit state. Two limit states are identified, namely the Serviceability limit state and the Ultimate limit state.

The criteria specified for the serviceability limit state deal with deformation and deflection limits which affect the appearance or function of buildings. Examples include impairment to drainage, damage to finishes and the occurrence of excessive deflection or vibration which may cause alarm or discomfort.

The criteria specified for the ultimate limit state relate to the strength and stability of all or part of a building. This limit state is reached when the building, or part of it, fails to support the specified loads and forces. This failure may be due to the provision of insufficient strength, ductility or stability or any combination of these.

Society dictates acceptable levels of building failures and these are incorporated into the levels of reliability implicit in limit state design. The level of reliability is expressed in the form of a safety index, β, which is a measure of the notional safety of a chosen ultimate limit state structural model, taking into account the statistical variation in design loads and structural capacities. The safety index is an integral part of the code calibration exercise that is undertaken during development of a limit state material standard in order to derive appropriate strength reduction factors for use in ultimate limit state design.

As a guide to the developers of material standards that are to be used with this limit state edition of NZS 4203, target values of safety index are given in this Foreword. The values for structural sub assemblages subject to earthquake loading should be regarded as interim target values only, pending further study. These values are:

(1) For members subject to gravity loading – average value 3, range from 2.5 to 3.5.

(2) For structural assemblages subject to gravity loading and wind loading – average value 2.5, range from 2.25 to 2.75.

(3) For structural assemblages subject to gravity and a specified level of earthquake lateral loading – average value 1.75, range from 1.5 to 2.0.

This Standard prescribes the required nominal loads and load factors required for design. Compatible material standards must, through limit state calibration processes, derive strength reduction factors in accordance with the level of loading specified herein and the target values of safety index specified above.

With regard to the nominal loads specified herein, the probabilities of exceedence vary between different loads and load combinations. For dead and live load combinations the ultimate loads are consistent with overseas practices, notably US and Australian practices [1,2] where the factored dead and live load combinations have a 5 percentile probability that the specified nominal loads will be exceeded within any year for the serviceability limit state and within the nominal 50 year life of the building for the ultimate limit state.

The ultimate limit state wind speeds and peak earthquake acceleration response spectra from which the design spectra have been derived, have a probability of exceedence of 10 % over the building assumed life of 50 years. Back calibration indicates that the loads so derived are consistent with those derived using previously accepted practice.

Part 1 Scope and interpretation

This revision has been written to be a means of compliance document with Clause B1 of the New Zealand Building Code. It is applicable to normal buildings and their parts. Unusual buildings are required to be subjected to special study. The requirements of the Standard are detailed in the clauses within Volume 1 and are required to be met to demonstrate compliance. The commentary clauses are published in Volume 2 and are provided as an expansion and to give guidance on the background to the clauses in the Standard. The commentary provisions are recommendations only, and departure from these recommendations does not constitute non-compliance with this Standard. An expanded list of definitions of terms used throughout the Standard is included in this Part, while the notation of symbols used remains with their respective parts.

Part 2 General requirements

This Part forms the core of this Standard, linking the loads derived from the other Parts through the load combinations. The classification of buildings and parts of buildings are defined in this Part, as are the acceptable analysis and design procedures for the various load conditions. The load combinations published in this Part have been assessed as providing an appropriate level of safety for New Zealand conditions. They are consistent with those used in the 1980 version of American National Standard A58. (Subsequently revised and redesignated ANSI/ASCE 7-1988). Load combinations are specified for both the serviceability and ultimate limit states. Live load duration factors are specified for the serviceability limit state which enables the nominal live loads to be adjusted for both short and long term

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1. ELINGWOOD, B. et al, "Development of a Probability Based Load Criterion for American National Standard A58", NBS Special Publication 577, June 1980.

2. "Safety of Limit - States Structural Design Codes", Seminar proceedings, Institute of Structural Engineers Australia, March 1986.

situations. A live load combination factor is specified for the ultimate limit state and is used when the live load is considered in conjunction with other rare events. Both factors have been derived from work undertaken in Australia and are consistent with the approach used in the Australian Loading Code (AS 1170:1989).

Designers should recognise that the (pre-1991) material standard provisions for deriving member strengths are written to be used in conjunction with the higher load factors (for the ultimate limit state) contained in the 1984 edition of this Standard. Their use with this edition will result in a lower safety index than that intended for the ultimate limit state and therefore such use should be considered as an interim measure pending the publication of formal limit state material Standards.

Part 3 Dead and live load provisions
In writing this Part of the Standard, some changes were made to the loading intensities in NZS 4203:1984 so that the values are generally in accordance with the requirements of AS 1170.1:1989

Part 4 Seismic provisions
Three methods of analysis for seismic actions are given. The equivalent static method is recommended for low-rise structures. Use of this method for higher structures is restricted to those which are reasonably regular and are expected to respond to a major earthquake principally in a translational first mode. Where these requirements are not satisfied, an analysis using either the modal response spectrum method or the numerical integration time history method is required. These two methods may be used for the analysis of all buildings.

Seismic actions are considered for both the serviceability and ultimate limit states.

Part 5 Wind loading
This Part is closely aligned to AS 1170.2:1989. Some additional multipliers which reflect the more rugged New Zealand terrain have been introduced for the determination of the site wind speed.

Structures may be designed using either a static analysis approach (similar to that used in earlier editions of this Standard) or, when they are wind sensitive (i.e. experience enhanced dynamic effects through the aerodynamic interaction of the building with the wind flow), the dynamic design procedures detailed in AS 1170.2 Section 4. The information necessary to determine the dynamic wind speeds for New Zealand conditions is provided within this Part.

A major change in this revision is the introduction of directionality for determining the site wind speeds, and thus the design wind speed. This revision includes many of the advances in recent years in understanding and quantifying effects of topography on wind speed. A conservative, non-directional approach is also outlined.

Changes in the tables of pressure coefficients reflect advances in knowledge in this area also. Several different building shapes are included, both in the body of the Standard, and in the Appendix to Part 5.

Part 6 Other loads

Code of practice for GENERAL STRUCTURAL DESIGN AND DESIGN LOADINGS FOR BUILDINGS

PART 1 SCOPE AND INTERPRETATION

1.1 Scope

1.1.1 *General*
This Standard sets out requirements for general structural design and design loadings for buildings, including the supports for services entering and within buildings, parts and portions of buildings, and pedestrian bridges within a building site. It is cited as a Verification Method in Approved Document B1/1: Structure – General, to the New Zealand Building Code.

1.1.2 *Unusual buildings*
A special study shall be made for any building which is sufficiently unusual for the provisions of this Standard to be appropriate only as a general guide.

1.1.3 *Non-brittle behaviour*
The seismic provisions of Part 4 of this Standard apply only to structures (or elements) which exhibit non-brittle behaviour.

1.2 Interpretation

1.2.1 *General*

1.2.1.1
In this Standard the word "shall" indicates a requirement that is to be adopted in order to comply with the Standard, while the word "should" indicates a recommended practice.

1.2.1.2
Cross-references to other clauses or clause sub-divisions within this Standard quote the number only.

1.2.1.3
Figures, tables and equations are numbered consecutively within each section of the Standard. For example, table 4.6.2 is the second table within section 4.6 of Part 4.

1.2.1.4
The full titles of reference documents cited in this Standard are given in the list of Related Documents immediately preceding the Foreword.

1.2.2 *Definitions*
For the purposes of this Standard, the following definitions shall apply:

BASE. The level at which earthquake motions are considered to be imparted to the structure, or the level at which the structure as a dynamic vibrator is supported.

CENTRE OF RIGIDITY. The centres of rigidity for a multi-storey building are defined as the set of points located at floor levels, such that when the given shears associated with the lateral forces pass through them no rotation occurs at any level.

CONCURRENCY. The occurrence of earthquake induced deformation in directions not necessarily aligned to either principal direction and therefore causing earthquake induced actions in both principal directions.

DESIGNER. The person who is responsible for the adequacy of all those aspects of the design which affect structural performance.

DESIGN METHODS

CAPACITY DESIGN. The design method in which elements of the primary lateral earthquake force resisting system are chosen and suitably designed and detailed for energy dissipation under severe deformations. All other structural elements are then provided with sufficient strength so that the chosen means of energy dissipation can be maintained.

PLASTIC METHOD. The design method in which members are proportioned and restrained as necessary to ensure that the intended collapse mechanism does not form under load levels less than the appropriately factored load combinations.

STRENGTH METHOD. The design method in which members are proportioned to sustain appropriately factored load combinations without exceeding their dependable strength. Allowance may be made for redistribution associated with inelastic behaviour where appropriate.

DESIGN SPECTRUM. A spectrum used for analysis and design of a structure.

DIAPHRAGM. A horizontal or near horizontal system which acts to transmit lateral forces to the lateral force resisting elements.

DIRECTIONALITY. The relationship established from wind speed records, corrected to uniform terrain category, which assigns individual values to wind speeds from discrete directions.

DOMINANT OPENING. An opening in the external surface of an enclosed building which directly influences the average internal pressure in response to external pressures at that particular opening. NOTE – Dominant openings need not be large.

DUCTILE STRUCTURE. A structure designed and detailed in accordance with this Standard and the appropriate material standards so that a structural ductility factor of more than 3 is appropriate in assessing the ultimate limit state seismic actions.

DUCTILITY. The ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake.

ELASTICALLY RESPONDING STRUCTURE. A structure designed and detailed in accordance with this Standard and the appropriate material standard so that a structural ductility factor of 1 to 1.25 is appropriate in assessing the ultimate limit state seismic actions.

ELEVATED BUILDING. A building with a clear, unwalled space underneath the first floor level with a height from ground to underside of the first floor of at least one third of the total height of the building.

EQUIVALENT GROUND SURFACE. The ground surface established by interpolation of residual ground levels around the perimeter of the building.

FORCE. Refers to forces induced by wind and earthquake.

EARTHQUAKE FORCES. Inertia-induced forces arising from the building response to earthquake. Refer to Part 4.

WIND FORCES. The forces imposed on a structure as a result of differential pressures caused by the passage of wind past the structure. Refer to Part 5.

FORCE COEFFICIENT. A coefficient which, when multiplied by the incident wind pressure and an appropriate area (defined in the text), gives the force in a specific direction.

FREESTREAM DYNAMIC PRESSURE. The theoretically computed incident pressure of a uniform airstream of known density, $q = 0.0006\ V^2$ (at ambient temperature and barometric pressure).

HOARDING. A free standing signboard, etc. supported clear of the ground.

IMMEDIATE SUPPORTS. (Cladding). Those supporting members to which cladding is directly fixed (e.g. battens, purlins, girts, studs.)

INTERNAL STRAIN. A design action associated with general self-equilibrating stresses such as those induced by foundation settlement, prestress, welding, shrinkage, creep, temperature etc.

INTER-STOREY HEIGHT. The overall height between two successive structural levels of a building.

LATTICE TOWER. A three dimensional framework comprising three or more linear boundary members interconnected by linear bracing members joined at common points (nodes).

LIMIT STATE

SERVICEABILITY LIMIT STATE. This condition is reached when the building becomes unfit for its intended use through deformation, vibratory response, degradation or other physical aspects.

ULTIMATE LIMIT STATE. This condition is reached when the building ruptures, becomes unstable or loses equilibrium.

LIMITED DUCTILE STRUCTURE. A structure designed and detailed in accordance with this Standard and the appropriate material standards so that a structural ductility factor of between 1.25 and 3 is appropriate in assessing the ultimate limit state seismic actions.

LOAD. Relates to the action of gravity on dead and live loads and other loads specified in Part 6.

DEAD LOAD. The weight of all permanent components, partitions, permanently fixed plant and fittings and relatively invariant long term loads (refer to Part 3). Soil and ground water loads are also regarded as dead loads (refer to Part 6).

LIVE LOAD. Loads assumed or known to result from the occupancy or use of a building (refer to Part 3). It does not include earthquake (Part 4), wind (Part 5) and other forces (Part 6).

LOAD COMBINATION and LOADING. Refer to the specified combination of loads (gravity related) and forces (wind and seismic) and internal strain effects.

MATERIAL STANDARD. A New Zealand Standard code of practice for design of structures in a specific material.

P-DELTA EFFECT. Refers to the structural actions induced as a consequence of the gravity loads being displaced laterally due to the action of lateral forces.

PART. An element which is not intended to participate in the overall resistance of the structure to lateral displacement under earthquake conditions in the direction being considered.

PARTITION. A permanent or relocatable internal dividing wall between floor spaces.

POROSITY. The ratio of open area to the total area of the surface.

SOFT STOREY. A storey where the ratio of the inter-storey deflection divided by the product of the storey shear and storey height exceeds 1.4 times the corresponding ratio for the storey immediately above this level.

SPECIAL STUDY. A procedure for the analysis and/or design of the building, in which some or all of the requirements of this Standard and of other Standards may be modified or waived.

STOREY. The space between two adjacent floors or platform levels.

STRENGTH

DEPENDABLE STRENGTH. The ideal strength multiplied by the strength reduction factor as given in the appropriate material standard.

IDEAL STRENGTH. The theoretical strength of a member section, calculated using the section dimensions as detailed and the characteristic strengths as defined in the appropriate material standard.

PROBABLE STRENGTH. The theoretical strength of a member section calculated using the expected mean material strengths as defined in the appropriate material standard.

OVERSTRENGTH. The maximum probable strength of a member section calculated taking into account the main factors that may contribute to an increase in strength as defined in the appropriate material standard.

STRUCTURAL DUCTILITY FACTOR. A numerical assessment of the ability of a structure to sustain cyclic inelastic displacements. Its value depends upon the structural form, the ductility of the material and structural damping characteristics.

STRUCTURAL PERFORMANCE FACTOR. A numerical assessment of the ability of the building to survive cyclic displacements. Its value depends on the material, form and period of the seismic resisting system, damping of the structure, and interaction of the structure with the ground.

TERRAIN. The ground roughness which creates a drag effect on the passage of wind as it passes over the earth's surface.

TOP (of a structure). The level of the uppermost principal seismic weight.

TOPOGRAPHY. Major land surface features (e.g. hills, valleys and ranges) which cause the free wind streams to accelerate.

TRANSITION ZONE. The fringe area over which the wind speed profile develops as it passes from ground of one roughness to a different roughness.

TRIBUTARY AREA. The area over which the total force or load supported by a member is accumulated.

VALLEY FLOOR. An upwind reach of minimum length of 500 m which has a gradient of less than 1:20.

WEAK STOREY. A storey, the shear strength of which is 80 % or less of that of the storey above.

PART 2 GENERAL REQUIREMENTS

2.1 Notation

2.1.1
For the purposes of this Part of the Standard, symbols shall have the following meanings:

E_s Earthquake forces for the serviceability limit state (2.4.2)

E_u Earthquake forces for the ultimate limit state (2.4.3)

G Dead load (2.4.2, 2.4.3)

h_i Height of level i above the level where the ground provides lateral restraint to the structure (2.5.4.2)

h_n Height from the base of the building to the level of the uppermost principal seismic weight (2.5.4.5)

Q Reduced live load (2.4.2, 2.4.3)

Q_b Basic live load (2.4.2, 2.4.3)

Q_s Live load for the serviceability limit state (2.4.2)

Q_u Live load for the ultimate limit state for combinations of load involving dead load and live load with other loads (2.4.3)

S Basic snow load for roofs (2.4.3)

S_s Snow load, ice load, or rain load for the serviceability limit state (2.4.2)

S_u Snow load, ice load, or rain load for the ultimate limit state (for ice load, includes wind load on coated members) (2.4.3)

W_s Wind load for the serviceability limit state (2.4.2)

W_u Wind load for the ultimate limit state (2.4.3)

ψ_a Area reduction factor for live load (2.4.2, 2.4.3)

ψ_l Long term factor for the serviceability limit state live loads, snow, ice, or rain loads (2.4.2)

ψ_s Short term factor for the serviceability limit state live loads, snow, ice, or rain loads (2.4.2)

ψ_u Live load combination factor for the ultimate limit state (2.4.3)

2.2 Application

2.2.1 *General*

2.2.1.1
Each building and its parts shall be analysed by one of the prescribed methods for the relevant combinations of factored loads and forces specified for each of the serviceability and ultimate limit states.

2.2.1.2
Loads and forces are determined from Parts 3, 4, 5 and 6, as appropriate to the load type and the classification of the building or part. Any other effect which may significantly affect the serviceability, strength, or stability of the structure shall be taken into account.

2.2.1.3
Buildings and their parts are classified in 2.3.

2.2.1.4
Combinations of loads and forces are specified in 2.4.

2.2.1.5
Methods of analysis are prescribed in 2.5.

2.2.1.6
Structures shall satisfy the requirements specified in this Standard, together with those specified in the appropriate material standards.

2.2.2 *Special studies*
Special studies, required by 1.1.2 or undertaken at the initiative of the designer, shall demonstrate that the building and its parts have low probability values of reaching any limit state.

2.3 Classification of buildings and parts

2.3.1 *Buildings*
Buildings shall be classified in accordance with table 2.3.1.

Table 2.3.1 – Classification of buildings

Category	Description
I	Buildings dedicated to the preservation of human life or for which the loss of function would have a severe impact on society.
II	Buildings which as a whole contain people in crowds.
III	Publicly owned buildings which house contents of a high value to the community.
IV	Buildings not included in any other category.
V	Buildings of a secondary nature.

2.3.2 *Parts of buildings*
Parts supported by buildings and the connections of the parts shall be classified in accordance with table 2.3.2. Connections shall be assigned the same category as the connected part.

Table 2.3.2 – Classification of parts of buildings

Category	Description
P.I	Parts, the failure of which could cause a life hazard.
P.II	Parts for which continuing function is important.
P.III	Other parts.

2.4 Load combinations

2.4.1 *General*

In the analysis for each limit state, load cases shall include the appropriate set of the following sets of combinations of factored loads and forces, and such additional cases as special circumstances may require. Inclusion of soil loads and hydrostatic water loads into these combinations shall be in accordance with Part 6.

2.4.2 *Serviceability limit state*

2.4.2.1

The building as a whole and all its members shall be designed for the combinations of loads in 2.4.2.2. In these combinations the live load and snow load (or rain or ice load, as the case may be) for the serviceability limit state, Q_s and S_s, shall be derived as follows, where the short term and long term factors, ψ_s and ψ_l, are as given in table 2.4.1.

(a) The live load for the serviceability limit state, Q_s, shall be obtained by multiplying the reduced live load, Q (given in Part 3 as the basic live load, Q_b, multiplied by the area reduction factor, ψ_a), by the short term or long term factor, as follows:

- (i) For combinations of dead load and live load only, the short term or long term factor shall be used, as appropriate to the combination considered.
- (ii) For combinations of dead load and live load with other loads or forces, the long term factor alone may be used.

(b) The snow load (or rain or ice load, as the case may be) for the serviceability limit state, S_s, shall be obtained by multiplying the load given in Part 6 by the short term or long term factor, as appropriate to the combination considered.

Table 2.4.1 – Short term and long term load factors, ψ_s & ψ_l for the serviceability limit state (see also 3.5.1 and 3.6.1)

Type of load	Short term factor (ψ_s)	Long term factor (ψ_l)
Live load		
Floors, domestic	0.7	0.4
Floors, offices	0.7	0.4
Floors, parking	0.7	0.4
Floors, retail	0.7	0.4
Floors, storage	1.0	0.6
Floors, other	As for storage, unless assessed otherwise	
Roofs	0.7	0.0
Snow load		
All cases	0.5	0.0

2.4.2.2

Combinations of loads for the serviceability limit state shall include the following:

(1) G & Q_s

(2) G & Q_s & E_s

(3) G & Q_s & W_s

(4) G & S_s

2.4.2.3
For all combinations of load listed in 2.4.2.2, likely combinations of internal strain effects shall be considered.

2.4.2.4
For combinations of load listed in 2.4.2.2 not involving earthquake or wind, the most adverse likely distribution of live load or of superimposed dead load shall be considered.

2.4.3 *Ultimate limit state*

2.4.3.1
The building as a whole and all its members shall be designed to support the combinations of factored loads and forces in 2.4.3.3 to 2.4.3.6 inclusive.

2.4.3.2
Where provided for in these combinations, the factored live load, for the ultimate limit state, Q_u, shall be obtained by multiplying the reduced live load, Q (given in Part 3 as the basic live load, Q_b, multiplied by the area reduction factor, ψ_a) by the live load combination factor, ψ_u, given in table 2.4.2.

Where provided for in these combinations, the snow load for the ultimate limit state, S_u, shall be taken equal to S from Part 6.

Table 2.4.2 – Live load combination factor, ψ_u, for the ultimate limit state

Type of live load	Combination factor (ψ_u)
Floors, domestic	0.4
Floors, office	0.4
Floors, parking	0.4
Floors, retail	0.4
Floors, storage	0.6
Floors, other	As for storage, unless assessed otherwise
Roofs	0.0

2.4.3.3
The combinations of factored loads and forces for the ultimate limit state shall include the following:

(1) 1.4G

(2) 1.2G & 1.6Q

(3) 1.2G & Q_u & W_u

(4) 0.9G & W_u

(5) 1.2G & Q_u & 1.2S_u

(6) G & Q_u & E_u

2.4.3.4
Strength and stability in fire emergency conditions and afterwards shall comply with (a) and (b) following:

(a) For that period of time during fire emergency conditions when the structure is subject to elevated temperatures and designated members are required to remain stable, the affected members shall be designed for the following combination of factored load.

(7) G & Q_u

(b) The stability of elements which could collapse onto adjacent household units or other properties shall be ensured:

Either by designing the element and supporting structure to resist the loads in combination (7) above, using a detailed stress analysis which considers elevated temperatures and appropriate structural deformations throughout the fire.

Or, as an approximation, by designing the element and an appropriately fire rated supporting structure so that after a fire the residual structure at ambient temperatures is able to resist the loads in combination (7) above, plus a uniformly distributed face load on the residual structure of 0.5 KPa.

2.4.3.5
The most adverse distribution of live load and of superimposed dead load shall be considered in design. For load combinations involving wind, earthquake, or fire emergency conditions, such adverse distributions need not include cases of loading on alternate spans of continuous beams or slabs.

2.4.3.6
When capacity design is required, as provided in 2.5.4.6, the gravity load assumed to be present with the earthquake effects generated by overstrength of members need not be other than G & Q_u.

2.5 Analysis and design

2.5.1 *General*

2.5.1.1
Methods of analysis and the properties of the members assumed in the analysis shall be appropriate to the form of the structure, the materials to be used in construction, and the limit state being considered.

2.5.1.2
Reduced resistance and increased deformation of the structure resulting from lateral deflection of members shall be considered.

2.5.1.3
When the effects of an element on the behaviour of the structure cannot be determined with confidence, that element shall not be considered to contribute to the load-resisting system. A cautious appraisal of the effect of the element on actions and displacements shall be made. Where the element is required to support gravity loads, but is assumed not to provide resistance to lateral forces, the effect of displacement of the structure on the resistance of the element shall be considered.

2.5.1.4
Consideration shall be given to the effects of flexibility of the foundations on the response of the building and on the allocation of shear to the lateral force resisting elements.

2.5.2 *Serviceability limit state*

2.5.2.1
For the serviceability limit state, the deflections of the structure shall not be such as to result in damage causing loss of function of the structure or its parts.

2.5.2.2
Where departure from linear elastic behaviour is assumed in analysis, the effect of the resulting deformations on serviceability shall be considered.

2.5.2.3
Where a floor system may be used for group rhythmic activity, such as dancing, concerts, jumping exercises or gymnastics, and has a fundamental frequency of vibration less than 8 Hz, the effects of resonance shall be investigated by means of a dynamic analysis to demonstrate that the building remains functional.

2.5.3 *Ultimate limit state – General*

2.5.3.1
Except as provided in 2.5.3.2, it shall be demonstrated that strains in any element calculated in accordance with established principles of elasticity and plasticity, do not exceed the strain capacity of the element. For the purposes of this clause, strain shall include generalised strains or deformations, such as curvature or plastic hinge rotation in regions of flexural yield or plastic elongation in regions of tensile yield.

2.5.3.2
The requirements of 2.5.3.1 do not apply when strength is calculated in accordance with the provisions of the material standards, and any of the following is additionally satisfied:

(a) The analysis is based on the elastic method, or

(b) The analysis is based on the elastic method modified by redistribution of flexural actions, and the extent of redistribution satisfies the limits specified in the material standards, or

(c) The analysis is based on the plastic method for structures or elements for which the method is permitted in the material standards.

2.5.3.3
Except as specified in 2.5.4.6, structural resistance shall be based on the dependable strength calculated in accordance with the provisions of the material standards. The dependable strength of supporting soils shall be assessed from measured soil strength parameters determined from appropriate site investigation techniques or from other reliable data, and a strength reduction factor which shall not be assumed greater than 0.6.

2.5.3.4
For combinations of factored loads not involving earthquake, the structure and its parts shall be designed to prevent instability due to overturning, sliding or uplift, as follows:

(a) For each type of potential instability, a mechanism, consistent with the potential instability, shall be assumed;

(b) Factored loads and forces as specified in 2.4.3.3 shall be applied (with superimposed dead loads and live loads treated as in 2.4.3.5);

(c) Loads and forces shall be subdivided into those tending to cause instability and those tending to resist instability;

(d) Stabilizing and destabilizing actions (moments about the centre of rotation in the case of overturning, forces parallel to the plane of sliding in the case of sliding, forces parallel to the direction of uplift in the case of uplift) shall be separately determined;

(e) Actions tending to cause instability shall be factored as provided in 2.4.3.3;

(f) Actions tending to resist instability shall be scaled to be consistent with a load factor of 0.9;

(g) Dependable strength (moment, shear, or axial force, as appropriate), if mobilized, may be added to the stabilizing actions;

(h) The sum of the actions resisting instability (factored as in (f) above) together with mobilized dependable strength (if any), shall not be less than the sum of the actions tending to cause instability.

2.5.4 *Ultimate limit state – Additional requirements involving earthquake effects*

2.5.4.1
Whenever the structural ductility factor forms a control for displacements, actions, and the like, its actual value which is consistent with the strength and deformation characteristics of the structure as detailed may be substituted for the initial design value.

2.5.4.2
(a) Under the earthquake effects specified for the ultimate limit state in 4.6 and 4.7 the deflections of the structure shall not be such as to:

   (i) Endanger life

   (ii) Cause loss of function to Category I or II buildings or Category P.I or P.II parts, or cause damage to the contents of Category III buildings

   (iii) Cause contact between parts if such contact would damage the parts to the extent that persons would be endangered, or detrimentally alter the response of the structure or reduce the strength of structural elements below the required strength

(iv) Exceed building separation from site boundaries or between neighbouring buildings on the same site (see (b) below); or

(v) Cause loss of structural integrity.

(b) All buildings shall be so located that the distance from their perimeter at levels above the ground to the vertical planes of the boundaries of adjacent sites is not less than the deflection at that level calculated in accordance with 4.7.3, nor less than 0.002 h_i or 12 mm. This requirement does not apply to street frontages or other boundaries where there is an effective assurance that no future structure can be erected adjacent to the boundary. Adjacent structures on the same site shall be separated at all levels by the sum of the deflections calculated for each structure at that level in accordance with 4.7.3.

2.5.4.3

In determining the magnitude of inelastic strains, as provided for the general analytic procedure in 2.5.3.1, the structure shall be taken as displaced as specified in 4.7.3.

2.5.4.4

Consideration shall be given to the consequences of possible yielding of components of the foundation structure or soil and of rocking or uplift of spread footings on the response and energy dissipation characteristics of the building (see also 4.11.1.2).

2.5.4.5

Deflections, calculated in accordance with 4.7.3, shall satisfy the limits in (a) or (b) as appropriate:

(a) Where the equivalent static or modal response spectrum method of analysis is used, interstorey deflections shall not exceed the following fractions of the corresponding storey height:

(i) 0.020 for $h_n \leq 15$ m

(ii) 0.015 for $h_n \geq 30$ m

(iii) $0.020 - 0.005 \dfrac{h_n - 15}{15}$ for $15 \text{ m} < h_n < 30$ m

(b) Where the numerical integration time history method incorporating inelastic member response is used, interstorey deflections shall not exceed 0.025 of the corresponding storey height.

2.5.4.6

Capacity design shall be applied to all structures for which earthquake effects have been derived assuming a ductility factor greater than 3.0. Capacity design may be applied to any structure or part of a structure to limit strength demands otherwise required by this clause.

For capacity design of elements, including joints and foundations, which are intended to remain elastic when the chosen energy dissipating mechanism develops, then in accordance with the provisions of the appropriate material standard:

(a) Account shall be taken of the development of overstrength in intended inelastic modes in members;

(b) Where the elements are part of a two-way horizontal force resisting system, account shall be taken of the effects of potential concurrent yielding of other elements framing into them from all directions at the level under consideration and as appropriate at other levels;

(c) Account shall be taken of the effects of dynamic response in the allocation of resistance to moments and forces induced by the yielding members, and

(d) Ideal strength may be assumed to provide the resistance to the actions so derived.

For all other structures non-ductile failure modes shall be suppressed by the provision of dependable strength. The dependable strength provided shall be based on earthquake forces derived assuming elastic response and on the provisions of the appropriate material standard.

PART 3 DEAD AND LIVE LOAD PROVISIONS

3.1 Notation

3.1.1
For the purposes of this Part of the Standard, symbols shall have the following meanings:

A	Tributary area of floor, the area from which the total load supported by the member is accumulated (3.4.2.1)
Q	Reduced live load (3.4.2.1)
Q_b	Basic live load (3.4.1.1)
ψ_a	Area reduction factor for live load (3.4.2)
ψ_u	Live load combination factor for the ultimate limit state (3.6.1.1).

3.2 General requirements

3.2.1
In all suspended workrooms and storage areas the live load appropriate to each section of the floor shall be displayed on a tablet. Refer to clause F8, "Signs" of the New Zealand Building Code.

3.3 Dead loads

3.3.1
Dead loads shall be calculated from the design or known dimensions of the structure and the unit weights of the construction.

3.3.2
The specified dead load for a structural member shall consist of:

(a) The self weight of the member;

(b) The weight of all materials of construction incorporated into the building to be supported permanently by the member;

(c) The weight of the partitions;

(d) The weight of permanent equipment.

3.3.3
The loads due to movable and future partitions shall be determined from the actual or anticipated weight of the partitions placed in any probable position. Such loads may be considered as a uniformly distributed load over the area of the floor being considered of not less than one third of the mass per metre length of the partition.

3.4 Live loads

3.4.1 *General*

3.4.1.1
The basic distributed live load and basic concentrated live load, Q_b, for particular occupancies and uses of each space or room shall be as set out in table 3.4.1.

3.4.1.2
The basic uniform and concentrated live loads may be considered separately and design carried out for whichever gives the more adverse effect.

3.4.1.3
Concentrated live loads shall be applied over the actual area of application where known. Where the area of the application is not known, the basic concentrated live load (table 3.4.1) shall be distributed over an area of not greater than 0.3 m x 0.3 m for floors and an area of not greater than 0.1 m diameter for roofs and applied in the position giving the most adverse effect.

3.4.1.4
Except as provided in 2.4.3.5 and 2.4.3.6, it shall be assumed that the prescribed load can be absent from any part or parts of a structure if its absence therefrom will cause a more adverse effect on that or any other part.

3.4.1.5
Where the occupancy of an area of floor or roof is not provided for in table 3.4.1, the live loads shall be determined as appropriate from an analysis of the loads resulting from:

(a) The assembly of persons;

(b) The accumulation of equipment and furnishings, and

(c) The storage of materials.

3.4.1.6
Special loads imposed during construction or maintenance shall also be considered.

Table 3.4.1 – Basic live loads for floors and stairs

Category		Spatial occupancy	Q_b Distr kPa	Q_b Conc kN
1 Domestic				
	1.1	Non-habitable roof spaces	0.5	1.8
	1.2	Balconies	2.0	1.8
	1.3	Other rooms, including service rooms	1.5	1.8
	1.4	Garages	2.5	9.0
2 Residential				
	2.1	Balconies	4.0	1.8
	2.2	Bars and public lounges	3.0	2.7
	2.3	Bedrooms	1.5	1.8
	2.4	Dining rooms	3.0	2.7
	2.5	Corridors, stairs, landings	3.0	4.5
	2.6	Other rooms, except service rooms	3.0	2.7
3 Educational				
	3.1	Class and lecture rooms	3.0	2.7
	3.2	Laboratories	3.0[(1)]	4.5[(1)]
	3.3	Library reading areas	3.0	2.7
	3.4	Library stacks:		
		Not exceeding 1.8 m high	4.0	4.5[(1)]
		For each additional 0.3 m, add	0.5	
4 Institutional				
	4.1	Bedrooms and wards	2.0	1.8
	4.2	Operating theatres	3.0[(1)]	4.5[(1)]
	4.3	Utility rooms	3.0	2.7
	4.4	Heavy equipment rooms	3.0[(1)]	4.5[(1)]
5 Assembly				
	5.1	Assembly areas, fixed seating	3.0	2.7
	5.2	Assembly areas, moveable seating	5.0[(1)]	3.6[(1)]
	5.3	Grandstands, fixed seating[(2)]	4.0	3.6
	5.4	Grandstands, moveable seating[(2)]	5.0	4.5
	5.5	Law courts	3.0	3.6
	5.6	Stages	5.0	3.6
6 Office				
	6.1	Banking chambers	4.0	4.5
	6.2	Offices for general use	2.5[(1)]	2.7[(1)]
7 Retail				
	7.1	Shop floors	4.0	3.6
8 Industrial				
	8.1	Workrooms without plant	2.5	2.7
	8.2	Workrooms with lightweight plant (no item more than 5 kN)	3.0	3.6
	8.3	Other workrooms	5.0[(1)]	4.5[(1)]
	8.4	Broadcasting studios	4.0	3.6
	8.5	Printing plants	12.5[(1)]	[(3)]
9 Access, Service				
	9.1	Corridors, Stairways: pedestrian As for floor serviced, but need not be greater than	5.0	4.5
	9.2	Corridors, passageways: vehicle	5.0[(1)]	9.5[(1)]
	9.3	Pedestrian bridges	4.0	3.6
	9.4	Plant rooms[(4)]	5.0[(1)]	4.5[(1)]
	9.5	Toilet and locker rooms	2.0	1.8

[(1)], [(2)], [(3)], [(4)] See notes at end of table

Table 3.4.1 (Continued)

Category	Spatial occupancy	Q_b Distr kPa	Q_b Conc kN
10 Storage			
	10.1 File and store rooms	5.0[(1)]	4.5[(1)]
	10.2 Mobile file storage rooms	7.0[(1)]	
	10.3 Vaults and strongrooms	5.0[(1)]	4.5[(1)]
	10.4 Cold storage	[(3)]	
	10.5 Timber pallets (per pallet)	7.2	
	– lamb carcasses	8.7	
	– mutton carcasses	8.7	
	– cartoned beef	14.1	
	10.6 Fly galleries	4.5 kN/m	
	10.7 Restaurants	3.0	2.7
	10.8 Parking areas and ramps:		
	– vehicles less than 2500 kg tare	2.5	9.0
	– vehicles above 2500 kg tare	5.0[(1)]	9.0[(1)]
11 Roofs			
	11.1 No access for pedestrian traffic	0.25 (on plan)	1.0
	11.2 Roof claddings only:		
	– slopes ≤ 30°	1.1	
	– slopes > 30°	0.5	
	11.3 Access for pedestrian traffic:		
	– dwellings	1.5	1.4
	– other	2.0	1.4
	11.4 Construction and demolition sites – Gantry roofs over public ways (refer to Approved Document F5 to the NZBC):		
	– where materials are stacked on, or crane loads are carried over the roof	7.0[(1)] +impact[(5)]	0.5[(1)]
	– where the roof supports a site office	3.5[(1)] +impact[(5)]	0.5[(1)]
12 Agriculture			
	12.1 Cattle pens	4.0	
	12.2 Sheep pens	1.5	
	12.3 Horse pens	5.0	
	12.4 Pig pens	2.0	
	12.5 Chicken coops	2.0	

NOTE – The live load shall be determined on the basis of occupancy of each room or space.

(1) To be calculated but not less than the value given.

(2) Refer to 3.4.3.2 for horizontal live loads.

(3) To be calculated.

(4) This live load is to apply to the floor space surrounding specific items of machinery. Where the weight of machinery is not known a live load of 7.5 kPa shall be used for the entire floor.

(5) Allow for impact of a compact mass equal to that of the concentrated load specified falling from the top of the construction. The contact area of the mass shall be as required by 3.4.1.3.

3.4.2 *Reduced live load*

3.4.2.1
The basic distributed live load, Q_b, may be reduced by multiplying by ψ_a to give the reduced live load, Q. Subject to 3.4.2.2, ψ_a shall be determined as follows;

(a) For storage, service and retail occupancies:

$$1.0 \geq \psi_a = 0.5 + 4.6/\sqrt{A} \quad \text{(Eq. 3.4.1)}$$

(b) For other occupancies:

$$1.0 \geq \psi_a = 0.4 + 2.7/\sqrt{A} \quad \text{(Eq. 3.4.2)}$$

3.4.2.2
ψ_a shall be taken as 1.0 for:

(a) One-way slabs except where it can be demonstrated that the unreduced load on the area under consideration can be supported by the whole of that area in two way action;

(b) Areas where the live load exceeds 5.0 kPa and results from storage;

(c) Assembly areas;

(d) Roofs with no pedestrian access;

(e) Live loads from machinery and equipment for which specific design allowance has been made.

3.4.3 *Additional considerations*

3.4.3.1 *Ceiling framing*
Ceiling joists and immediate supporting members in ceiling spaces with access for maintenance only shall be designed to support a point load, Q = 1 kN at any location. This load need not be applied at the same time as the roof live load.

3.4.3.2 *Stadiums and the like*
In addition to other design requirements of this Standard, grandstands, stadiums, assembly platforms, reviewing stands, and the like shall be designed to resist a horizontal force applied to seats of Q = 350 N per linear metre along the line of the seats and Q = 150 N per linear metre perpendicular to the line of the seats. These loadings need not be applied simultaneously. Platforms without seats shall be designed to resist a minimum horizontal force of Q = 250 N per square metre of plan area (0.25 KPa). The horizontal loadings of this clause need not be added to the required seismic horizontal forces.

3.5 Balustrades and parapets

3.5.1 *Serviceability limit state*
The reduced live load, Q, acting on balustrades, handrails and parapets shall be as given in table 3.5.1. The short term and long term load factors (see 2.4.2.1) shall be taken as equal to 1.0.

3.5.2 *Ultimate limit state*
The reduced live load, Q, acting on balustrades, handrails and parapets shall be as given in table 3.5.1. The loads for top edge and infill may be applied separately. Loads specified in this clause and those due to wind and earthquake need not be assumed to act concurrently.

Table 3.5.1 – Reduced live loads, Q, for balustrades and parapets

	Top edge		Infill	
	Horizontal kN/m	**Vertical kN/m**	**Horizontal kPa**	**Any kN**
Light access stairs, gangways	0.22	0.22	N/A	N/A
Residential buildings	0.36	0.36	0.75	0.25
Other buildings & public areas of residential buildings	0.75	0.75	1.0	0.50
Theatres, cinemas, assembly halls, stadiums etc.	3.0	0.75	1.5	0.50

Horizontal and vertical loads need not be assumed to act concurrently.

3.6 Vehicle barriers for parking areas

3.6.1 *Ultimate limit state*

3.6.1.1
In the absence of wheel stops, the reduced live load, Q, for vehicle barriers at parking bays in car parks shall be a horizontal force of 10 kN spread over any 1.5 m length and applied at 375 mm above the floor and normal to the line of the barrier. The value of the live load combination factor, ψ_u (see 2.4.3.2) shall be taken as 1.0.

3.6.1.2
For vehicles other than cars, a special study shall be used to determine the barrier design forces.

3.7 Machinery live loads

3.7.1 *Vertical forces*
The effect on the strength of buildings or parts of buildings from moving live loads such as cranes, lifts or machinery shall be provided for by an assumed increase in such live loads. The minimum increase shall be:

(a) For support of lifts .... 100 %

(b) For travelling crane gantry girders and their connections, the increase in maximum vertical static wheel loads shall be –

Electric overhead cranes .... 25 %
Hand operated cranes .... 10 %

(c) For supports of non-reciprocating machinery .... 20 %

(d) For supports of reciprocating machinery .... 50 %

3.7.2 *Horizontal forces*

3.7.2.1
The horizontal force, Q, acting transverse to crane rails shall be taken as a percentage of the combined weight of the crab and the load lifted as follows:

(a) For electric overhead cranes .... 10 %

(b) For hand operated cranes .... 5 %

3.7.2.2
Horizontal forces, Q, acting along the rails shall be taken as a percentage of the static wheel loads which can occur on the rails as follows:

For overhead cranes, either electric or hand operated .... 5 %

3.7.2.3
The forces specified in 3.7.2.1 and 3.7.2.2 shall be considered as acting at the rail head level, separately applied.

3.7.2.4
Gantry girders and their vertical supports shall be designed on the assumption that either of the horizontal forces specified in 3.7.2.1 or 3.7.2.2 may act at the same time as the augmented vertical load.

PART 4 EARTHQUAKE PROVISIONS

4.1 Notation

4.1.1
For the purposes of this Part of the Standard, symbols shall have the following meanings:

C	Lateral force coefficient for the equivalent static method (4.6.2)
$C(T)$	Ordinate of the design spectrum (given in units of g) at period T (4.6.2)
$C_h(T,\mu)$, $C_h(T_i,\mu)$	Basic seismic hazard acceleration coefficient which accounts for different soil conditions, structural ductility factors, μ, and translational periods of vibration, T or T_i, (4.6.2)
C_{fi}	Floor coefficient at level i (4.12.2)
C_{fn}	Floor coefficient at the level of the uppermost principal seismic weight (4.12.2)
C_{fo}	Floor coefficient at and below the base of the building (4.12.2)
C_{ph}	Basic horizontal seismic coefficient for a part (4.12.1)
C_{pi}	Basic horizontal seismic coefficient for a part at level i (4.12.1)
C_{pv}	Basic vertical seismic coefficient for a part (4.12.1)
F_i	Equivalent static lateral force at level i (4.8.1); or inertial force at level i found from combination of modal inertial forces (4.12.2)
F_n	Inertial force at the height of the uppermost principal seismic weight, h_n, used for the design of the structure, either the equivalent static force or the force from the combination of modal forces, as appropriate to the method of analysis used for the building structure (4.12.2)
F_{ph}	Horizontal seismic force on a part (4.12.1)
F_{pv}	Vertical seismic force on a part (4.12.1)
g	Acceleration due to gravity. To be taken as 9.81 m/s^2 (4.5.2)
h_i	Height of level i above the base of the structure (4.8.1.3)
h_n	Height from the base of the building to the level of the uppermost principal seismic weight (4.12.2)
i	A level or storey of a building. NOTE – Level i is immediately above storey i.
K_m	Modal analysis scaling factor (4.6.2.8)
L_s	Limit state factor for the serviceability limit state (4.6.2.6)
L_u	Limit state factor for the ultimate limit state (4.6.2.6)
n	The maximum value of i
Q	Reduced live load (4.2.6)

Q_u	Live load for the ultimate limit state as defined in Part 2 (4.12.1.5)
R	Risk factor for a structure, (4.6.2.4)
R_p	Risk factor for a part (table 4.12.1)
S_m,S_{m1},S_{m2}	Design spectrum scaling factors (4.6.2.8)
S_p	Structural performance factor (4.2.4.1)
T, T_i	Translational periods of vibration (4.5.1)
T_p	Fundamental translational period of vibration of a part for the direction being considered. (4.12.2.3)
T_{pe}	Equivalent period for a part for the direction being considered (4.12.2.3)
T_1	Fundamental translational period of vibration for the direction being considered (4.5.2)
u_{el}	Elastic component of the deflection of a building at the level of the uppermost principal seismic weight (4.7.3)
u_i	Lateral displacement of the centre of mass at level i (4.5.2)
V	Horizontal seismic shear force acting at the base of a structure (4.8.1)
$V_{base(1)}$	Combined modal shear at the base of a building determined for the direction being considered taking S_m =1 (4.6.2.8)
V_i	Storey shear in storey i, i.e. below level i (4.7.5)
W_i	Seismic weight at level i (4.2.6)
W_n	Seismic weight at height h_n (4.12.2)
W_p	Seismic weight of a part (4.12.1)
W_t	Total seismic weight of a structure (4.6.2.8, 4.8.1)
Z	Zone factor (4.6.2.5)
γ_i	Interstorey deflection for storey i divided by the storey height (4.7.5)
μ	Structural ductility factor (4.2.4.2)
μ_o	Structural ductility factor calculated using the overstrength values (4.12.2)
μ_p	Structural ductility factor used for the design of a part (4.12.2)
ψ_l	Long term factor for the serviceability limit state live loads (4.2.6) (see also table 2.4.1)
ψ_u	Live load combination factor for the ultimate limit state (4.2.6) (see also table 2.4.2)

4.2 General requirements

4.2.1 *Structural system*
Structures shall be designed with a clearly defined load path, or paths, to transfer the inertial forces generated in an earthquake to the supporting soils.

4.2.2 *Limit states*
All structures shall be designed to have:

(a) Adequate strength and stiffness to satisfy the serviceability limit state, and

(b) Adequate strength, ductility and stiffness to satisfy the ultimate limit state.

4.2.3 *Ductility and structural type*
To satisfy the ultimate limit state requirements of this Part, the structure shall be designed as:

(a) A ductile structure, or

(b) A limited ductile structure, or

(c) An elastically responding structure, or

(d) A combination of the above.

4.2.4 *Structural performance factor and structural ductility factor*

4.2.4.1 *Choice of structural performance factor*
The structural performance factor, S_p, shall be taken as equal to 0.67 unless specified otherwise in the appropriate limit state format material standard for the material or form of seismic resisting system under consideration.

4.2.4.2 *Choice of structural ductility factor*

(a) The structural ductility factor, μ, for the serviceability limit state shall be taken as 1.0.

(b) The structural ductility factor, μ, for the ultimate limit state shall be obtained from the appropriate limit state format material standard. Where such provisions are not available for the material or form of the seismic resisting system under consideration, the structural ductility factor for the ultimate limit state may be obtained or rationally deduced from the permissible values given in table 4.2.1, in accordance with the associated notes to that table and the provisions of an appropriate material standard or other published source.

4.2.5 *Capacity design*
Capacity design is required for all ductile structures, and for limited ductile structures only if required by the appropriate material standard (see 2.5.4.6).

4.2.6 *Seismic weight*
The seismic weight at each level, W_i, shall be taken as the sum of the dead loads and the seismic live loads between the mid heights of adjacent storeys. The seismic live load shall be determined from the product of ψ_l, for the serviceability limit state (from table 2.4.1), or ψ_u, for the ultimate limit state (from table 2.4.2), and Q (see 3.4.2) for each occupancy class on the floor under consideration. For roofs the seismic weight shall include allowance for ice, as specified in 6.5.4.

Table 4.2.1 – Structural ductility factor, μ (for use where required by 4.2.4.2(b))

	Structural steel	Reinforced concrete	Prestressed concrete	Reinforced masonry	Timber
1. Elastically responding structures	1.25	1.25	1.0	1.25	1.0
2. Structures of limited ductility					
(a) Braced frames:					
(i) Tension & compression yielding	3	–	–	–	3
(ii) Tension yielding only (Two storeys maximum)	3	–	–	–	–
(b) Moment resisting frame	3	3	2	2	3
(c) Walls	3	3	–	2	3
(d) Cantilevered face loaded walls (single storey only)	–	2	–	2	–
3. Ductile structures (see Note 6)					
(a) Braced frames (tension & compression yielding)	$6 \le 20(1-T_1) \le 10$	–	–	–	–
(b) Moment resisting frames	$6 \le 20(1-T_1) \le 10$	$6 \le 20(1-T_1) \le 10$	$5 \le 20(1-T_1) \le 8$	$4 \le 20(1-T_1) \le 6$	$4 \le 20(1-T_1) \le 6$
(c) Walls	–	$5 \le 20(1-T_1) \le 8$	–	$4 \le 20(1-T_1) \le 6$	$4 \le 20(1-T_1) \le 6$
(d) Eccentrically braced frames	$6 \le 20(1-T_1) \le 10$	–	–	–	–

NOTES TO TABLE

(1) Except where permitted by the provisions of the appropriate material standard, or published source, the specified value of the structure ductility factor is the maximum which may be adopted.

(2) The seismic forces associated with these basic structural ductility factors may be further modified to account for differing levels of inelastic performance with varying numbers of storeys and seismic resisting system characteristics. Refer to the appropriate source standard for guidance.

(3) Refer to the appropriate source for design and detailing requirements for each category of ductility. These requirements must be complied with to ensure satisfactory behaviour for the level of ductility chosen.

(4) For timber structures exhibiting brittle modes of failure use 1.0. Glued timber structures are in this category. All other categories of timber structure included in this table require inelastic deformations to be developed in connections suitably designed and detailed for the anticipated level of inelastic response.

(5) For mixed systems, comprising different forms of seismic-resisting systems acting in parallel for a given direction of loading, the design seismic force on each system shall be determined by a rational assessment, taking into account the relative stiffness and structural ductility factor associated with each system.

(6) For ductile structures, the selection of a structural ductility factor, for design, greater than the lower value of a specified range is not expected to attract detailing penalties in the material standards.

4.3 Methods of analysis

4.3.1 *General*
Analysis for the design earthquake actions shall be in accordance with one of the following methods:

(a) The equivalent static method as outlined in section 4.8,

(b) The modal response spectrum method as outlined in section 4.9, or

(c) The numerical integration time history method as outlined in section 4.10.

4.3.2 *Limitation on the use of the equivalent static method*
The equivalent static method of analysis may be used only where at least one of the following criteria is satisfied:

(a) The height between the base and the top of the structure does not exceed 15 m,

(b) The fundamental period calculated as specified in section 4.5 does not exceed 0.45 seconds

(c) The structure satisfies the horizontal and vertical regularity requirements of section 4.4 and has a fundamental period less than 2.0 seconds.

4.3.3 *Use of dynamic analyses*
Modal response spectrum analyses or numerical integration time history analyses shall be used for structures which do not comply with 4.3.2. Where the horizontal regularity criteria of 4.4.1 are not met, such analyses shall be three-dimensional analyses.

4.4 Structural regularity

4.4.1 *Horizontal regularity*
A structure satisfies the horizontal regularity requirement for the use of the equivalent static or two-dimensional modal response spectrum and two-dimensional numerical integration time history methods of analysis where the following criteria are satisfied:

(a) Either

(i) The horizontal distance between the shear centre at any level and the centre of mass of all levels above shall neither exceed 0.3 times the maximum plan dimension of the structure at that particular level, measured perpendicular to the direction of the applied lateral forces, nor change sign over the height of the structure, or

(ii) Under the action of the equivalent static lateral forces as defined in 4.8.1 and 4.8.2, the ratio of the horizontal displacements at the ends of an axis transverse to the direction of the applied lateral forces shall be in the range 3/7 to 7/3.

and

(b) The diaphragms shall not contain abrupt variations in stiffness or major re-entrant corners such as could significantly influence the distribution of the lateral forces in the structure.

4.4.2 *Vertical regularity*
To satisfy the vertical regularity requirements for the use of the equivalent static method, the lateral displacement of each level shall be reasonably proportional to the height of the level above the base.

4.5 Period of vibration

4.5.1

The periods of vibration, T_i, shall be established from properly substantiated data or computation or both.

4.5.2

Where the equivalent static method of analysis is used, the fundamental translational period in the direction under consideration shall be calculated from the Rayleigh method, as given by equation 4.5.1 or from an equivalent method.

$$T_1 = 2\pi \sqrt{\frac{\Sigma(W_i \ u_i^2)}{g \ \Sigma(F_i \ u_i)}} \qquad \text{.......... (Eq. 4.5.1)}$$

where u_i may be calculated ignoring the effects of torsion.

4.6 Seismic design actions

4.6.1 *General*

The seismic actions for the serviceability and ultimate limit states shall be evaluated using one of the methods of analysis given in section 4.3. The design spectra and lateral force coefficients shall be derived in accordance with 4.6.2.

4.6.2 *Design spectra and lateral force coefficients*

4.6.2.1 *Basic seismic hazard acceleration coefficient*

The basic seismic hazard acceleration coefficient, $C_h(T,\mu)$, shall be selected from parts (a), (b) or (c) of figure 4.6.1 and table 4.6.1, as appropriate to the site subsoil category (a), (b) or (c) as defined in 4.6.2.2. Provided that $C_h(T,\mu)$, need not be taken greater than $C_h(0.45,\mu)$ for site subsoil categories (a) and (b) nor greater than $C_h(0.6,\mu)$ for site subsoil category (c).

4.6.2.2 *Site subsoil categories*

Site subsoil category (a) (Rock or very stiff soil sites)

Sites where the low amplitude natural period is less than 0.25 s, or sites with bedrock, including weathered rock, with unconfined compressive strength greater than or equal to 500 kPa, or with bedrock overlain by:

(i) Less than 20 m of very stiff cohesive material with undrained shear strength exceeding 100 kPa; or

(ii) Less than 20 m of very dense sand, with N_1 >30, where N_1 is the SPT (N) value corrected to an effective overburden pressure of 100 kPa; or

(iii) Less than 25 m of dense sandy gravel with N_1 >30.

Site subsoil category (b) (Intermediate soil sites)

Sites not described as category (a) or (c) may be taken as intermediate soil sites.

Site subsoil category (c) (Flexible or deep soil sites)

Sites where the low amplitude natural period exceeds 0.6 s, or sites with depths of soils exceeding the following values:

Soil type and description		Depth of soil (m)
Cohesive soil	Representative undrained shear strengths (kPa)	
Soft	12.5 – 25	20
Firm	25 – 50	25
Stiff	50 – 100	40
Very stiff	100 – 200	60
Cohesionless soil	Representative SPT (N) values	
Loose	4 – 10	40
Medium dense	10 – 30	45
Dense	30 – 50	55
Very dense	> 50	60
Gravels	> 30	100

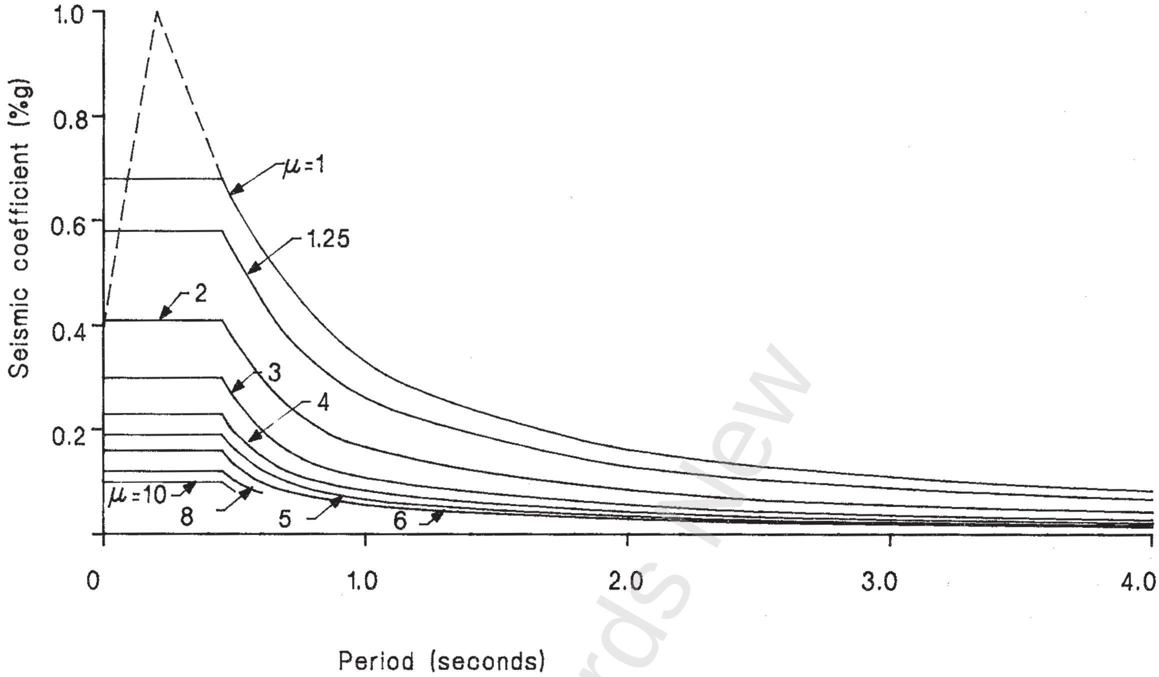


Figure 4.6.1 – Basic seismic hazard acceleration coefficient (a) Site subsoil category (a) (Rock or very stiff soil sites)

Table 4.6.1 – Basic seismic hazard acceleration coefficient (a) Site subsoil category (a) (Rock or very stiff soil sites)

Period, T (seconds)	**Structural ductility factor, μ**								
	1.0	1.25	2.0	3.0	4.0	5.0	6.0	8.0	10.0
0	0.40* 0.68	0.58	0.41	0.30	0.23	0.19	0.16	0.12	0.10
0.09	0.68	0.58	0.41	0.30	0.23	0.19	0.16	0.12	0.10
0.20	1.00**	0.58	0.41	0.30	0.23	0.19	0.16	0.12	0.10
0.45	0.68	0.58	0.41	0.30	0.23	0.19	0.16	0.12	0.10
0.50	0.63	0.54	0.37	0.26	0.20	0.16	0.14	0.11	0.082
0.60	0.55	0.45	0.30	0.20	0.15	0.13	0.10	0.08	
0.70	0.48	0.38	0.24	0.16	0.12	0.10	0.082		
0.80	0.42	0.34	0.21	0.14	0.11	0.084	0.071		
0.90	0.37	0.30	0.19	0.12	0.093	0.074	0.063		
1.0	0.33	0.26	0.17	0.11	0.083	0.066	0.056		
1.5	0.23	0.18	0.12	0.076	0.058	0.046	0.039		
2.0	0.17	0.14	0.085	0.056	0.043	0.034	0.029		
2.5	0.13	0.10	0.065	0.043	0.033	0.026	0.022		
3.0	0.11	0.088	0.055	0.036	0.028	0.022	0.019		
4.0	0.083	0.066	0.042	0.027	0.021	0.017	0.014		

NOTE – For intermediate periods and ductility factors, interpolate linearly.

Figures below the stepped line may be limited by 4.6.2.7(b) when R = 1.0.

* This value shall be used only for the derivation of the design spectra in accordance with 4.6.2.8 and 4.6.2.9.

** This value need not be taken greater than 0.68 (see 4.6.2.1).

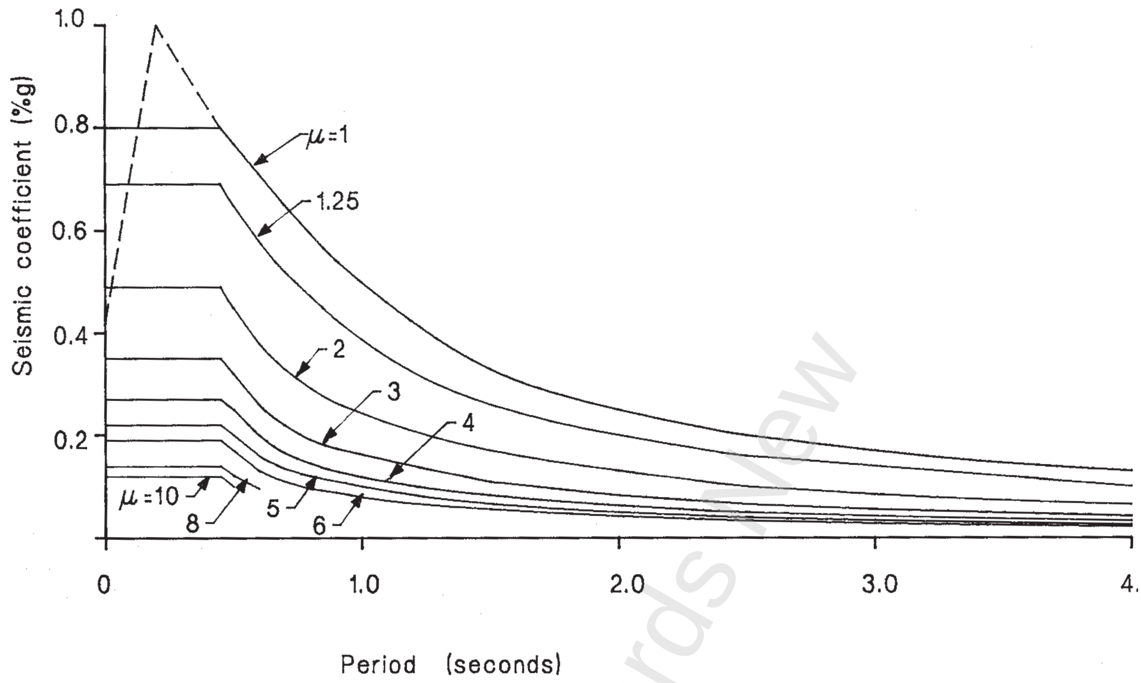


Figure 4.6.1 – Basic seismic hazard acceleration coefficient
(b) Site subsoil category (b) (Intermediate soil sites)

Table 4.6.1 – Basic seismic hazard acceleration coefficient
(b) Site subsoil category (b) (Intermediate soil sites)

Period, T or T_{pe} (seconds)	**Structural ductility factor, μ**								
	1.0	1.25	2.0	3.0	4.0	5.0	6.0	8.0	10.0
0	0.42* 0.80	0.69	0.49	0.35	0.27	0.22	0.19	0.14	0.12
0.13	0.80	0.69	0.49	0.35	0.27	0.22	0.19	0.14	0.12
0.20	1.00**	0.69	0.49	0.35	0.27	0.22	0.19	0.14	0.12
0.45	0.80	0.69	0.49	0.35	0.27	0.22	0.19	0.14	0.12
0.50	0.77	0.65	0.45	0.32	0.25	0.20	0.17	0.13	0.10
0.60	0.71	0.58	0.38	0.26	0.20	0.16	0.13	0.10	
0.70	0.65	0.52	0.33	0.22	0.17	0.13	0.11		
0.80	0.60	0.48	0.30	0.20	0.15	0.12	0.10		
0.90	0.55	0.44	0.28	0.18	0.14	0.11	0.094		
1.0	0.50	0.40	0.25	0.17	0.13	0.10	0.085		
1.5	0.33	0.26	0.17	0.11	0.083	0.066	0.056		
2.0	0.25	0.20	0.13	0.083	0.063	0.050	0.043		
2.5	0.20	0.16	0.10	0.066	0.050	0.040	0.034		
3.0	0.17	0.14	0.085	0.056	0.043	0.034	0.029		
4.0	0.13	0.10	0.065	0.043	0.033	0.026	0.022		

NOTE – For intermediate periods and ductility factors, interpolate linearly.

Figures below the stepped line may be limited by 4.6.2.7(b) when R = 1.0.

* This value shall be used only for the derivation of the design spectra in accordance with 4.6.2.8 and 4.6.2.9.

** This value need not be taken greater than 0.8 (see 4.6.2.1).

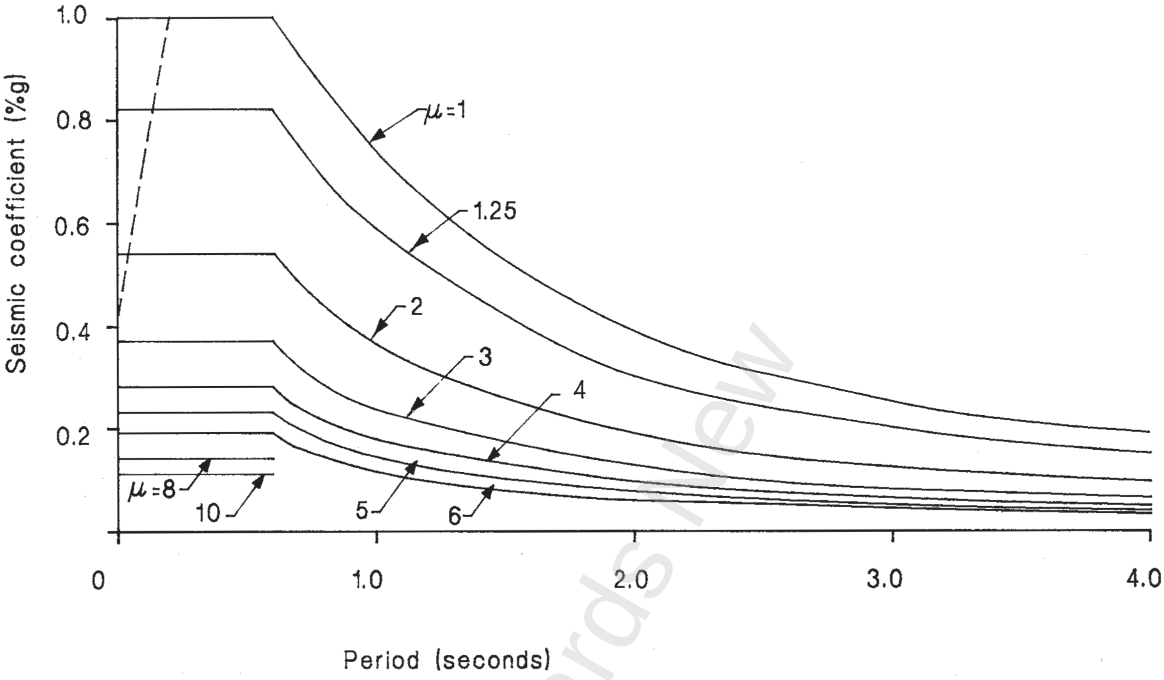


Figure 4.6.1 – Basic seismic hazard acceleration coefficient
(c) Site subsoil category (c) (Flexible or deep soil sites)

Table 4.6.1 – Basic seismic hazard acceleration coefficient
(c) Site subsoil category (c) (Flexible or deep soil sites)

Period, T (seconds)	Structural ductility factor, μ								
	1.0	1.25	2.0	3.0	4.0	5.0	6.0	8.0	10.0
0.0	0.42* 1.00	0.82	0.54	0.37	0.28	0.23	0.19	0.14	0.11
0.10	0.72* 1.00	0.82	0.54	0.37	0.28	0.23	0.19	0.14	0.11
0.20	1.00	0.82	0.54	0.37	0.28	0.23	0.19	0.14	0.11
0.45	1.00	0.82	0.54	0.37	0.28	0.23	0.19	0.14	0.11
0.50	1.00	0.82	0.54	0.37	0.28	0.23	0.19	0.14	0.11
0.60	1.00	0.82	0.54	0.37	0.28	0.23	0.19	0.14	0.11
0.70	0.94	0.75	0.47	0.31	0.24	0.19	0.16		
0.80	0.88	0.70	0.44	0.29	0.22	0.18	0.15		
0.90	0.81	0.65	0.41	0.27	0.20	0.16	0.14		
1.0	0.75	0.60	0.38	0.25	0.19	0.15	0.13		
1.5	0.52	0.42	0.26	0.17	0.13	0.10	0.088		
2.0	0.38	0.30	0.19	0.13	0.095	0.076	0.065		
2.5	0.30	0.24	0.15	0.099	0.075	0.060	0.051		
3.0	0.25	0.20	0.13	0.083	0.063	0.050	0.043		
4.0	0.19	0.15	0.095	0.063	0.048	0.038	0.032		

NOTE – For intermediate periods and ductility factors, interpolate linearly.

Figures below the stepped line may be limited by 4.6.2.7(b) when R = 1.0.

* These values shall be used only for the derivation of the design spectra in accordance with 4.6.2.8 and 4.6.2.9.

4.6.2.3 *Structural performance factor*
The structural performance factor, S_p, shall be determined in accordance with 4.2.4.1.

4.6.2.4 *Risk factor for structure*
The risk factor, R, for the structure shall be obtained from table 4.6.2.

4.6.2.5 *Zone factor*
The zone factor, Z, shall be derived from figure 4.6.2 as appropriate for the site location.

4.6.2.6 *Limit state factor*
The limit state factors, L_s and L_u, shall be obtained from table 4.6.3.

4.6.2.7 *Lateral force coefficients for the equivalent static method*

(a) For the serviceability limit state, the lateral force coefficient, C, is given by equation 4.6.1:

$$C = C_h(T_1, 1)\, S_p\, R\, Z\, L_s \quad \text{(Eq. 4.6.1)}$$

(b) For the ultimate limit state, the lateral force coefficient, C, is given by equation 4.6.2, but shall not be taken as less than 0.03:

$$C = C_h(T_1, \mu)\, S_p\, R\, Z\, L_u \quad \text{(Eq. 4.6.2)}$$

4.6.2.8 *Design spectra for the modal response spectrum method*

(a) For the serviceability limit state the design spectrum, C(T), is given by equation 4.6.3:

$$C(T) = C_h(T, 1)\, S_p\, R\, Z\, L_s \quad \text{(Eq. 4.6.3)}$$

(b) For the ultimate limit state the design spectrum, C(T), is given by equation 4.6.4:

$$C(T) = S_m\, C_h(T, 1)\, S_p\, R\, Z\, L_u \quad \text{(Eq. 4.6.4)}$$

where S_m is the maximum of S_{m1} and S_{m2} from (i) and (ii) following.

(i) S_{m1} shall be obtained from table 4.6.4 using period T_1

(ii) S_{m2} is given by equation 4.6.6:

$$S_{m2} = \frac{K_m\, C\, W_t}{V_{base(1)}} \quad \text{(Eq. 4.6.6)}$$

where C is as given in 4.6.2.7(b) for the equivalent static method, including the 0.03 limit

K_m = 0.8 for elastically responding (i.e., $\mu = 1$) structures or for structures where the horizontal regularity provisions of 4.4.1 are met, or

= 1.0 otherwise.

4.6.2.9 *Design spectra for the numerical integration time history method*

(a) For the serviceability limit state, the design spectrum, C(T), is given by equation 4.6.7.

$$C(T) = C_h(T, 1)\, S_p\, R\, Z\, L_s \quad \text{(Eq. 4.6.7)}$$

(b) For the ultimate limit state, the design spectrum, C(T), shall be appropriate for the aspect of design being undertaken as follows:

(i) For determination of minimum strength requirements in accordance with 4.10.5.1
$C(T)=S_{m1}C_h\,(T,1)\;S_p\;RZL_u$ .... (Eq 4.6.8)

(ii) For determination of inelastic effects and capacity actions in accordance with 4.10.5.2.
$C(T)=C_h\,(T,1)\;RZL_u$ .... (Eq 4.6.9)

Table 4.6.2 – Risk factor for structure

Category (Refer table 2.3.1)	Risk factor, R
I	1.3
II	1.2
III	1.1
IV	1.0
V	0.6

Table 4.6.3 – Limit state factor

Limit state	Limit state factor
Serviceability	L_s = 1/6
Ultimate	L_u = 1.0

Table 4.6.4 – Design spectrum scaling factor, S_{m1}

T_1 (seconds)	Structural ductility factor, μ								
	1.0	**1.25**	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**8.0**	**10.0**
≤0.45	1.0	0.86	0.61	0.44	0.34	0.28	0.24	0.18	0.15
0.50	1.0	0.85	0.58	0.41	0.32	0.26	0.22	0.17	0.13
0.60	1.0	0.82	0.54	0.37	0.28	0.23	0.19	0.14	0.11
≥0.70	1.0	0.80	0.50	0.33	0.25	0.20	0.17	–	–

NOTE –

(1) For intermediate periods and ductility factors interpolate linearly.
(2) For site subsoil category (C) S_{m1} need not be taken greater than the value for T_1=0.6s

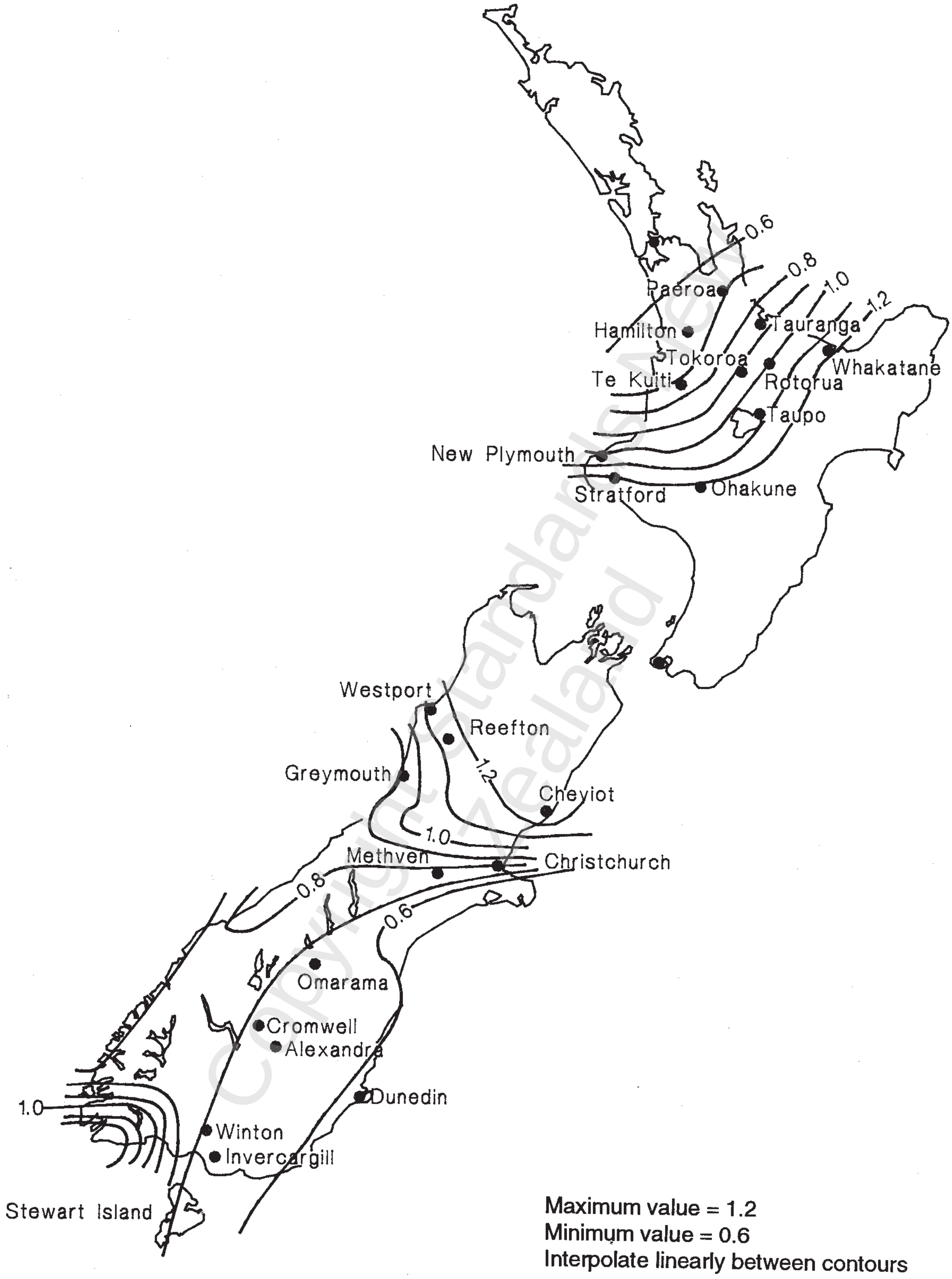


Figure 4.6.2 – Zone factor, Z

4.6.3 *Direction of forces and accidental eccentricity*

4.6.3.1 *Direction of forces*
For buildings with seismic-resisting systems located along two perpendicular directions, the specified forces may be assumed to act separately along each of these two horizontal directions. For other buildings, different directions of application of the specified forces shall be considered so as to produce the most unfavourable effect in any structural element.

4.6.3.2 *Accidental eccentricity*
For each required direction of earthquake loading, allowance shall be made for accidental eccentricity of the applied inertial forces or of the mass, as appropriate to the method of analysis. The accidental eccentricity shall be measured from the nominal centre of mass and shall be determined as follows:

(a) For forces applied in a direction parallel to the principal orthogonal axes of the structure, the eccentricity shall be taken as ±0.1 times the plan dimension of the structure at right angles to the direction of loading.

(b) For forces applied in other directions, the accidental eccentricity may be assumed to lie on the outline of an ellipse with semi-axes equal to the eccentricities specified for the orthogonal directions.

4.7 Seismic deflections and P-delta effects

4.7.1 *General*

4.7.1.1
Deflections in each limit state shall not exceed the appropriate limits specified in 2.5.2.1, 2.5.4.2 and 2.5.4.5.

4.7.1.2
For each limit state, deflections shall be calculated using the same stiffness properties used in the calculation of period and seismic actions.

4.7.1.3
Where the equivalent static method of analysis is used, deflections derived from an elastic analysis for the specified earthquake forces may be reduced in accordance with 4.8.1.5.

4.7.2 *Serviceability limit state deflections*
Calculation of deflections for the serviceability limit state shall take into account any departures from linear elastic behaviour, as required by 2.5.2.2.

4.7.3 *Ultimate limit state lateral deflections*

4.7.3.1
Where the equivalent static method or modal response spectrum method of analysis is used, the design lateral deflection at each level shall be taken as the larger of the values determined from (a) and (b) below:

(a) Deflections found using the equivalent static method or using the modal response spectrum method multiplied by a scale factor equal to the structural ductility factor.

(b) Deflections found by adding the elastic deflection profile determined in accordance with (i) to each possible sidesway mechanism deflection profile determined in accordance with (ii):

(i) The elastic deflection profile, u_{el}, shall be determined in accordance with 4.7.3.1(a) but with a scale factor of unity.

(ii) The sidesway mechanism deflection profiles shall be constructed by considering all potential sidesway mechanisms except those which are specifically suppressed through the application of capacity design procedures. The deflection profile for each sidesway mechanism shall be consistent with obtaining a deflection of $(\mu-1)u_{el}$ at the level of the uppermost principal seismic weight.

4.7.3.2
Where the numerical integration time history method is used, the design lateral deflections shall be taken as the maxima of the appropriate deflections obtained for each of the required ground motions.

4.7.3.3
Where analysis for P-delta effects is required by 4.7.5.1 lateral deflections calculated in accordance with 4.7.3.1 and 4.7.3.2 shall be modified as required by 4.7.5.3.

4.7.4 *Ultimate limit state interstorey deflections*

4.7.4.1
Where the equivalent static method is used, the design interstorey deflection between adjacent levels shall be taken as the maximum value found from the deflection profiles determined in accordance with 4.7.3.1.

4.7.4.2
Where the modal response spectrum method is used, the design interstorey deflection between adjacent levels shall be the greater of that determined in accordance with 4.7.3.1(b) and that determined from the combined modal interstorey deflection multiplied by the structural ductility factor.

4.7.4.3
Where the numerical integration time history method is used, the design interstorey deflection between adjacent levels shall be taken as the maximum of the interstorey deflections obtained for each of the required ground motions.

4.7.4.4
Where analysis for P-delta effects is required by 4.7.5.1 interstorey deflections calculated in accordance with 4.7.4.1, 4.7.4.2 and 4.7.4.3 shall be modified as required by 4.7.5.3.

4.7.5 *P-delta effects – Ultimate limit state*

4.7.5.1
An analysis for P-delta effects shall be carried out unless any one of the following criteria is satisfied;

(a) The fundamental period does not exceed 0.45 seconds;

(b) The height of the structure measured from the base does not exceed 15 m and the fundamental period does not exceed 0.8 seconds;

(c) The structural ductility factor does not exceed 1.5;

(d) The ratio of the design interstorey deflection calculated in accordance with 4.7.4 divided by the storey height for each storey, i, in the appropriate region of the structure defined in (i) or (ii) below, shall not exceed the limit given in equation 4.7.1.

$$\gamma_i \leq \frac{1}{7.5} \times \frac{V_i}{\sum_{j=i}^{n} W_j} \qquad \text{(Eq. 4.7.1)}$$

where $\sum_{j=i}^{n} W_j$ is the sum of the seismic weights above and including level i (i.e., gravity load carried above storey i).

(i) For multi-storey buildings where capacity design is used to specifically exclude column sway mechanisms, equation 4.7.1 shall apply between the base and the mid height of the structure.

(ii) For other buildings, equation 4.7.1 shall apply over the full height of the structure.

4.7.5.2
Where required by 4.7.5.1, a rational method of analysis, which takes into account the ductility demand required in the structure, shall be used to determine the P-delta effects.

4.7.5.3
Unless otherwise included in the analysis method adopted, increases in displacements due to P-delta effects shall be added to the displacements calculated in 4.7.3 and 4.7.4, and the total displacements so derived shall satisfy the limits in 2.5.4.5.

4.8 Equivalent static method

4.8.1 *Equivalent static forces*

4.8.1.1
The set of equivalent static forces specified in this clause shall be assumed to act simultaneously at each level in the direction being considered.

4.8.1.2
The horizontal seismic shear force acting at the base of the structure in the direction being considered shall be calculated from

$$V = C\,W_t \qquad \text{(Eq. 4.8.1)}$$

where the lateral force coefficient, C, is defined in 4.6.2.7(a) for the serviceability limit state and in 4.6.2.7 (b) for the ultimate limit state.

4.8.1.3
The equivalent static lateral force at each level, i, shall be obtained from equation 4.8.2.

$$F_i = 0.92\,V \frac{W_i\,h_i}{\Sigma(W_i\,h_i)} \qquad \text{(Eq. 4.8.2)}$$

4.8.1.4
At the top of the structure an additional horizontal force of 0.08 V shall be added to the value given by equation 4.8.2.

4.8.1.5
The magnitudes of the deflections, including the interstorey deflections, may be reduced by multiplying by the appropriate scale factor given below:

(a) For buildings which have a soft or weak storey 1.0

(b) For other buildings:

(i) With six or more storeys 0.85

(ii) With less than six storeys, by interpolating between 1.0 for a single storey structure and 0.85 for a six storey structure.

4.8.2 *Points of application of equivalent static forces*

4.8.2.1
The equivalent static design forces shall be applied through points eccentric to the nominal centre of mass at each level as specified in 4.6.3.2.

4.9 Modal response spectrum method

4.9.1 *General*

4.9.1.1
The design response spectrum used for the modal response spectrum method shall be as required by 4.6.2.8(a) for the serviceability limit state and 4.6.2.8(b) for the ultimate limit state.

4.9.1.2
Sufficient number of modes shall be included in the analysis to ensure that 90 % or more of the mass is participating in the direction under consideration.

4.9.2 *Torsion*

4.9.2.1 *General*
Where the horizontal regularity provisions of 4.4.1 are satisfied, and a two-dimensional modal response spectrum analysis is used for translational effects, an analysis for torsional effects may be conducted by the static method of 4.9.2.2. In all other cases torsional effects shall be included in a three-dimensional analysis method using the provisions of 4.9.2.3.

4.9.2.2 *Static analysis for torsional effects*
For a static analysis for torsional effects, applied torsion at each level shall use either the forces calculated by the equivalent static method or the combined storey inertial forces found in a two-dimensional modal response spectrum analysis for translation. The eccentricity used shall be as required in 4.6.3.2. Torsional effects shall be combined with the translational effects by direct summation, with signs chosen to produce the most adverse combined effects in the resisting members.

4.9.2.3 *Three-dimensional analysis*

(a) Except as provided in (b) below, for each direction of loading, the position and distribution of the mass shall be adjusted to account for the eccentricity specified in 4.6.3.2. The sign of the eccentricity shall be that producing the largest design actions in the resisting members.

(b) If a rigid floor diaphragm is assumed, the effects of eccentricity for any of the required directions of loading may be allowed for by either of the procedures (i) or (ii) following:

   (i) The general procedure of (a) above may be used, with the centre of mass adjusted, but the designer need not modify the rotational inertia of the floor about the nominal centre of mass to account for the altered distribution of mass; or

   (ii) The mass position and distribution need not be adjusted, but the line of action of the inertial forces shall be taken as eccentric to the nominal centre of mass.

4.9.3 *Combination of modal effects*
The method for combination of modal effects shall take into account the effect of closely spaced modes. Modes shall be considered to be closely spaced if their frequencies are within 15 %.

4.9.4 *Design actions*
Design actions for inclusion in the load combinations specified in Part 2 shall be taken as the combined modal actions, each action having been combined as specified in 4.9.3.

4.10 Numerical integration time history method

4.10.1 *General*

4.10.1.1
Numerical integration time history analyses may be used to:

(a) Determine the strength requirements of a structure, or

(b) Determine the deflection of the structure, or

(c) Ensure that the ductility demands in a structure do not exceed the limits specified in the appropriate material standard, or

(d) Verify that the requirements of capacity design are satisfied, or

(e) Determine the forces generated on parts, or

(f) Any combination of the above.

4.10.1.2
A time history analysis shall be conducted in accordance with sound analytical practice, and all modelling of the structure shall be cautiously appraised. Unless otherwise justified, material and structural properties, including the effects of post-yield behaviour where appropriate, and damping, shall be determined from the appropriate material standards.

4.10.1.3
Analysis of structures by this method shall use at least three different earthquake records of acceleration versus time.

4.10.1.4
The design response spectrum used for the numerical integration time history method shall be as required by 4.6.2.9(a) for the serviceability limit state and 4.6.2.9(b) for the ultimate limit state.

4.10.2 *Scaling of input earthquake records*
The chosen earthquake records shall be scaled by a recognized method. Scaling shall be such that over the period range of interest for the structure being analysed, the 5 % damped spectrum of the earthquake record does not differ significantly from the design spectrum for the limit state being considered.

4.10.3 *Length of input earthquake records for ultimate limit state*
The input earthquake records for the ultimate limit state shall either contain at least 15 seconds of strong ground shaking, or have a strong shaking duration of at least 5 times the fundamental period of the structure, whichever is the greater.

4.10.4 *P-delta effects*
Either the effects of P-delta shall be included directly in the analysis to demonstrate compliance with 2.5.4, or the additional strength to satisfy these requirements may be calculated from the methods of 4.7.5.

4.10.5 *Design using numerical integration time history method*

4.10.5.1
The strength requirements of the yielding members may be taken as the maximum values obtained from elastic time history analyses, using earthquake records scaled in accordance with 4.10.2 to match the spectrum given in 4.6.2.9(b) (i), but shall not be taken less than necessary to satisfy the requirements of the serviceability limit state.

4.10.5.2
Inelastic demands placed on the members and capacity actions shall be obtained from inelastic time history analyses using earthquake records scaled in accordance with 4.10.2 to match the design spectrum given in 4.6.2.9(b)(ii). Inelastic deformation demands shall not exceed the limits given in the appropriate material standard.

4.10.5.3
Deflections shall be determined in accordance with 4.7.3.2 and 4.7.4.3.

4.11 Foundations and soil retaining structures

4.11.1 *Foundations*

4.11.1.1 *General*
In the analysis of building structures, consideration shall be given to the requirements of 2.5.1.4 and 2.5.4.4.

4.11.1.2 *Rocking*
Where dissipation of energy is primarily through rocking of foundations, the structure shall be subject to a special study, provided that this need not apply if the structural ductility factor is equal to or less than 2.0.

4.11.2 *Soil retaining structures*

4.11.2.1
Soil retaining structures which are neither part of a building nor attached to a building shall be designed for earthquake effects if any of the following conditions apply:

(a) The height of the retained ground at the face of the retaining structure exceeds 4.5 m;

(b) The height of the retained ground at the face of the retaining structure exceeds 2.0 m, and the distance, measured at right angles to the face of the retaining structure, to a building other than a Category V building, or to an access to a Category I or II building, is less than the height of the retaining structure at its front face; or

(c) In event of failure of the retaining structure there is a risk to services to Category I or II buildings.

4.11.2.2
The provisions of section 6.6 shall apply to the design of soil retaining structures subject to earthquake loading.

4.11.2.3
For design by the equivalent static method, the seismic coefficient to be applied to the retained soil and to the soil retaining elements shall be taken as 0.25 RZL_s and 0.25 RZL_u for the serviceability and ultimate limit states respectively. The distribution and magnitude of lateral pressure shall be compatible with the deformations at the soil-structure interface, as determined from established soil mechanics theory or from comprehensive tests.

4.11.2.4
Controlled deflection of the soil retaining structure by sliding or tilting of the structure or by structural yield may be used as a mechanism to limit the forces on the soil retaining structure.

4.12 Requirements for parts

4.12.1 *General*

4.12.1.1
All parts of structures, including permanent non-structural components and their connections, and the connections for permanent services equipment supported by structures, shall be designed for the seismic forces specified herein. The value of the risk factor for the parts shall be as provided in table 4.12.1.

Table 4.12.1 – Risk factor for parts

Category (Refer table 2.3.2)	R_p
P.I	1.10
P.II	1.10
P.III	1.00

4.12.1.2
Except as otherwise determined from 4.12.1.3, the seismic force on parts of structures for the serviceability or ultimate limit state as appropriate shall be determined from a design coefficient, C_{pi}, calculated in accordance with 4.12.2.

4.12.1.3
The design forces on a part of a building may be determined from an analysis of the response of the part performed in accordance with established principles of structural design. The response of the part shall be the total response at the level of the part (i.e., with the ground motion added to the structural motion relative to the ground). The analysis shall include modelling of the connections of the part to the structure and allowance for potential overstrength of the structure and of the part. Where vertical ground acceleration effects are included in the analysis, it may be assumed that the ordinates of the corresponding response spectrum are two-thirds those of the spectrum for horizontal earthquake effects. If a modal response spectrum analysis is used, this shall be carried out in accordance with section 4.9. If a numerical integration time history analysis is used, this shall be carried out in accordance with section 4.10. The resulting seismic coefficient may be substituted for the specified values of C_{ph} or C_{pv}.

4.12.1.4
The horizontal seismic force on parts, F_{ph}, shall be determined from:

$$F_{ph} = C_{ph} \; W_p \; R_p \qquad \text{(Eq. 4.12.1)}$$

where C_{ph} shall be taken equal to C_{pi} at the level of the part from 4.12.2.3, and W_p is the weight of the part.

4.12.1.5
Horizontally cantilevered parts, beams supporting columns, and other members where required by the material standards shall be designed for vertical earthquake effects. Unless otherwise determined from 4.12.1.3, the vertical seismic force on parts, F_{pv}, shall be determined from:

$$F_{pv} = C_{pv} \; W_p \; R_p \qquad \text{(Eq. 4.12.2)}$$

where C_{pv} shall be taken as 0.6 RZL_s for the serviceability limit state and 0.6 RZL_u for the ultimate limit state, and W_p is the weight of the part including Q_u tributary to it.

4.12.1.6
Connections for parts shall be designed for seismic forces corresponding to a structural ductility factor for the part, μ_p, equal to 1.0, unless a capacity design is employed to demonstrate that a greater ductility factor is achievable. Where, in the event of failure of connections, there is a risk to persons, design forces on connections shall be multiplied by 1.5 or the connection shall be detailed for displacement ductility factor of not less than 2.0.

4.12.1.7
Deflections of parts under the prescribed seismic forces shall be limited so as not to impair their strength or function, or lead to damage to other building components.

4.12.1.8
Connections between the parts and the building structure shall be designed to accommodate the interstorey deflections determined in accordance with 4.7.4.

4.12.2 *Basic horizontal coefficient for parts*

4.12.2.1 *General*
The basic horizontal coefficient for a part at level i, C_{pi}, shall be determined from the floor coefficient at level i, C_{fi}. C_{fi} is determined from 4.12.2.2, and C_{pi} is determined from it in accordance with 4.12.2.3.

4.12.2.2 *Floor coefficient*
The floor coefficients at and below the base of the structure, C_{fo}, and at the level of the uppermost principal seismic weight, C_{fn}, shall be as given in equations 4.12.3(a) and (b) and 4.12.4 respectively. The floor acceleration coefficient at levels between the base and the level of the uppermost principal seismic weight, C_{fi}, shall be determined by either method (a) or (b) below. For levels other than at floors, linear variation of C_{fi} between adjacent floor levels may be assumed.

C_{fo} $= 0.25\ RZL_s$ for the serviceability limit state .... (Eq. 4.12.3(a))
$= 0.25\ RZL_u$ for the ultimate limit state .... (Eq. 4.12.3(b))

$$C_{fn} = \frac{C_h(T_1,\mu_o)}{C_h(T_1,\mu)} \times \frac{F_n}{W_n} \quad \text{.......... (Eq. 4.12.4)}$$

For the serviceability limit state, $\mu_o=\mu=1$. For the ultimate limit state, μ_o is the structural ductility factor that would apply to the building structure calculated with overstrength, and shall be taken as 1.0 unless capacity design is applied to the building structure to justify a larger value.

(a) Where the equivalent static method is used, the floor coefficient shall be as given by equation 4.12.5.

$$C_{fi} = \frac{C_h(T_1,\mu_o)}{C_h(T_1,1)} \times C_{fo}(1-h_i/h_n) + C_{fn}(h_i/h_n) \quad \text{.......... (Eq 4.12.5)}$$

(b) Where the modal response spectrum method of analysis is used the floor coefficient shall be as given by equation 4.12.6.

$$C_{f\,i} = \frac{C_h(T_1,\mu_o)}{C_h(T_1,\mu)} \times \frac{F_i}{W_i} \quad \text{.......... (Eq. 4.12.6)}$$

4.12.2.3 *Basic horizontal coefficient*
The basic horizontal coefficient for parts at level i, C_{pi}, shall be as given by equation 4.12.7.

$$C_{pi} = C_h(T_{pe},\mu_p).C_{fi} /0.4 \quad \text{(Eq. 4.12.7)}$$

where $C_h(T_{pe},\mu_p)$ is the seismic hazard acceleration coefficient for site subsoil category (b), T_{pe} is the equivalent period of the part given by = 0.2 T_p/T_1 but not to be taken less than 0.45 s.

PART 5 PROVISIONS FOR WIND FORCES

5.1 Notation

5.1.1
For the purposes of this Part of the Standard, symbols shall have the following meanings.

A Surface area of an element or tributary area which transmits wind forces to the element (5.6.3)

A_z Area of a structure or a part of a structure, at height z, upon which the design wind pressure (p_z) operates, being –

(a) Where used in conjunction with the pressure coefficient (C_p), the area upon which the pressure acts, which may not always be normal to the windstream (5.6.1)

(b) Where used in conjunction with a drag force coefficient (C_d), the projected area normal to the windstream (5.8.5), and

(c) Where used in conjunction with a force coefficient, ($C_{F,x}$) or ($C_{F,y}$), the areas as defined in applicable clauses (5.8.2)

a Dimension used in defining the extent of application of local pressure factors (5.6.3)

b Breadth of a structure normal to the windstream.
NOTE – The breadth, b, of a tapered structure should be assessed as the average breadth over the narrowest half of the structure, and for a circular structure as the diameter.

b_s Average breadth of shielding buildings, normal to the windstream (5.4.5)

C_d Drag force coefficient for a structure or member in the direction of the windstream (5.8)

$$= \frac{F_d}{A_z q_z}$$

$$= \frac{F_x}{A_z q_z},$$ the force coefficient for a structure or member, in the direction of the member's x-axis

$$= \frac{F_y}{A_z q_z},$$ the force coefficient for a structure or member, in the direction of the member's y-axis

C_F Frictional drag force coefficient which is resolved into $C_{F,x}$ and $C_{F,y}$ along the member axes (5.6.4, 5.8.2)

C_{pe} External pressure coefficient (5.6.3)

C_{pi} Internal pressure coefficient (5.6.2)

C_{pl} Net pressure coefficient for the leeward half of a free roof (5.7.3); or external pressure coefficient for a bin, silo or tank of unit aspect ratio (5.A4)

C_{pn} Net pressure coefficient for canopies, free standing roofs, walls, etc (5.7)

C_{pw} Net pressure coefficient for the windward half of a free roof (5.7.3)

c Net height of a hoarding, bin, silo or tank (5.7.5)

D	Down-wind roof slope (figure 5.6.1)
d	Depth of a structure parallel to the windstream (5.6.1); or span of a curved roof (5.A2)
d_a	Along-wind depth of a surface, m, =d for walls and flat roofs (5.6.3)
d_t	Depth of the transition zone over which the wind profile becomes established with changes in terrain roughness, m (5.4.4, 5.9.6)
E	Site elevation above mean sea level, m (5.4.6)
F	Wind force normal to the element surface (5.6.1)
F_d	Wind induced drag force parallel to the windstream (5.8.2)
F_f	Wind induced frictional force parallel to the windstream (5.6.4)
F_x	Wind force component resolved along the x-axis of a body (5.8.2)
F_y	Wind force component resolved along the y-axis of a body (5.8.2)
H	Height of the crest of a hill, ridge or escarpment above a valley floor up-wind of the feature (5.4.6)
h	Effective height of a structure (5.6.1)
h_c	Height from ground to the attached canopy, etc. (5.7.2)
h_e	Vertical height from the eaves of a building to the ground immediately below (5.6.3)
h_s	Average height of shielding buildings (5.4.5)
h_t	Vertical height from the apex of a building to the equivalent ground surface immediately below (5.A2)
K_a	Area reduction factor (5.6.3, 5.7)
K_{ar}	Aspect ratio correction factor for individual member forces (5.8.2, 5.B6.1)
K_i	Factor to account for the angle of inclination of the axis of members to the wind direction (5.8.2)
K_l	Local pressure factor (5.6.3, 5.7)
K_p	Reduction factor for porous cladding (5.6.3)
K_{sh}	Shielding factor for multiple open frames (5.8)
k_c	Multiplier for C_{pe} (on circular tanks, bins and silos) (5.A4)
L_u	Horizontal distance up-wind from the crest of a hill, ridge or escarpment to a level half the height below the crest (5.4.6)
l	Length of a frame member; or length of a cantilevered roof beam
l_s	Average spacing of shielding buildings (5.4.5)

M_c Channelling multiplier (5.4.6)

$\bar{M}_c$ Channelling multiplier used with the dynamic wind design procedure (5.9.8)

M_e Site elevation multiplier (5.4.6)

M_h Hill-shape multiplier (5.4.6)

$\bar{M}_h$ Hill-shape multiplier used with the dynamic wind design procedure (5.9.8)

M_{ls} Limit state multiplier (5.4.3, 5.9.5)

M_L Lee multiplier (5.4.6)

$\bar{M}_L$ Lee multiplier used with the dynamic wind design procedure (5.9.8)

M_r Structure risk multiplier (5.4.7)

$\bar{M}_r$ Structure risk multiplier used with the dynamic wind design procedure (5.9.9)

M_s Shielding multiplier (5.4.5, 5.9.7)

M_t Topographic multiplier (the maximum of M_p, M_L and M_c) (5.4.6)

$\bar{M}_t$ Topographic multiplier used with the dynamic wind design procedure (5.9.8)

$M_{(z,cat)}$ Site terrain/height multiplier (5.4.4)

$\bar{M}_{(z,cat)}$ Site terrain/height multiplier used with the dynamic wind design procedure (5.9.6)

n Number of spans of a multi-span roof (5.A1)

n_s Number of up-wind shielding buildings within a 45° sector of radius 20 h_t and with height $h>h_t$ (5.4.5)

p_e External wind pressure, kPa (5.6.3)

p_i Internal wind pressure, kPa (5.6.2)

p_n Net wind pressure, kPa (5.7)

p_z Design wind pressure at height z, kPa (5.6.1)

q_h Design wind pressure at height h, kPa (5.6.3, 5.7)

q_{hc} Design wind pressure at height h_c, kPa (5.7.2)

$q_{(z)}$ Design wind pressure at height z resulting from the design wind speed $V_{(z)}$, kPa (5.5.1)

r Corner radius of a structural shape, m, (5.B2); or the rise of a curved roof, m (5.A2)

S Sidewall (figure 5.6.1)

s Shielding parameter (5.4.5)

U Up-wind roof slope (figure 5.6.1)

V Basic directional wind speed for the region and direction under consideration, m/s (5.4.2, 5.9.4)

$V_{d(z)}$ Design wind speed at height z derived from $V_{(z)}$ to suit the building orientation, m/s (5.3.1)

$\bar{V}_{d(z)}$ Hourly mean design wind speed, m/s (5.9.2)

$V_{(z)}$ Site wind speed at height z for the direction under consideration, m/s (5.4.1)

$\bar{V}_{(z)}$ Hourly mean site wind speed, m/s (5.9.3)

W Windward wall (figure 5.6.1)

w_c Width of canopy, etc, from the face of a building (5.7.2)

x Horizontal distance from a site to the crest of a hill or escarpment, m

z Distance or height above the ground, m (5.4.4, 5.4.6)

α Angle of slope of a roof, ° (5.6.3)

β Angle from the wind direction to a point on the wall of a circular bin silo or tank (5.A4)

δ Actual solidity ratio equal to the ratio of solid wall area to total wall surface area (5.7.5, 5.8)

δ_e Effective solidity ratio for an open frame (5.8.2)

θ Angle between the wind direction and the axis of a building (5.6.3); or angle between the wind direction and the reference axis of a structural section (5.B5)

λ Spacing ratio for open frames (5.8.2)

ν Kinematic viscosity (5.B2)

ϕ Upwind slope of a hill, ridge or escarpment (5.4.6.2, 5.9.8.2)

ϕ_d Average down-wind slope measured from the crest of a hill, ridge or escarpment to the ground level at a distance of 5H (5.4.6.2, 5.9.8.2)

5.2 General requirements

5.2.1 *Determination of wind forces*
Wind forces on a structure or part of a structure shall be determined by one or more of the following:

(a) The applicable clauses of this Part

(b) Reliable references used consistently with the clauses of this Part

(c) Reliable data on wind speed and direction. (The use of uncorrected anemometer data is not permitted)

(d) Wind tunnel or similar tests carried out for a specific structure or reference to such tests on a similar structure together with applicable clauses of this Part.

5.2.2 *Limits of application*

5.2.2.1
Sections 5.3 to 5.8 inclusive of this Part outline a “Static Analysis” procedure which may be used to calculate the wind pressures, forces and moments for all structures less than 15 m height and for all other structures and components which are not wind sensitive. It shall be used to calculate the wind pressures, forces and moments for all cladding elements (including doors and windows) and their supporting frameworks unless wind tunnel or similar test results provide alternative data or design information.

5.2.2.2
Wind sensitive structures and wind sensitive parts of structures shall be designed in accordance with a dynamic analysis procedure such as that set out in section 4 of AS 1170, Part 2 with the supplementary information provided in section 5.9 of this Part.

5.2.2.3
Wind sensitive structures are those which satisfy **both** criteria for their particular category indicated below:

(a) Enclosed buildings

(i) The ratio of height to square root of plan area at three quarters of building height, is greater than 3.3; and

(ii) The fundamental period of the building or component is greater than 3 seconds.

(b) Exposed structures (such as towers, lattice structures or chimneys), and (essentially) two dimensional structures or parts of structures (such as walls, or cantilever roof systems)

(i) The ratio of the long dimension (height or length) of the structure to its width is greater than 5; and

(ii) The fundamental period of the building or component is greater than 1 second.

5.2.2.4
Tornadoes are considered to be special phenomena and are not covered by this Part.

5.3 Design wind speed, $V_{d(z)}$

5.3.1

The design wind speed, $V_{d(z)}$, at height z, and for wind from a given direction shall equal the maximum site wind speed (at the height z) which occurs within a 90˚ arc (i.e, + or – 45˚) centred on the direction under consideration. For directions other than those given in section 5.4, the site wind speed shall be determined by interpolation of the given data.

5.3.2

Structures shall be designed to withstand wind forces derived by considering wind actions from no fewer than four orthogonal directions.

5.4 Site wind speed

5.4.1 *Derivation of the site wind speed, $V_{(z)}$*

5.4.1.1
The site wind speed, $V_{(z)}$ at height, z, shall be determined for each direction from:

$$V_{(z)} = V\, M_{ls}\, M_{(z,cat)}\, M_s\, M_t\, M_r \text{ (m/s)} \quad \text{(Eq 5.4.1)}$$

where

V	is the basic directional wind speed, m/s (see 5.4.2)
M_{ls}	is the limit state multiplier (see 5.4.3)
$M_{(z,cat)}$	is the site terrain/height multiplier (see 5.4.4)
M_s	is the shielding multiplier (see 5.4.5)
M_t	is the topographic multiplier (see 5.4.6)
M_r	is the structure risk multiplier (see 5.4.7)

$V_{(z)}$ shall not be taken as less than 30 m/s for the ultimate limit state.

5.4.1.2
The site wind speed shall be derived by either:

(a) Determining the wind speed for each of the 8 directions given in table 5.4.1 in conjunction with the appropriate wind speed modification multipliers for that direction, as given in equation 5.4.1, or

(b) Determining a non-directional wind speed by using the non-directional basic wind speed (from table 5.4.1) and the maximum value of each multiplier (identified in equation 5.4.1) applicable to the site from any direction.

5.4.2 *Basic directional wind speed, V*

5.4.2.1
The basic directional wind speed, V shall be taken from table 5.4.1 for the region and direction under consideration. The regional boundaries are shown in figure 5.4.1.

Table 5.4.1 – Basic directional wind speed, V, m/s

Wind Direc-tion	Wind region						
	I North-land	II Auck-land	III Central N.I.	IV Sthn N.I.	V Wgtn	VI South Island	VII Far South
Non-direc-tional	49	45	43	46	48	45	48
N	49	44	44	44	47	46	45
NE	49	44	42	44	43	43	41
E	50	45	42	43	39	39	39
SE	46	43	42	44	43	39	38
S	46	43	42	44	46	44	43
SW	49	46	43	44	46	44	46
W	50	46	44	48	46	46	50
NW	47	42	44	48	50	48	48

NOTE – Regional boundaries are shown on figure 5.4.1.

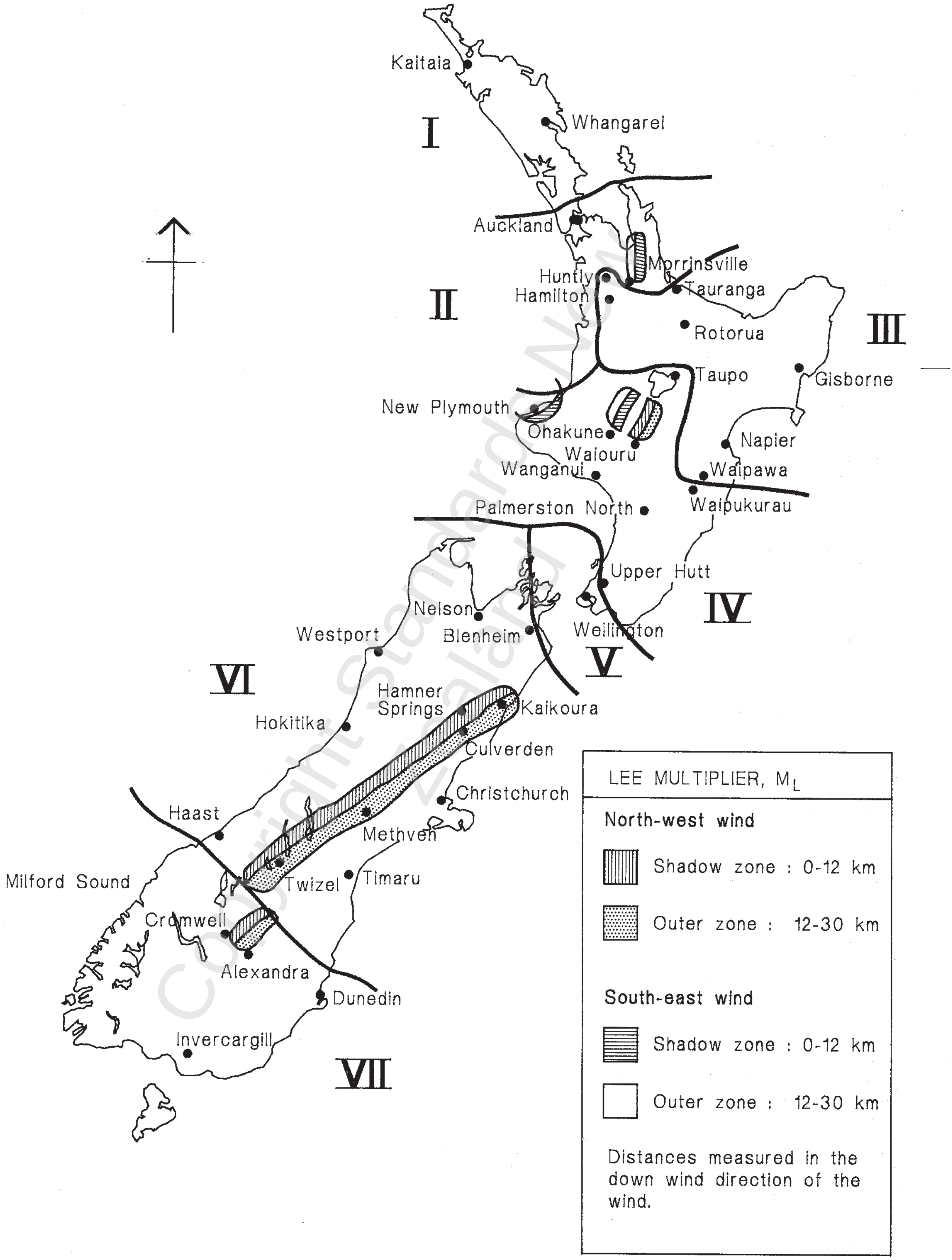


Figure 5.4.1 – Wind regions and lee zones

5.4.3 *Limit state multiplier, M_{ls}*

5.4.3.1
The limit state multiplier, M_{ls}, shall be taken from table 5.4.2 for the limit state and region under consideration.

Table 5.4.2 – Limit state multiplier, M_{ls}

Limit state	Wind region	
	I Northland	II to VII
Serviceability	0.70	0.75
Ultimate	0.93	0.93

5.4.4 *Site terrain/height multiplier, $M_{(z,cat)}$*

5.4.4.1
The variation of wind speed with height, $M_{(z,cat)}$ is defined in table 5.4.3 for heights above ground, z, and for fully developed wind flow over ground roughnesses of category, cat. Where changes in ground roughness occur up-wind of the site in the direction under consideration, and within the depth of the transition zone (refer 5.4.4.4)), the values given in table 5.4.3 are to be determined by interpolation based upon the relative lengths of each roughness present within the transition zone.

5.4.4.2 *Terrain categories*
Terrain, over which the approach wind flows towards a structure shall be assessed on the basis of the following category descriptions:

(a) Category 1 Exposed open terrain with few or no obstructions and water surfaces for the serviceability limit state only

(b) Category 2 Open terrain, grassland with few well scattered obstructions having heights generally from 1.5 m to 10.0 m and water surfaces for the ultimate limit state. (See figure C5.4.2 of Commentary)

(c) Category 3 Terrain with numerous closely spaced obstructions having the size of domestic houses (3.0 m to 5.0 m high). (See figure C5.4.3 of Commentary)

(d) Category 4 Terrain with numerous large, high (10.0 m to 30.0 m high) and closely spaced obstructions such as large city centres and well-developed industrial complexes. (See figure C5.4.4 of Commentary)

5.4.4.3
The terrain roughness applicable at a site, for a given direction, shall be determined from the weighted average of the terrain roughnesses encountered, based on the length of each terrain within the depth of the transition zone for the particular building. Changes in terrain less than 150 m deep (in the direction of wind flow) may be ignored.

5.4.4.4
The depth of the transition zone, d_t, in the direction of the wind flow, shall be taken as 500 m or 50 h_t whichever is the greater.

Table 5.4.3 – Terrain/height multipliers for gust wind speeds in fully developed terrains

Height, z (m)	Multiplier, $M_{(z,cat)}$			
	Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
≤3	0.99	0.85	0.75	0.75
5	1.05	0.91	0.75	0.75
10	1.12	1.00	0.83	0.75
15	1.16	1.05	0.89	0.75
20	1.19	1.08	0.94	0.75
30	1.22	1.12	1.00	0.80
40	1.24	1.16	1.04	0.85
50	1.25	1.18	1.07	0.90
75	1.27	1.22	1.12	0.98
100	1.29	1.24	1.16	1.03
150	1.31	1.27	1.21	1.11
200	1.32	1.29	1.24	1.16

NOTE – For intermediate values of height, z, and terrain category, interpolation is permitted.

5.4.5 *Shielding multiplier, M_s*

5.4.5.1
The shielding multiplier, M_s, shall be taken from table 5.4.4. Where the effects of shielding are ignored, or are not appropriate for a particular wind direction, or where the average up-wind ground gradient is greater than 0.2, M_s shall equal 1.0.

5.4.5.2
The shielding parameter, s, in table 5.4.4 shall be determined from:

$$s = \frac{l_s}{\sqrt{(h_s b_s)}} \quad \text{.......... (Eq. 5.4.2)}$$

where $l_s = h_t \left(\frac{10}{n_s} + 5\right)$ .... (Eq. 5.4.3)

5.4.5.3
For the purposes of 5.4.5 shielding buildings are those within a 45° sector of radius 20 h_t (symmetrically positioned about the directions being considered) and whose height is greater than or equal to h_t.

Table 5.4.4 – Shielding multiplier, M_s

Shielding parameter, s	Shielding multiplier, M_s
≤1.5	0.7
3.0	0.8
6.0	0.9
≥12.0	1.0

NOTE – For intermediate values of s, interpolation is permitted.

5.4.6 *Topographic multiplier, M_t*

5.4.6.1
For sites below an elevation of 500 m, the topographic multiplier, M_t, shall be taken as the largest appropriate value of any of the following:

(a) The hill-shape multiplier, M_h (see 5.4.6.2)

(b) The lee effect multiplier, M_L (see 5.4.6.3)

(c) The channelling multiplier, M_c (see 5.4.6.4)

For sites at or above an elevation of 500 m, the topographic multiplier shall be taken as the product of the applicable values of M_h, M_L and M_c enhanced by an elevation multiplier, M_e, where M_e = 1+0.00015 E, where E is the site elevation above mean sea level (m).

5.4.6.2 *Hill-shape multiplier, M_h*
The multiplier, M_h, shall equal 1.0 except where the site is within a local topographical zone as defined in figures 5.4.2 (a) and (b). In such cases the hill-shape multiplier applicable to the site shall be obtained by interpolating the crest value, established from table 5.4.5, and the value at the boundary, where M_h = 1.0. The interpolation shall be based on the horizontal distance from the crest and on the height above ground, z.

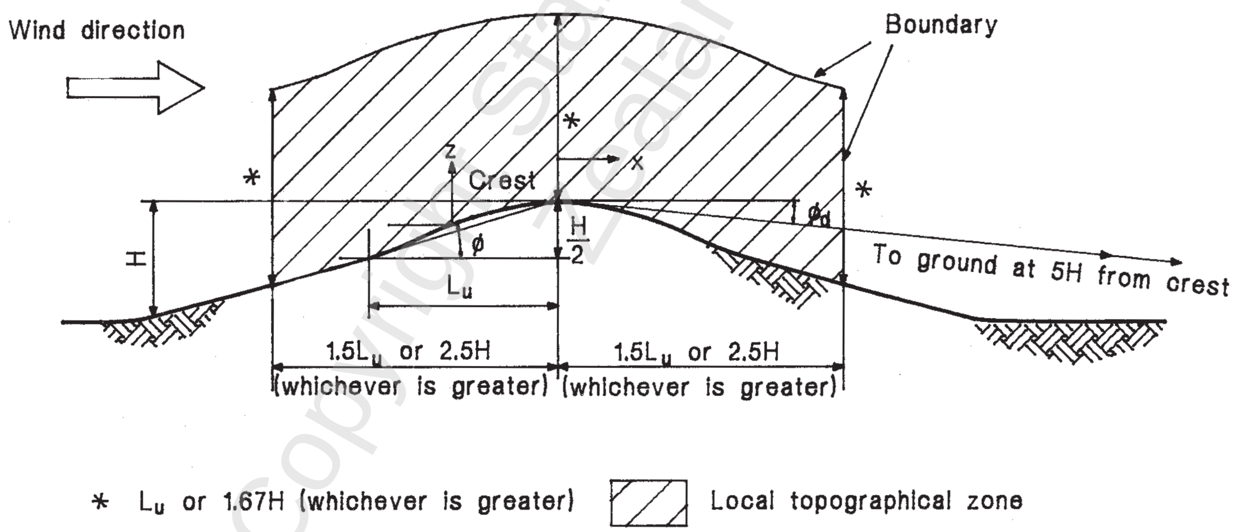


Figure 5.4.2(a) – Definition of the local topographical zone – hills and ridges

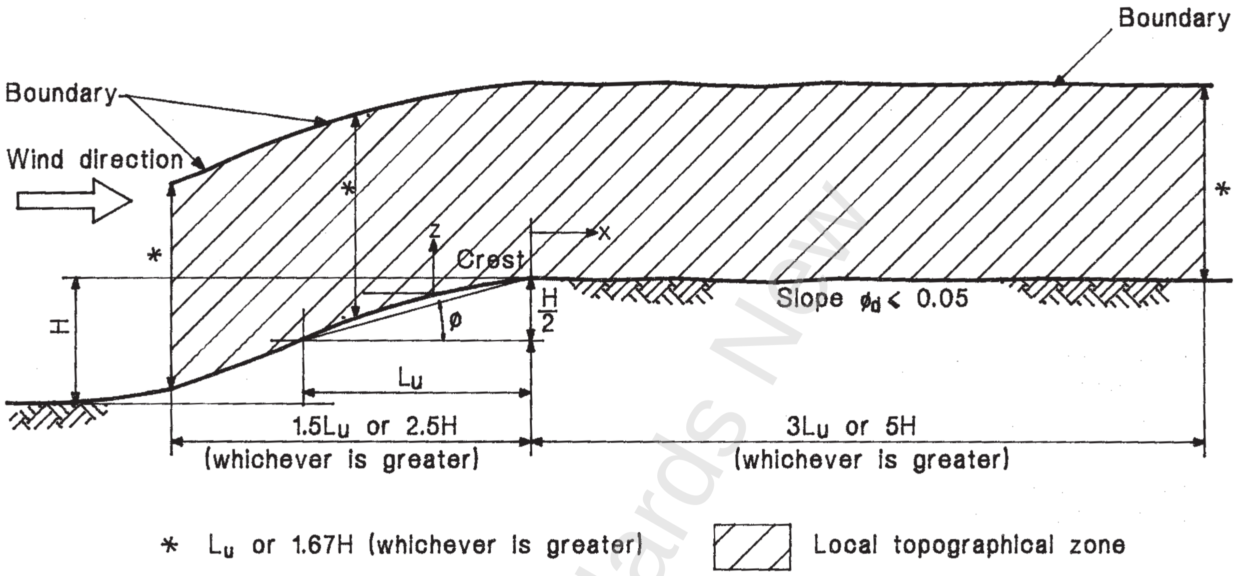


Figure 5.4.2 (b) – Definition of the local topographical zone – two-dimensional escarpments

Table 5.4.5 – Hill shape multiplier, M_h, at crest (z = o)

Up-wind slope, ϕ	Hill shape multiplier, M_h	
	Escarpments $\phi_d \leq 0.05$	Hills and ridges, $\phi_d \geq 0.10$
0.05	1.04	1.09
0.1	1.08	1.18
0.2	1.16	1.36
≥ 0.3	1.24	1.54

NOTES – For intermediate values of either up-wind slope, ϕ, or down-wind slope ϕ_d, linear interpolation is permitted.

5.4.6.3 *Lee multiplier, M_L*

The lee (effect) multiplier, shall equal 1.0 unless the site is within one of the lee zones as indicated in figure 5.4.1. Each lee zone is considered to be 30 km in width, measured from the leeward crest of the initiating range, in the direction of the wind nominated. The lee zone comprises a "shadow lee zone", which extends 12 km from the up-wind boundary of the lee zone, and an "outer lee zone" over the remaining 18 km.

The lee multiplier equals 1.35 for sites within the shadow lee zone, and, within the outer lee zone, the lee multiplier shall be determined by linear interpolation with horizontal distance from the shadow/outer zone boundary (where M_L = 1.35), and the down-wind lee zone boundary (where M_L = 1.0).

The lee multiplier shall apply only to wind from the direction nominated.

5.4.6.4 *Channelling multiplier, M_c*
A channelling multiplier, M_c, of up to 1.2 shall be applied in areas where local acceleration of wind due to channelling effects are known.

A multiplier of not less than 1.1 shall apply in Region V for winds from north, south and south west directions (refer to Commentary).

Elsewhere M_c = 1.0.

5.4.7 *Structure risk multiplier, M_r*

5.4.7.1
The structure risk multiplier, M_r, shall be as given in table 5.4.6.

Table 5.4.6 – Structure risk multiplier, M_r

Classification of building (see 2.3.1)	Multiplier, M_r
Category I	1.1
Categories II, III and IV	1.0
Category V	0.9

5.5 Design wind pressure

5.5.1
The design wind pressure, q_z, at a height, z, shall be calculated from:

$q_{(z)} = 0.6\ V_{d(z)}^2 \times 10^{-3}$ (kPa) .... (Eq.5.5.1)

5.6 Forces and pressure on enclosed buildings

5.6.1 *General*

5.6.1.1 *Forces on building elements*
The forces, F, on enclosed building elements, such as a wall or a roof, shall be calculated from:

$$F = \Sigma p_z A_z \qquad \text{(Eq.5.6.1)}$$

where $p_z = (p_e - p_i)$ at height, z,

NOTE – Where variations in the surface pressure with height are considered, the area may be subdivided so that the specified pressures are taken over appropriate areas.

5.6.1.2 *Forces and moments on complete buildings*
The total resultant forces and overturning moments on a complete building shall be taken to be the summation of the effects of the external pressures on all surfaces of the building.

For rectangular buildings where the ratio d/h or d/b is greater than 4 the total resultant force on a complete structure shall include the frictional drag force, F_f, calculated in accordance with 5.6.4.

5.6.2 *Internal pressures, p_i*

5.6.2.1
The internal wind pressures, p_i, shall be determined using the internal pressure coefficients, C_{pi}, given in table 5.6.1 and the values of q_z corresponding to those adopted for the external surfaces from equation 5.6.3. For multi-storey buildings, the internal pressures shall be calculated separately for each storey of the building:

$$p_i = C_{pi} q_z \qquad \text{(Eq.5.6.2)}$$

Table 5.6.1 – Average internal pressure coefficients, C_{pi}, for buildings of rectangular plan and open interior plan

Condition	Internal pressure coefficient, C_{pi}
1. One wall permeable, other walls impermeable:	
(a) Windward wall permeable	0.6
(b) Windward wall impermeable	–0.3
2. Two or three walls equally permeable, other walls impermeable:	
(a) Windward wall permeable	0.2
(b) Windward wall impermeable	–0.3
3. All walls equally permeable or openings of equal area on all walls	–0.3 or 0.0 whichever is the more severe for combined loadings
4. Dominant openings on one wall:	
(a) Dominant openings on windward wall, giving a ratio of open windward area to total open area (including permeability) of other walls and roofs subject to external suction, equal to–	
0.5 or less	–0.3 or 0.0
1	±0.1
1.5	±0.3
2	±0.5
3	±0.6
6 or more	Value of C_{pe} for the windward wall as in table 5.6.2 (a)
(b) Dominant openings on leeward wall	Value of C_{pe} for leeward external wall surface as in table 5.6.2 (b)
(c) Dominant openings on side wall	Value of C_{pe} for side external wall surface as in table 5.6.2 (c)
(d) Dominant openings in a roof segment	Value of C_{pe} for external surface of roof segment as in table 5.6.3 (a),(b) or (c)
5. A building effectively sealed and having non–opening windows	–0.2 or 0.0, whichever is the more severe for combined loads

5.6.3 *External pressures, p_e*

5.6.3.1
The external wind pressure, p_e, on a surface of an enclosed structure shall be calculated from:

$p_e = C_{pe} K_a K_l K_p q_z$ .... (Eq.5.6.3)

provided that the product $K_l \times C_{pe}$ need not be taken as less than –2.0 when evaluating negative pressure.

5.6.3.2 *External pressure coefficients, C_{pe}, for rectangular enclosed buildings*

The external pressure coefficients, C_{pe}, for walls and roofs of rectangular enclosed buildings are given in tables 5.6.2(a), (b), (c) and 5.6.3(a), (b), (c). The parameters referred to in these tables are set out in figure 5.6.1.

In tables 5.6.2(a), (b), (c) and 5.6.3(a), (b), (c), the height, h, shall be taken as the height from the top of the building to the effective ground level vertically below, h_t, except for $\theta=0^\circ$ and $\alpha<60^\circ$ when h shall be taken to the eaves ($h=h_e$) (see figure 5.6.1).

The external pressure coefficients, C_{pe}, for non-rectangular enclosed buildings are given in Appendix 5.B.

The external pressure coefficient, C_{pe}, on the underside of highset buildings shall be taken as 0.8 and –0.6. For other buildings elevated above the ground, interpolation between these values and 0.0, according to the ratio of clear unwalled height underneath first floor level to the total building height is permitted. For the calculation of underside external pressures, take $q_z = q_h$.

Under-eaves pressures shall be taken as equal to those applied to the adjacent wall face below the surface under consideration.

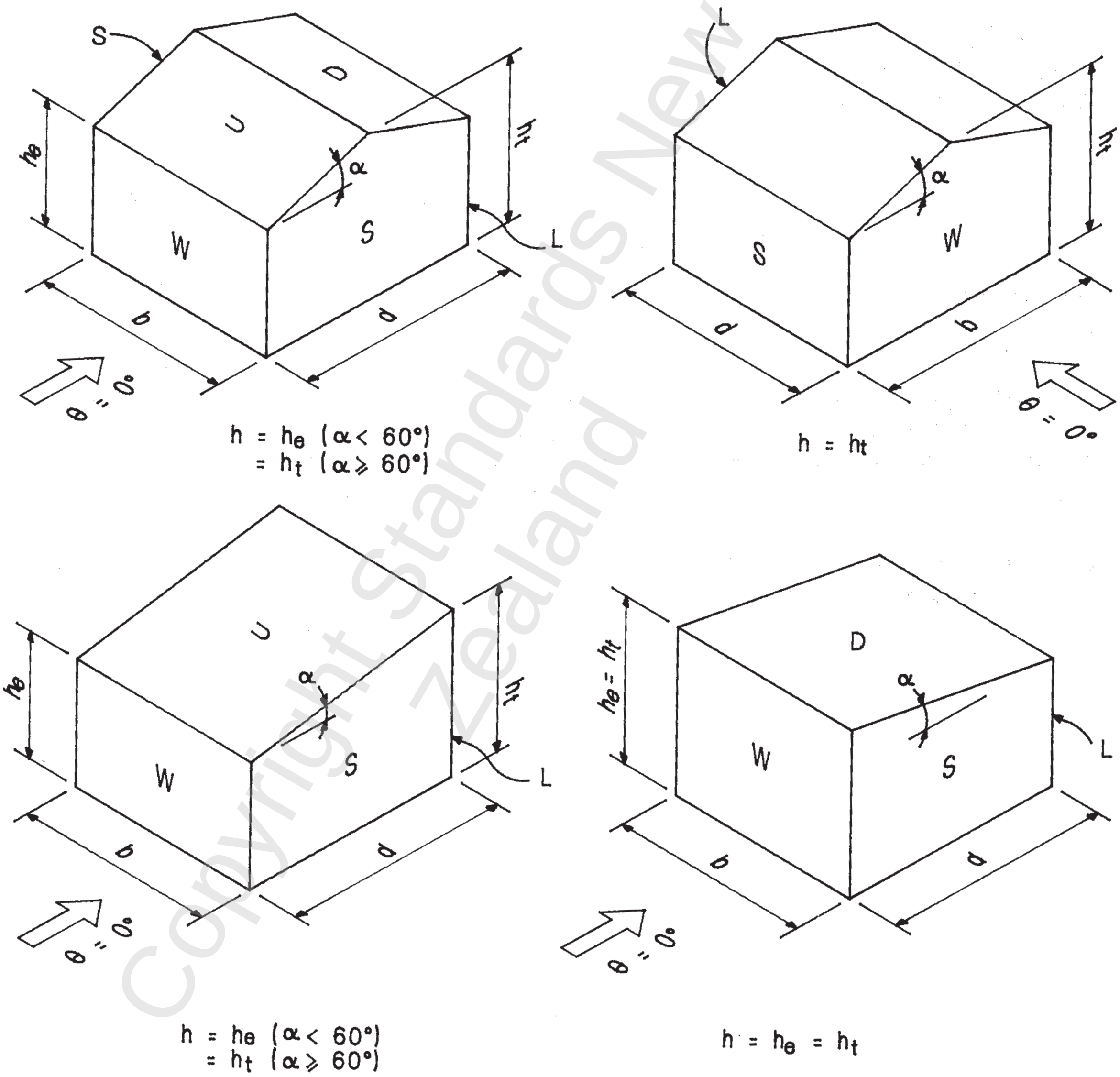


Figure 5.6.1 – Parameters for rectangular enclosed buildings

Table 5.6.2 – Walls: Average external pressure coefficients, C_{pe} for rectangular enclosed buildings

Table 5.6.2 (a) Windward wall, W

Average external pressure coefficients, C_{pe}	
$h \leq 25.0$ m	$h > 25.0$ m
For buildings on ground – 0.8, when q_z varies with height, or 0.7, when used with $q_z = q_h$ For elevated buildings – 0.8 when used with $q_z = q_h$	0.8, when q_z varies with height

Table 5.6.2 (b) Leeward wall, L

Average external pressure coefficients, C_{pe}					
$\theta = 90°$, for all α $\theta = 0°$, with $\alpha < 10°$			$\theta = 0°$		
$d/b \leq 1$	$d/b = 2$	$d/b \geq 4$	$10° \leq \alpha \leq 15°$	$\alpha = 20°$	$\alpha \geq 25°$
–0.5	–0.3	–0.2	–0.3	–0.4	–0.5

NOTE –

(1) For intermediate values of d/b and α, linear interpolation is permitted.

(2) For leeward walls, q_z shall be taken as q_h in all cases.

Table 5.6.2 (c) Side walls, S

Horizontal distance from windward edge	Average external pressure coefficients, C_{pe}
0 to 1 h	–0.65
1 h to 2 h	–0.5
2 h to 3 h	–0.3
> 3 h	–0.2

NOTE – For the sidewalls, q_z shall be taken as q_h throughout.

Table 5.6.3 – Roofs: Average external pressure coefficients, C_{pe}, for rectangular enclosed buildings

Table 5.6.3 (a) Up-wind slope, U, and down-wind slope, D

For: $\theta = 0^\circ$, for $\alpha < 10^\circ$
$\theta = 90^\circ$, for all α

Horizontal distance from windward edge	**Average external pressure coefficient, C_{pe}**	
	h/d ≤ 0.5	h/d ≥ 1.0
0 to 0.5h	–0.9, –0.4	–1.3, –0.6
0.5h to 1h	–0.9, –0.4	–0.7, –0.3
1h to 2h	–0.5, 0	(–0.7)*, (–0.3)*
2h to 3h	–0.3, 0.3	
> 3h	–0.2, 0.3	

* Value is provided for interpolation purposes

For NOTES, see foot of table 5.6.3 (c)

Table 5.6.3 (b) – Up-wind slope, U. For: $\theta = 0^\circ$, all α and h/d ratios

Ratio h/d	**Average external pressure coefficients, C_{pe},**								
	Roof pitch, α degrees								
	<10	10	15	20	25	30	35	45	≥60
≤0.25	See table 5.6.3(a)	–0.7	–0.5	–0.3 0.2	–0.2 0.3	–0.2 0.3	0.4	0.5	0.01α
0.5	–0.9	–0.9	–0.7	–0.4	–0.3 0.2	–0.2 0.2	–0.2 0.3	0.4	0.01α
≥1.0	–1.3	–1.3	–1.0	–0.7	–0.5	–0.3 0.2	–0.2 0.2	0.3	0.01α

For NOTES, see foot of table 5.6.3 (c)

Table 5.6.3 (c) – Down-wind slope, D. For: $\theta = 0^\circ$, all α and h/d ratios

Ratio h/d	Average external pressure coefficients, C_{pe}			
	Roof pitch, α degrees			
	< 10	10	15	≥ 20
≤ 0.25	See table 5.6.3(a)	–0.3	–0.5	–0.6
0.5	–0.5	–0.5	–0.5	–0.6
≥ 1.0	–0.7	–0.7	–0.6	–0.6

NOTES to tables 5.6.3 (a), (b) and (c)

(1) Where two values are listed, the roof shall be designed for both positive pressures over the surface under consideration, and also for negative pressure. The positive or negative pressures shall be considered to act independently from each other and are alternative loading patterns, either of which may develop depending on the turbulence effects present.

(2) To obtain intermediate values for roof slopes and h/d ratios other than those shown, linear interpolation is permitted. Interpolation should only be carried out between values of the same sign.

5.6.3.3 *Area reduction factor, K_a, for roofs and side walls*
For roofs and side walls, the area reduction factor, K_a, shall be as given in table 5.6.4. For all other cases, K_a shall be taken as 1.0.

Table 5.6.4 – Area reduction factor, K_a

Tributary area, A (m^2)	Area reduction factor, K_a
≤ 10	1.0
25	0.9
≥ 100	0.8

NOTE –

(1) For intermediate values of A, linear interpolation is permitted.

(2) K_a = 1.0 for windward and leeward walls and for the side walls of circular bins, silos and tanks.

5.6.3.4 *Local pressure factor, K_l*
The local pressure factor, K_l, shall be taken as 1.0 in all cases except when determining the wind forces applied to claddings, their fixings, the members which directly support the cladding, and the immediate fixings of these members. In these cases K_l shall either be taken from table 5.6.5 for the area and locations indicated or be taken as 1.0, whichever gives the most adverse effect when combined with the external and internal pressure.

The cladding, cladding fixings, the members directly supporting the cladding and their immediate fixings shall be considered to be subjected to wind forces determined by using the appropriate value of K_l over the area indicated in figure 5.6.2, and where the cladding or the supporting member extends beyond that area, a value of K_l = 1.0 shall apply to wind force contributions imposed from beyond that area.

Where interaction is possible, these external pressures, (p_e), shall be assumed to act simultaneously with the internal pressures, p_i, given in 5.6.2 and the under eaves pressures given in 5.6.3.

In all cases the negative limit on the product K_l xC_{pe} shall be –2.0.

Table 5.6.5 – Local pressure factor, K_l

Load case	h_t (m)	Area, A	Proximity to edge	K_l
1. Positive pressures				
Windward walls	all	$A \leq (0.5a)^2$	anywhere	1.25
All other areas	all	....	....	1.0
2. Negative pressures				
(a) Roof edges	all	$A < (0.5a)^2$	<0.5a	2.0
	all	$(0.5a)^2 < A < (1.0a)^2$	<1.0a	1.5
(b) Hips and ridges of roofs with pitch <10°	all	....	....	1.0
(c) Hips and ridges of roofs with pitch ≥ 10°	all	$A \leq (0.5a)^2$	<0.5a	2.0
	all	$(0.5a)^2 < A \leq (1.0a)^2$	<1.0a	1.5
(d) Side walls near windward wall edges	<25	$(0.5a)^2 < A < (1.0a)^2$	<a	1.5
		$A < (0.5a)^2$	<0.5a	2.0
	≥25	$A < (0.5a)^2$	>a	1.5
		$(0.5a)^2 < A < (1.0a)^2$	<a	2.0
		$A < (0.5a)^2$	<0.5a	3.0
(e) All other areas	all	....	....	1.0

NOTE –

(1) The dimension a is defined in figure 5.6.2.

(2) Load cases attracting K_l = 1.5 or 2.0 or 3.0 are alternative cases and need not be applied simultaneously.

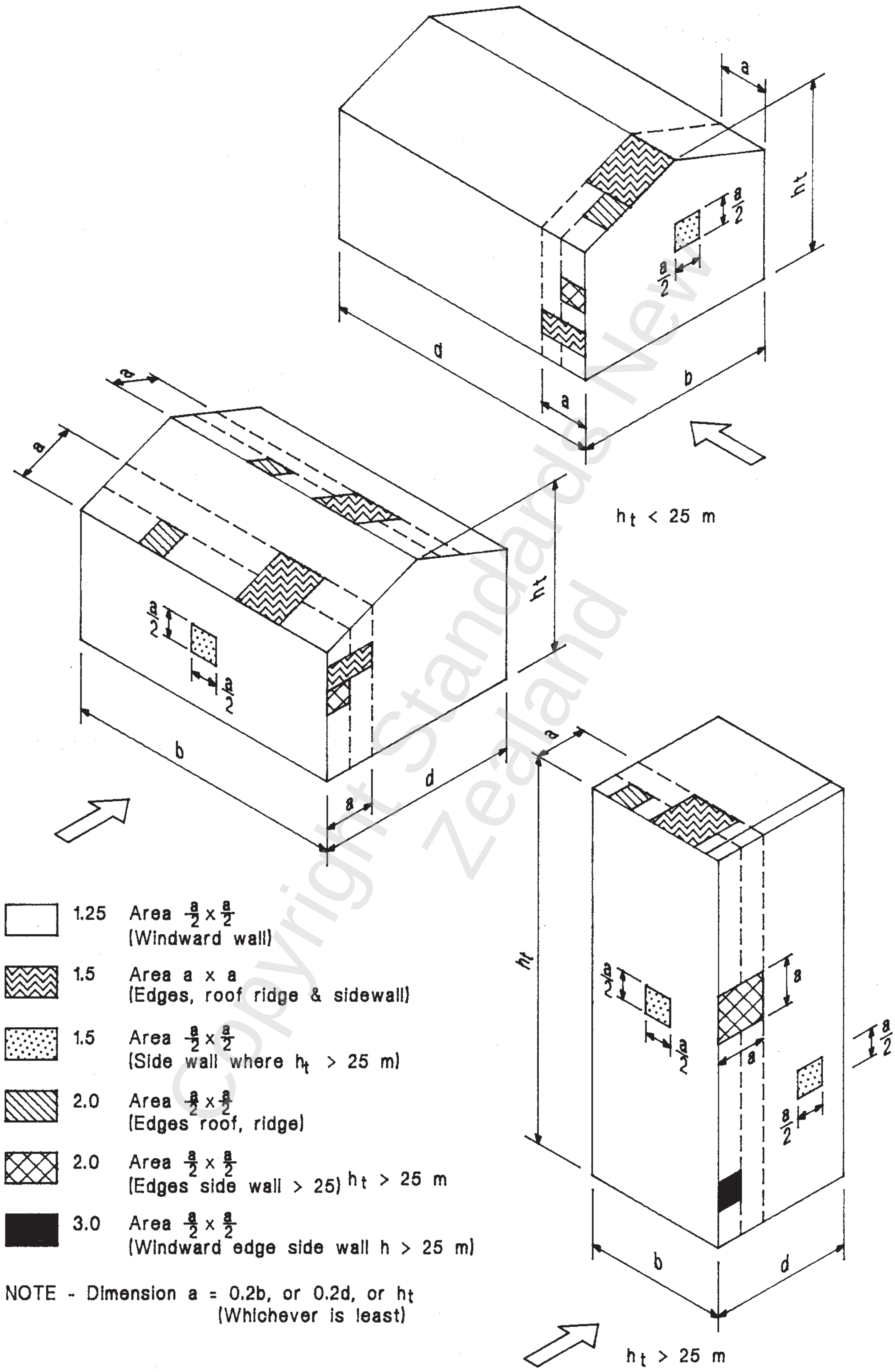


Figure 5.6.2 – Local pressure factors (K_l)

5.6.3.5 *Porous cladding reduction factor, K_p*
Where the ratio of the open area to the total surface area of cladding exceeds 0.001 and is less than 0.01, the porous cladding reduction factor, K_p, shall be as given in table 5.6.6. For all other cases K_p = 1.0.

Table 5.6.6 – Porous cladding reduction factor, K_p

Distance from windward edge	Factor, K_p
0 to $0.2d_a$	0.9
$0.2d_a$ to $0.4d_a$	0.8
$0.4d_a$ to $0.6d_a$	0.7
$0.6d_a$ to $0.8d_a$	0.7
$0.8d_a$ to $1.0d_a$	0.8

5.6.4 *Frictional drag force for rectangular enclosed buildings, F_f*
The frictional drag force, F_f, shall be taken in addition to those loads calculated for rectangular clad buildings only where the ratio d/h or d/b is greater than 4. The frictional drag force in the direction of the wind is given by equations 4.6.4 and 4.6.5.

if $h \le b$, $F_f = C_F q_z b(d - 4h) + C_f q_z 2h(d - 4h)$ .... (Eq.5.6.4)

if $h \ge b$, $F_f = C_F q_z b(d - 4b) + C_f q_z 2h(d - 4b)$ .... (Eq.5.6.5)

where

C_F = 0.01 for smooth surfaces without corrugations or ribs, or with corrugations or ribs parallel to the wind direction

= 0.02 for surfaces with corrugations across the wind direction

= 0.04 for surfaces with ribs across the wind direction.

The first term in each case gives the drag force on the roof and the second the drag force on the walls. The terms are given separately to allow for the use of different values of C_F and q_z on the different surfaces.

5.7 Forces and pressures on canopies, free-standing walls and roof structures

5.7.1 *General*

5.7.1.1 *Forces on elements*
The forces, F, on awnings and canopies attached or adjacent to buildings, and on free-standing walls or roof structures, shall be calculated from:

$F = \Sigma p_n A_z$ .... (Eq. 5.7.1)

NOTE – If the net wind pressure, p_n, varies with height, the area may be subdivided so that the specified pressures are taken over appropriate areas.

5.7.1.2 *Resultant forces and moments*
The total resultant forces and overturning movements on such structures shall be taken to be the summation of the effects of the net pressures on all surfaces of the structure.

In the direction of the windstream, the total resultant force shall include the frictional drag force, F_f, calculated in accordance with 5.7.6.

5.7.2
Net pressures for canopies, awnings and carports, attached to enclosed buildings

5.7.2.1
The net wind pressures, p_n, acting on canopies, awnings or carports attached to an enclosed building, and with a roof slope of 5° or less, shall be calculated from:

$p_n = C_{pn} K_a K_l q_{(z)}$ .... (Eq.5.7.2)

where

C_{pn} is determined according to 5.7.2.2 or 5.7.2.3 as appropriate

K_a shall be taken from table 5.6.4

K_l shall be taken from table 5.7.3

Such structures shall be designed for both downward (positive) and upward (negative) net wind pressures where indicated.

5.7.2.2
For wind directions normal to the attached wall, C_{pn} for canopies and awnings shall be taken from tables 5.7.1(a) or (b) with reference to figure 5.7.1. The reference height (z) at which the design wind pressure is to be determined is also defined in these tables.

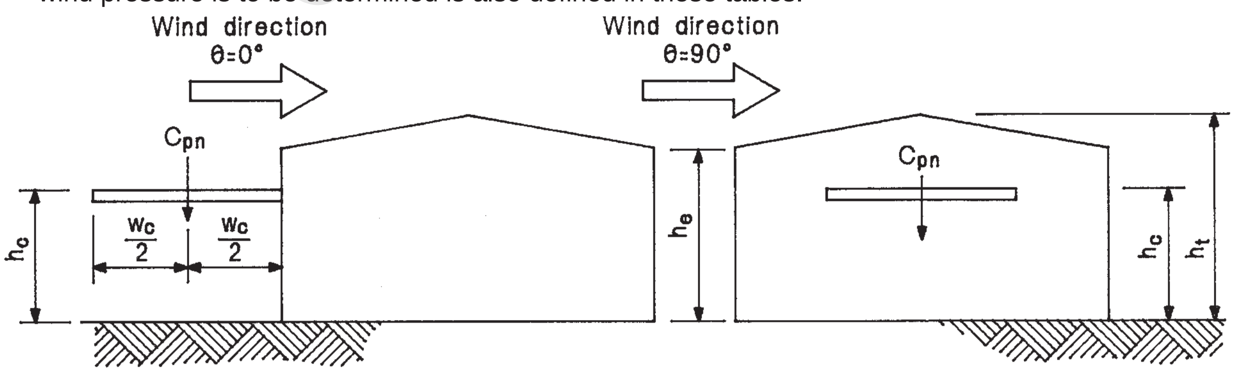


Figure 5.7.1 – Net pressure coefficients, C_{pn}, for canopies and awnings attached to buildings

Table 5.7.1 – Net pressure coefficients, C_{pn}, for canopies and awnings attached to buildings (refer to figure 5.7.1)

Wind normal to wall

Table 5.7.1 (a) For $h_c/h < 0.5$, $\theta = 0^\circ$

Ratio h_c/h	Net pressure coefficients, C_{pn} ($q_z = q_h$)	
	Downwards	Upwards
0.1	1.2	–0.2
0.2	0.7	–0.2
0.5	0.4	–0.2

NOTE – For canopies attached to side walls, h shall be taken as the height h_e, and for canopies attached to gable ends, h_t, as shown in figure 5.7.1 (a).

Table 5.7.1 (b) For $h_c/h \geq 0.5$, $\theta = 0^\circ$

Ratio h_c/h	Net pressure coefficients, C_{pn} ($q_z = q_{hc}$)	
	Downwards	Upwards
0.5	0.5	–0.3
0.75	0.4	[–0.3 – 0.2(h_c/w_c)] or –1.5*
1.0	0.2	[–0.3 – 0.6(h_c/w_c)] or –1.5*

* whichever is the lower magnitude.

NOTE –

(1) q_{hc} is the free stream dynamic wind pressure at height of canopy, awning or carport.

(s) For intermediate values of h_c/h in tables 5.7.1 (a) and (b), linear interpolation is permitted.

5.7.2.3
For wind directions parallel to the wall of the attached building, the canopy or awning shall be considered as a free roof and the net pressure coefficients, C_{pn}, shall be obtained in accordance with the requirements of 5.7.3.

5.7.2.4
Where the ratio h_c/h is less than 0.5, the net pressure coefficient , C_{pn}, shall be used with $q_{(z)} = q_h$, and for h_c/h greater than or equal to 0.5, C_{pn} shall be used with $q_{(z)} = q_{hc}$.

5.7.2.5
For partially enclosed carports, the net pressure coefficient, C_{pn}, shall be determined from table 5.7.2.

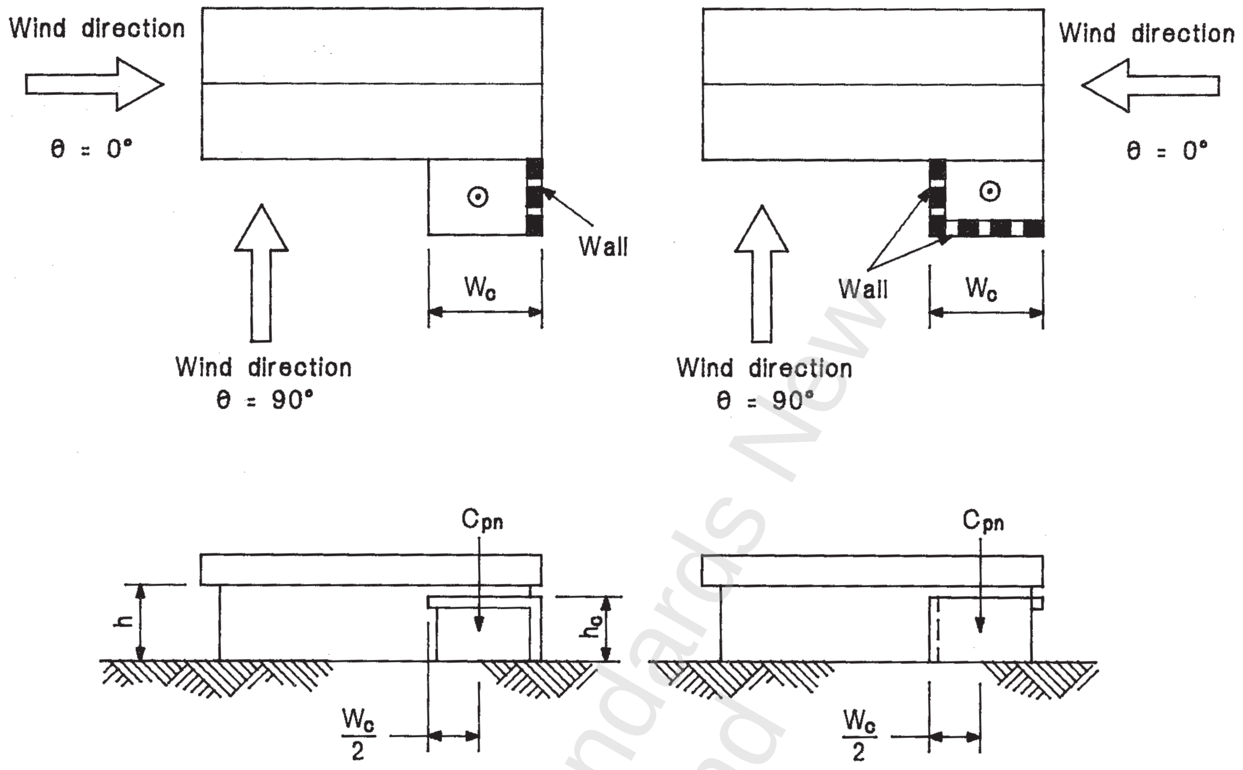


Figure 5.7.2 – Net pressure coefficients, C_{pn}, for partially enclosed carports ($h_c/w_c \le 0.5$) AND ($q_{(z)} = q_{hc}$)

Table 5.7.2 – Net pressure coefficients, C_{pn}, for partially enclosed carports ($h_c/w_c \le 0.5$) AND ($q_z = q_{hc}$). (Refer to figure 5.7.2 above)

Wind direction (θ)	Net pressure coefficients, C_{pn} (upwards) ($q_z = q_{he}$)	Wind direction (θ)	Net pressure coefficients, C_{pn} (upwards) ($q_z = q_{he}$)
0° 90°	–1.0 –0.7	0° 90°	–1.2 –0.6

5.7.2.6
For the design of cladding and its immediate supporting structure, the local pressure coefficients, K_l, given in table 5.7.3 shall be used.

Table 5.7.3 – Local net pressure factors, K_l, for free roofs and canopies

Case	Description	Local net pressure factor, K_l
1	Pressures on an area of 1.0 a^2 or less, within a distance 1.0 a from a roof edge, or a ridge of a roof with a pitch of more than 10°.	1.5
2	Pressures on an area of $(0.5\ a)^2$ or less, within a distance 0.5 a from a roof edge, or a ridge of a roof with a pitch of more than 10°.	2.0

5.7.3 *Net pressures, p_n, on free roofs*

5.7.3.1
A monoslope, pitched or troughed free roof, shall be assumed to be acted on by the resultant pressures acting on each roof half, derived from the pressure coefficients given in tables 5.7.4 (a) and (b) and tables 5.7.5 and 5.7.6. Coefficients for the windward and leeward halves of each roof are shown in figures 5.7.3 to 5.7.5.

5.7.3.2
All values shall be used with the value of q_z applying at height, h. Where two values are listed, the roof shall be designed for both values, taking the combination from the two roof halves giving the worst effect.

5.7.3.3
The net wind pressure (p_n) across the roof surface shall be calculated from:

$$p_n = C_p K_a K_l q_z \quad \text{(Eq.5.7.3)}$$

where

C_p = C_{pw} or C_{pl} depending on the location from the up-wind edge

K_a shall be taken from table 5.6.4

K_l shall be taken from table 5.7.3

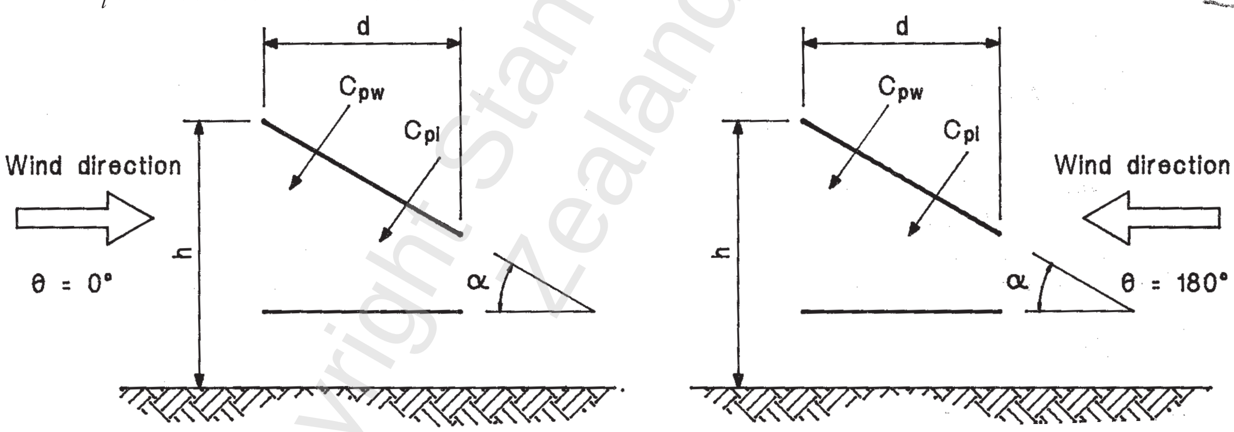


Figure 5.7.3 – Net pressure coefficient for monoslope free roofs

Table 5.7.4 – Net pressure coefficients for monoslope free roofs

Table 5.7.4(a) For 0.25 ≤ h/d ≤ 1

Roof pitch (α) degrees	θ = 0°				θ = 180°			
	C_{pw}		C_{pl}		C_{pw}		C_{pl}	
	Empty under	Blocked under	Empty under	Blocked under	Empty under	Blocked under	Empty under	Blocked under
0	–0.6 0.6	–1.0 0.4	–0.4 0.2	–0.8 0.4	–0.6 0.6	–1.0 0.4	–0.4 0.2	–0.8 0.4
15	–1.0	–1.5	–0.6 0.0	–1.0 0.2	0.8	0.8	0.4	–0.2
30	–2.2	–2.7	–1.1–0.2	–1.3 0.0	1.6	1.6	0.8	0.0

For NOTES, see foot of table 5.7.4(b)

Table 5.7.4(b) For $\alpha = 0^\circ$, $\theta = 0^\circ$ and 90° (h/d < 0.25)

Horizontal distance from windward edge	**Net pressure coefficient, C_{pn}**
0 to 1 h	Values given for C_{pw} in table 5.7.4(a) for $\alpha = 0^\circ$
1 h to 2 h	Values given for C_{pl} in table 5.7.4(a) for $\alpha = 0^\circ$
>2 h	–0.2, 0.2 for empty under –0.4, 0.2 for blocked under

NOTES to tables 5.7.4(a) and (b)

(1) Where two values are listed, the roof shall be designed for both positive pressures over the surface under consideration, and also for negative pressure. The positive or negative pressures shall be considered to act independently from each other and are alternative loading patterns, either of which may develop depending on the turbulence effects present.

(2) To obtain intermediate values for roof slopes other than those shown, linear interpolation is permitted. Interpolation shall be carried out only between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.

(3) For $\theta = 90^\circ$ and all configurations, the roof pitch is effectively zero, and table 5.7.4 (a) with $\theta = 0^\circ$ and $\alpha = 0^\circ$ shall be used.

(4) Net wind pressures are modified when the 'blocked under' condition exists under a canopy or free roof.

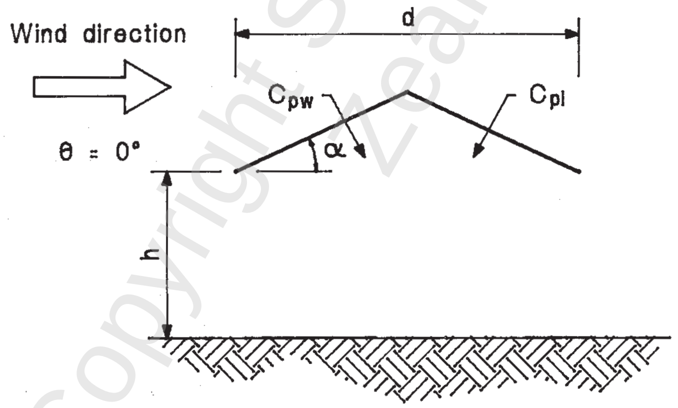


Figure 5.7.4 – Net pressure coefficients for pitched free roofs ($0.25 \leq h/d \leq 1$)

Table 5.7.5 – Net pressure coefficients for pitched free roofs ($0.25 \leq h/d \leq 1$)

Roof pitch, α degrees	$\theta = 0^\circ$				
	C_{pw}			C_{pl}	
	Empty under		Blocked under	Empty under	Blocked under
7.5	–0.6	0.4	–1.4	–0.7	–1.0
15	–0.4	0.6	–1.2	–1.0	–1.3
22.5	–0.4	0.8	–0.9	–1.1	–1.4
30	–0.4	0.9	–0.5	–1.2	–1.5

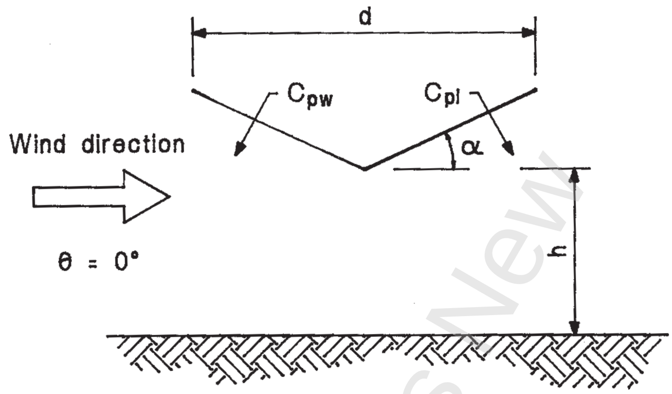


Figure 5.7.5 – Net pressure coefficients for troughed free roofs (0.25 ≤ h/d ≤ 1)

Table 5.7.6 – Net pressure coefficients for troughed free roofs(0.25 ≤ h/d ≤ 1)

Roof pitch, α degrees	**θ = 0°**			
	C_{pw}		**C_{pl}**	
	Empty under	Blocked under	Empty under	Blocked under
7.5	–0.6 0.4	–0.7	0.3	–0.3
15	–0.6 0.4	–0.8	0.5	–0.2
22.5	–0.7 0.3	–1.0	0.7	–0.2

5.7.3.4
In tables 5.7.4(a) and 5.7.6, ‘Empty under’ implies goods and materials stored under the roof, blocking less than 50 % of the cross-section area exposed to the wind. ‘Blocked under’ implies 75 % or more of the cross-section area blocked. For intermediate values of blockage, linear interpolation is permitted.

5.7.3.5
For free roofs of low pitch with fascia panels, the fascia panel shall be treated as the windward wall of an elevated building, and pressures obtained from table 5.6.2 (a).

5.7.3.6
For the design of cladding and its immediate supporting structure, the values of local net pressure factors, K_l, given in table 5.7.3 shall be used.

5.7.4 *Net pressures, p_n, for cantilevered roofs*

5.7.4.1
The design wind loads on cantilevered roofs, such as grandstands, where the cantilever length is greater than 5.0 m, shall be determined by taking into account the dynamic response (refer 5.2.2.3). For cantilevered roofs less than 5.0 m in length, the design wind loads may be obtained as for free roofs (see 5.7.3).

5.7.5 *Net pressures, p_n, on hoardings and free-standing walls*

5.7.5.1
The net wind pressure, p_n, across flat rectangular hoardings or free standing walls shall be calculated from:

$p_n = C_{pn}K_pq_h$ .... (Eq.5.7.4)

where

C_{pn} shall be given by tables 5.7.7 (a), (b) and (c)

$K_p = [1 - (1 - \delta)^2]$

Height, h, in table 5.7.7 shall be as defined in figure 5.7.6.

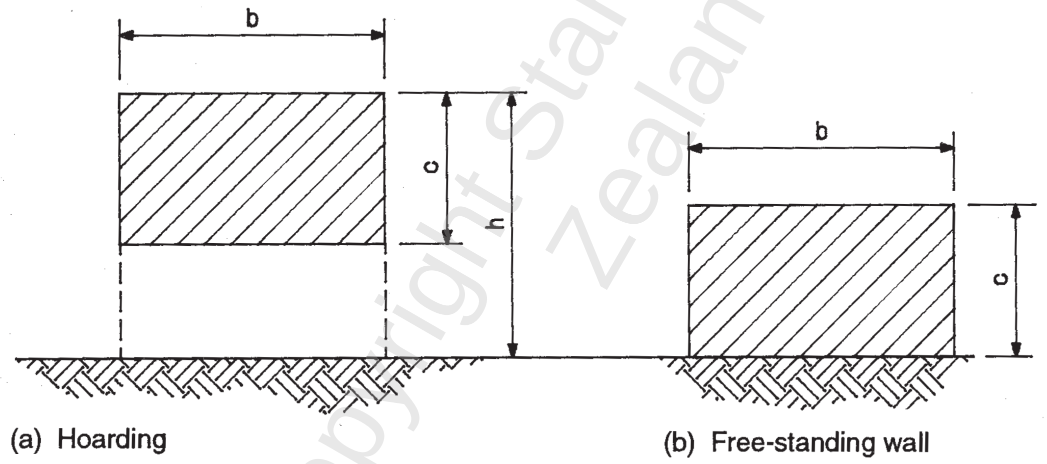


Figure 5.7.6 – Hoardings and free-standing walls

NOTE – Where the ratio (c/h) exceeds 0.7, the hoarding shall be treated as a free-standing wall.

Table 5.7.7 – Net pressure coefficients, C_{pn}, for hoardings (0.5 ≤ b/c ≤ 45) and free-standing walls (0.5 ≤ b/h ≤ 45)

Table 5.7.7(a) Wind normal to hoarding or wall (θ = 0˚)

NET PRESSURE COEFFICIENTS, C_{pn}		
Hoardings		**Free-standing walls**
0.2 ≤ (c/h) ≤ 0.7	0 < (c/h) < 0.2	
1.5	1.2 + 0.02[(b/c) – 5]	1.2

Table 5.7.7 (b) Wind at 45˚ to hoarding or wall (θ = 45˚)

NET PRESSURE COEFFICIENTS, C_{pn}					
Hoardings (5 ≤ b/c < 45)			**Free-standing walls (5 ≤ b/h < 45)**		
Distance from windward free end			Distance from windward free end		
0 to 2 c	2 c to 4 c	> 4 c	0 to 2 h	2 h to 4 h	> 4 h
3.0	1.5	0.75	2.4	1.2	0.6
Hoardings (0.5 ≤ b/c < 5)			**Free-standing walls (0.5 ≤ b/c < 5)**		
Distance from windward free end			Distance from windward free end		
0 to b/2	b/2 to b		0 to b/2	b/2 to b	
1.7	1.1		1.7	1.1	

Table 5.7.7 (c) Wind parallel to hoarding or wall (θ = 90˚)

NET PRESSURE COEFFICIENTS, C_{pn}					
Hoardings			**Free-standing walls**		
Distance from windward free end			Distance from windward free end		
0 to 2 c	2 c to 4 c	> 4 c	0 to 2 h	2 h to 4 h	> 4 h
±1.2	±0.6	±0.3	±1.0	±0.5	±0.25

5.7.6 *Frictional drag forces for free roofs and canopies*

5.7.6.1
For cases given in tables 5.7.4 (a) and (b) and tables 5.7.5 and 5.7.6, frictional drag forces, F_f, acting horizontally in the direction of the windstream shall be calculated using equations 5.7.5, 5.7.6 and 5.7.7.

For $\theta = 0^\circ$: $F_f = 0.01\ bdq_z$ .... (Eq.5.7.5)

where surfaces are smooth without corrugations or ribs, and where wind is parallel to corrugations or ribs; and

$= 0.02\ bdq_z$ .... (Eq.5.7.6)

where wind is across corrugations; and

$= 0.04\ bdq_z$ .... (Eq.5.7.7)

where wind is across ribs

5.8 Forces on exposed structural members

5.8.1 *General*
This section sets out procedures for determining wind forces and moments on structures and components, consisting of exposed members, such as lattice frames, trusses and towers.

5.8.2 *Wind forces on individual structural members*

5.8.2.1 *General*
The wind force on individual exposed structural members, whose aspect ratio, l/b, is greater than 8, shall be calculated either for wind axes from equation 5.8.1:

$$F_d = K_i K_{sh} K_{ar} C_d A_z q_z \qquad \text{(Eq.5.8.1)}$$

or for body axes from equations 5.8.2 and 5.8.3:

$$F_x = K_i K_{sh} K_{ar} C_{F,x} A_z q_z \qquad \text{(Eq.5.8.2)}$$

$$F_y = K_i K_{sh} K_{ar} C_{F,y} A_z q_z \qquad \text{(Eq.5.8.3)}$$

If the wind speed varies over the height of the member, the member shall be subdivided so that the specified forces are taken over appropriate areas.

5.8.2.2 *Angle of inclination factor, K_i*
The angle of inclination factor, K_i, shall be taken as $\sin^2\theta$ where θ is the angle between the wind direction and the reference axis of the structural member.

5.8.2.3 *Shielding factor for multiple open frames, K_{sh}*
The shielding factor for multiple open frames, K_{sh}, shall be taken from tables 5.8.1 (a) and (b).

5.8.2.4 *Aspect ratio correction factor, K_{ar}*
The aspect ratio correction factor, K_{ar}, for individual member forces shall be taken from table 5.8.2.

5.8.2.5 *Drag force coefficient, C_d*
The drag force coefficient, C_d, for use in this clause is given in Appendix 5.B.

5.8.2.6 *Force coefficients, $C_{F,x}$ and $C_{F,y}$*
The force coefficients, $C_{F,x}$ and $C_{F,y}$, for use in this clause are given in Appendix 5.B.

Table 5.8.1 – Shielding factors, K_{sh}, for multiple frames

Table 5.8.1 (a) Wind normal to frames (θ = 0°)

Effective solidity, δ_e	Shielding factors, K_{sh} Frame spacing ratio, λ					
	≤0.2	0.5	1.0	2.0	4.0	≥8.0
0	1.0	1.0	1.0	1.0	1.0	1.0
0.1	0.8	1.0	1.0	1.0	1.0	1.0
0.2	0.5	0.8	0.8	0.9	1.0	1.0
0.3	0.3	0.6	0.7	0.7	0.8	1.0
0.4	0.2	0.4	0.5	0.6	0.7	1.0
0.5	0.2	0.2	0.3	0.4	0.6	1.0
0.7	0.2	0.2	0.2	0.2	0.4	1.0
1.0	0.2	0.2	0.2	0.2	0.2	1.0

Table 5.8.1 (b) Wind at 45° to frames (θ = 45°)

Effective solidity, δ_e	Shielding factors, K_{sh} Frame spacing ratio, λ				
	≤0.5	1.0	2.0	4.0	≥8.0
0	1.0	1.0	1.0	1.0	1.0
0.1	0.9	1.0	1.0	1.0	1.0
0.2	0.8	0.9	1.0	1.0	1.0
0.3	0.7	0.8	1.0	1.0	1.0
0.4	0.6	0.7	1.0	1.0	1.0
0.5	0.5	0.6	0.9	1.0	1.0
0.7	0.3	0.6	0.8	0.9	1.0
1.0	0.3	0.6	0.6	0.8	1.0

where $\lambda = \dfrac{\text{frame spacing centre-to-centre}}{\text{projected frame width normal to wind}}$

NOTE – For intermediate values of δ_e and λ in tables 5.8.1 (a) and (b), interpolation is permitted.

Table 5.8.2 – Aspect ratio correction factors, K_{ar}

Aspect ratio, l/b	Correction factor K_{ar}
8	0.7
14	0.8
30	0.9
40 or more	1.0

NOTE – For intermediate values of l/b, linear interpolation is permitted.

5.8.3 *Wind forces on single open frames*

The wind force on a structure, of open frame type comprising a number of members, lying in a single plane normal to the wind direction, shall be taken as the sum of the wind forces on the individual members from the drag force coefficients for their respective shapes as in 5.8.2. For single open frames the value $K_{sh} = 1.0$.

5.8.4 *Wind forces on multiple open frames*

5.8.4.1

For structures comprising a series of similar open frames in parallel, the force on the second and subsequent frames shall be taken as the force on the windward frame calculated as in 5.8.3, multiplied by a shielding factor (K_{sh}) obtained from tables 5.8.1 (a) and (b).

5.8.4.2

The effective solidity ratio, δ_e, in tables 5.8.1 (a) and (b) shall be taken as the actual solidity ratio, δ = solid area/total area, for flat-sided members.

5.8.4.3

For circular cross-section members, the effective solidity, δ_e, shall be obtained from the actual solidity ratio, δ, using the equation:

$$\delta_e = 1.2\,\delta^{1.75} \quad \text{(Eq.5.8.4)}$$

5.8.5 *Drag forces on lattice towers*

5.8.5.1

For the wind blowing against any face, the design drag force, F_d, on a lattice tower acting parallel to the windstream shall be calculated from:

$$F_d = C_d\,A_z\,q_z \quad \text{(Eq.5.8.5)}$$

5.8.5.2

The overall drag force coefficients, C_d, for lattice towers are given for various arrangements in tables 5.8.3, 5.8.4 and 5.8.5.

Table 5.8.3 – Drag force coefficients, C_d, for square and equilateral-triangle plan lattice towers with flat-sided members

Actual solidity of front face, δ	Drag force coefficients, C_d	
	Square towers	Equilateral-triangle towers
0.1	3.5	3.1
0.2	2.8	2.7
0.3	2.5	2.3
0.4	2.1	2.1
0.5	1.8	1.9

Table 5.8.4 – Drag force coefficients, C_d, for square plan lattice towers with circular members

Actual solidity of front face, δ	Drag force coefficients, C_d			
	Parts of tower in sub-critical flow $bV < 3\ m^2/s$		Parts of tower in super-critical flow $bV \geq 6\ m^2/s$	
	On to face	On to corner	On to face	On to corner
0.05	2.4	2.5	1.4	1.2
0.1	2.2	2.3	1.4	1.3
0.2	1.9	2.1	1.4	1.6
0.3	1.7	1.9	1.4	1.6
0.4	1.6	1.9	1.4	1.6
0.5	1.4	1.9	1.4	1.6

Table 5.8.5 – Drag force coefficients, C_d, for equilateral-triangle plan lattice towers with circular members

Actual solidity of front face, δ	Drag force coefficients, C_d	
	Parts of tower in sub-critical flow $bV < 3\ m^2/s$ (all wind directions)	Parts of tower in super-critical flow $bV \geq 6\ m^2/s$ (all wind directions)
0.05	1.8	1.1
0.1	1.7	1.1
0.2	1.6	1.1
0.3	1.5	1.1
0.4	1.5	1.1
0.5	1.4	1.2

where

δ = the actual solidity ratio (solid area/total enclosed area).

b = the average member diameter.

NOTE –

(1) For intermediate values of bV, interpolation is permitted.

(2) For equilateral-triangle lattice towers with flat-sided members, the force coefficient shall be assumed to be constant for any inclination of the wind to a face.

(3) For square lattice towers with flat-sided members, the maximum force coefficient which occurs when the wind blows on to a corner, shall be taken as 1.2 times the force coefficient for the wind blowing against a face.

5.9 Dynamic wind design procedure

5.9.1 *Application*

5.9.1.1
The dynamic wind design procedure outlined in this section is to be read in conjunction with AS 1170.2 clauses 4.3 and 4.4 of Section 4 "Detailed design procedure : dynamic analysis".

5.9.1.2
The dynamic analysis procedure enables the determination of wind forces and moments on the overall structure only. The static analysis methods outlined in Part 5 of this Standard shall be used to determine the wind forces and actions on individual parts of any building.

5.9.1.3
Dynamic analysis is permitted for all classes of structures and must be used for the analysis of wind sensitive structures and wind sensitive buildings determined in accordance with section 5.2 of this Standard.

5.9.2 *Hourly mean design wind speed* $\bar{V}_{d(z)}$

5.9.2.1
The hourly mean design wind speed $\bar{V}_{d(z)}$, at height z, and for wind from a given direction shall equal the maximum site wind speed (at the height z) which occurs within a 90° arc (i.e., + or – 45°) centred on the direction under consideration. For directions other than those given in section 5.4, the site wind speed shall be determined by interpolation of the given data.

5.9.2.2
Structures shall be designed to withstand wind forces derived by considering wind actions from no fewer than four orthogonal directions.

5.9.3 *Hourly mean site wind speed,* $\bar{V}_{(z)}$

5.9.3.1
The hourly mean site wind speed at height, z, shall be determined from:

$$\bar{V}_{(z)} = V\, M_{ls}\, \bar{M}_{(z,cat)}\, M_s\, \bar{M}_t\, \bar{M}_r \text{ (m/s)} \qquad \text{(Eq. 5.9.1)}$$

where

V is the basic directional wind speed (see 5.9.4)

M_{ls} is the limit state multiplier (see 5.9.5)

$\bar{M}_{(z,cat)}$ is the site terrain/height multiplier (see 5.9.6)

M_s is the shielding multiplier (see 5.9.7)

$\bar{M}_t$ is the topographic multiplier (see 5.9.8)

$\bar{M}_r$ is the structure risk multiplier (see 5.9.9)

5.9.3.2
The hourly mean site wind speed shall be derived by determining the wind speed for each of the 8 directions given in table 5.4.1 in conjunction with the appropriate wind speed modification multipliers for that direction as given in equation 5.9.1.

5.9.4 *Basic directional wind speed, V*

5.9.4.1
The basic directional wind speed, V shall be taken from table 5.4.1 for the region and direction under consideration. The regional boundaries are shown in figure 5.4.1.

5.9.5 *Limit state multiplier, M_{ls}*

5.9.5.1
The limit state multiplier, M_{ls}, shall be as given in table 5.4.2.

5.9.6 *Site terrain/height multiplier, $\bar{M}_{(z,cat)}$*

5.9.6.1
The variation of wind speed with height, $\bar{M}_{(z,cat)}$ is defined in table 5.9.1 for heights above ground, z, and for fully developed wind flow over ground roughnesses of category, cat. Where changes in ground roughness occur up-wind of the site in the direction under consideration, and within the depth of the transition zone (refer 5.9.6.3), the values given in table 5.9.1 are to be determined by interpolation based upon the relative lengths of each roughness present within the transition zone.

5.9.6.2
The terrain roughness shall be determined in accordance with 5.4.4.2.

5.9.6.3
The terrain roughness applicable at a site, for a given direction, shall be determined from the weighted average of the terrain roughnesses encountered, based on the length of each terrain within the depth of the transition zone for the particular building. Changes in terrain less than 150 m deep (in the direction of wind flow) may be ignored.

5.9.6.4
The depth of the transition zone, d_t, in the direction of the wind flow shall be taken as 500 m or 50 h_t whichever is the greater.

Table 5.9.1 – Terrain/height multipliers for hourly mean wind speeds in fully developed terrain

Height (z) m	Multiplier $\bar{M}_{(z, cat)}$			
	Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
≤3	0.61	0.48	0.38	0.35
5	0.65	0.53	0.38	0.35
10	0.71	0.60	0.44	0.35
15	0.74	0.64	0.49	0.35
20	0.77	0.66	0.52	0.35
30	0.80	0.70	0.57	0.38
40	0.83	0.74	0.60	0.40
50	0.85	0.76	0.63	0.42
75	0.89	0.81	0.68	0.51
100	0.92	0.84	0.72	0.55
150	0.97	0.89	0.78	0.62
200	1.00	0.93	0.82	0.67
250	1.03	0.96	0.86	0.71
300	1.06	0.99	0.89	0.74
400	1.10	1.04	0.94	0.81
500	1.14	1.08	0.99	0.86

5.9.7 *Shielding multiplier, M_s*
The shielding multipliers, M_s, shall be taken from table 5.4.4. Where the effects of shielding are ignored or are not appropriate for a particular wind direction, M_s shall equal 1.0 for that direction.

5.9.8 *Topographic multiplier, $\bar{M}_t$*

5.9.8.1
The topographic multiplier, $\bar{M}_t$, shall be taken as the maximum of the hill-shape multiplier, M_h, the lee multiplier, $\bar{M}_L$, and the channelling multiplier, $\bar{M}_c$.

5.9.8.2 *Hill-shape multiplier, $\bar{M}_h$*
The hill-shape multiplier, $\bar{M}_h$, shall equal 1.0 except where the site is within a local topographic zone as defined in figures 5.4.2 (a) and (b). In such cases the profile multiplier applicable to the site shall be obtained by interpolating the crest value, established from table 5.9.2, and the value at the boundary, where $\bar{M}_h = 1.0$. The interpolation shall be based on the horizontal distance from the crest and on the height above ground, z.

Table 5.9.2 – Topographic multiplier at crest (x = 0) for hourly mean wind speeds

Up-wind slope (ϕ) $\phi_d \le 0.05$	Topographical multiplier, $\bar{M}_h$	
	Escarpments $\phi_d \ge 0.10$	Hills and ridges
0.05	1.07	1.16
0.1	1.14	1.32
0.2	1.28	1.64
≥ 0.3	1.42	1.96

5.9.8.3 *Lee multiplier, $\bar{M}_L$*
The lee (effect) multiplier, shall equal 1.0 unless the site is within one of the lee zones as indicated in figure 5.4.1. Each lee zone is considered to be 30 km in width, measured from the leeward crest of the initiating range, in the direction of the wind nominated. The lee zone comprises a "shadow lee zone", which extends 12 km from the up-wind boundary of the lee zone, and an "outer lee zone" over the remaining 18 km.

The lee multiplier equals 1.6 for sites within the shadow lee zone, and, within the outer lee zone, the lee multiplier shall be determined by linear interpolation with horizontal distance from the shadow/outer zone boundary (where $\bar{M}_L$=1.6), and the down-wind lee zone boundary (where $\bar{M}_L$ = 1.0).

The lee multiplier shall apply only to wind from the direction nominated.

5.9.8.4 *Channelling multiplier, $\bar{M}_c$*
A channelling multiplier, $\bar{M}_c$ of up to 1.4 shall be applied to the mean hourly wind speed in areas where local wind acceleration due to channelling effects are known to occur. Elsewhere $\bar{M}_c$ = 1.0.

5.9.9 *Structure risk multiplier, $\bar{M}_r$*

5.9.9.1
The structure risk multiplier $\bar{M}_r$, shall be as given in table 5.4.6.

APPENDIX 5.A
ADDITIONAL PRESSURE COEFFICIENTS

(This Appendix forms an integral part of this Standard.)

5.A1
External pressure coefficients C_{pe}, for multi-span buildings

5.A1.1

External pressures for multi-span buildings, with roofs of pitched or saw-tooth shape (see figures 5.A1.1 and 5.A1.2), shall be calculated in accordance with 5.6.3. External pressure coefficients for multi-span buildings for wind directions $\theta = 0^\circ$ and $\theta = 180^\circ$, shall be obtained from tables 5.A1.1 or 5.A1.2. Where two values are listed, the roof shall be designed for both values.

5.A1.2

The height, h, shall be taken as the height to eaves, h_e, for wind directions of $\theta = 0^\circ$ and $\theta = 180^\circ$. For wind direction $\theta = 90^\circ$, the height, h, shall be taken to the top of the building, h_t.

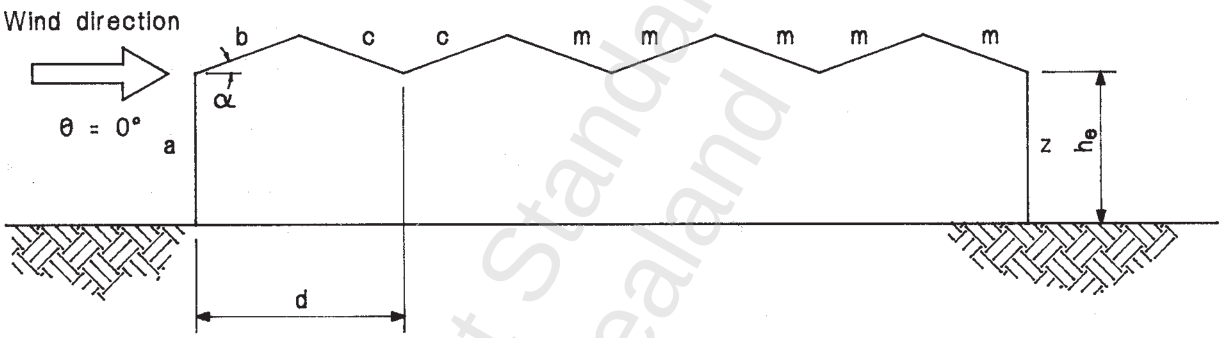


Figure 5.A1.1 – External pressure coefficients, C_{pe}, for multi-span buildings with pitched roofs

Table 5.A1.1 – External pressure coefficients, C_{pe}, for multi-span buildings with pitched roofs

Wind direction (θ) degrees	External pressure coefficients, C_{pe}				
	a	**b**	**c**	**m**	**z**
0	0.7	Use table 5.6.2(a), (b) or (c) for same (h/d) and α as appropriate		–0.3 and 0.2 for $\alpha \le 10^\circ$; –0.5 and 0.3 for $\alpha > 10^\circ$	–0.2

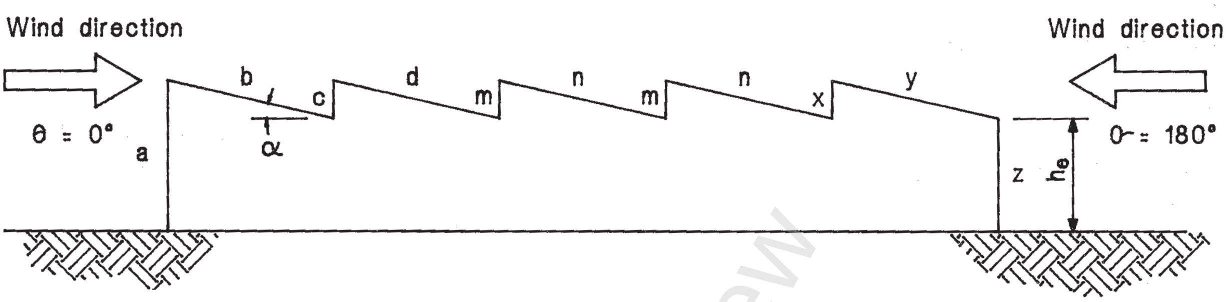


Figure 5.A1.2 – External pressure coefficients, C_{pe}, for multi-span buildings with saw-tooth roofs

Table 5.A1.2 – External pressure coefficients, C_{pe}, for multi-span buildings with saw-tooth roofs

Wind direct-ion (θ) degrees	External pressure coefficients (C_{pe})								
	First span		Second span		Other intermediate spans		End span		
	a	b	c	d	m	n	x	y	z
0	0.7	–0.9	–0.9	–0.5, 0.2	–0.5,0.5	–0.5,0.3	–0.3,0.5	–0.4	–0.2
180	–0.2	–0.2,0.2	–0.3	–0.2, 0.2	–0.4	–0.4	–0.7	–0.3	0.7

5.A1.3
External pressure coefficients for wind directions of θ = 90° and θ = 270° shall be obtained from tables 5.6.2(a), (b) and (c) and 5.6.3(a), (b) and (c) but –0.05(n–1) should be added to the roof pressure coefficients in the region 0 to 1 h from the leading edge, where n is the total number of spans. For this calculation, take n = 4, if n is greater than 4.

NOTE –

(1) The pressure coefficients in tables 5.A1.1 and 5.A1.2 were derived from wind tunnel tests described by Holmes [5.32]. (References at end of commentary).

(2) The increased suction on the end of the building is due to the large frontal dimension presented by the building to the wind, which causes higher wind speeds over the roof.

5.A2
External pressure coefficients, C_{pe}, for curved roofs

5.A2.1
External pressures on curved or arched roofs with profiles approximating a circular arc (see figure 5.A2.1), shall be calculated in accordance with 5.6.3.

5.A2.2
External pressure coefficients for wind directions normal to the axis of the roof shall be obtained from table 5.A2.1. Where two values are listed, the roof shall be designed for both values.

5.A2.3
All pressure coefficients shall be used with the value of q_z applying at height, h_t, where h_t is the height of the highest point on the roof.

5.A2.4
External pressure coefficients, C_{pe}, for wind directions parallel to the axis (ridge) of the roof shall be obtained from table 5.6.3(a).

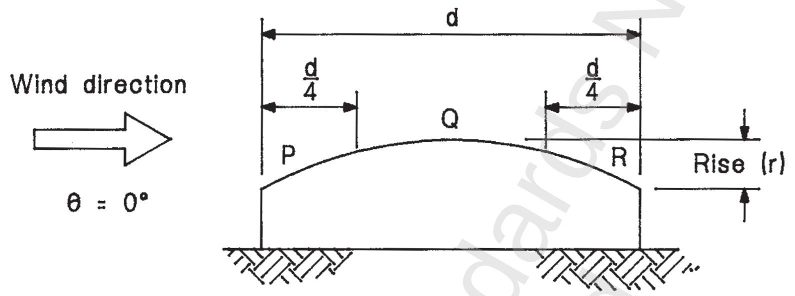


Figure 5.A2.1 – External pressure coefficients, C_{pe} for curved roofs

Table 5.A2.1 – External pressure coefficients, C_{pe} for curved roofs

Exposure	**Rise-to-span ratio r/d**	**External pressure coefficients, C_{pe}**		
		Windward quarter (P)	**Centre half (Q)**	**leeward quarter (R)**
Roof springing from walls	0 < r/d < 0.2	– 0.9	–0.7 – r/d	–0.5
	0.2 ≤ r/d < 0.3 (1.5r/d – 0.3)	(6r/d – 2.1),	–0.7 – r/d	–0.5
	0.3 ≤ r/d ≤ 0.6	(2.75r/d – 0.675)	–0.7 – r/d	–0.5
Roof springing from ground level	0 < r/d < 0.6	1.4r/d	–0.7 – r/d	–0.5

NOTE –

(1) Table 5.A2.1 provides external pressure coefficients for circular arc roofs with no substantial interference to the air flow over the roof. Where a ridge ventilator of a height at least 5 % of the total height of the roof is present, the external pressure coefficient on the central half of the roof may be increased by 0.3, i.e. a suction coefficient is reduced by 0.3, (Holmes, [5.33]). Such reductions should not be made for the wind direction along the axis of the roof, for which the ridge ventilator has little effect on the airflow and resulting pressures.

(2) For roofs which depart substantially from a circular arc in profile, specialist advice should be sought.

5.A3 External pressure coefficients, C_{pe}, for mansard roofs

5.A3.1
The external pressures on a flat-topped mansard roof (see figure 5.A3.1) shall be determined in accordance with 5.6.3.

5.A3.2
The external pressure coefficients, C_{pe}, for the wind direction, $\theta = 0°$, shall be determined as follows:

(a) For up-wind slope (U). Using values for up-wind slope given in table 5.6.3(b).

(b) For flat top (T) and down-wind slope (D). Using values for down-wind slope, with same roof pitch, α, given in table 5.6.3(c).

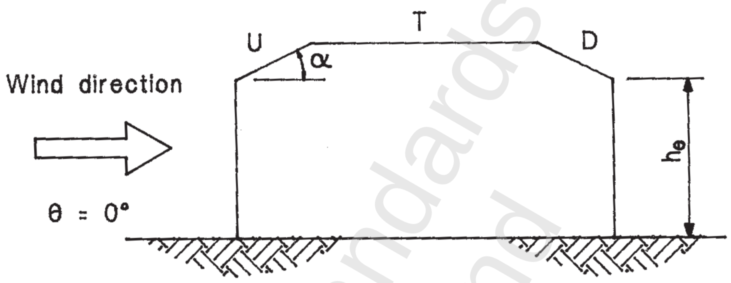


Figure 5.A3.1 – External pressure coefficients, C_{pe}, for mansard roofs

5.A3.3
The external pressure coefficients, C_{pe}, for the wind direction, $\theta = 90°$, shall be determined from table 5.6.3(a).

5.A4 External pressure coefficients, C_{pe}, for circular bins, silos and tanks

5.A4.1
External pressures for circular bins, silos and tanks, shall be calculated in accordance with 5.6.3. External pressure coefficients, $C_{pe}(\beta)$, for the sides of bins, silos and tanks, of circular cross-section, standing on the ground or supported by columns of a height not greater than the height of the cylinder (c) shall be obtained from:

$C_{pe}(\beta) = k_c C_{pl}(\beta)$ .... (Eq. 5.A4.1)

where

$k_c = 1.0$ for $C_{pl} \geq -0.15$
$k_c = 1.0 - 0.55(C_{pl} + 0.15)\log_{10}(c/b)$ for $C_{pl} < -0.15$

$C_{pl}(\beta) = -0.5 + 0.4\cos\beta + 0.8\cos 2\beta + 0.3\cos 3\beta - 0.1\cos 4\beta - 0.05\cos 5\beta$

5.A4.2

The overall drag force, F_d, for the wall section of circular bins, silos and tanks (both elevated and on ground) shall be calculated from:

$F_d = 0.63\ q_h bc$ .... (Eq. 5.A4.2)

External pressures on the underside of elevated bins, silos and tanks shall be calculated as for enclosed rectangular buildings (see 5.6.3.2).

5.A4.3

External pressure coefficients for the roofs or lids of bins, silos or tanks of circular cross-section shall be obtained from figure 5.A4.2.

5.A4.4

The area reduction factor, K_a, from 5.6.3.3 can be applied to the roof or lid of a bin or silo. However K_a should be taken as 1.0 for the wall pressures.

5.A4.5

Local pressure factors, K_l, in table 5.6.4 are applicable to the windward edges of roofs with slope less than or equal to 15°, and to the region near the cone apex for roofs with slope greater than 15°. The applicable areas are shown in figure 5.A4.2. The local pressure factor, K_l, should be taken as 1.0 on the sides of the bin, silo or tank.

NOTE –

(1) The data provided in figures 5.A4.1 and 5.A4.2 are largely based on wind tunnel tests carried out at high Reynolds numbers by Sabransky [5.34] and Macdonald, Kwok and Holmes [5.35].

(2) Figure 5.A4.3 is a graphical presentation of the external pressure coefficient (C_{pl}) for circular bins, silos and tanks of unit aspect ratio at individual locations around the perimeter, and β° from the incident-wind direction as computed from equation 5.A4.1.

(3) The loads specified in 5.A4 are based on data from isolated silos. The grouping of silos may, in some cases, produce loads which are significantly different from those specified. Designers should seek specialist advice if in doubt about such cases. In the absence of more detailed information, grouped silos with spacing between walls greater than two diameters can be treated as isolated silos. A group of closely spaced silos with spacing less than 0.1 diameters, can be treated as a single structure for wind loads, and tables 5.6.2(a), (b), (c) and 5.6.3(a), (b), (c) can be used. Loads for intermediate spacings can be obtained approximately by linear interpolation.

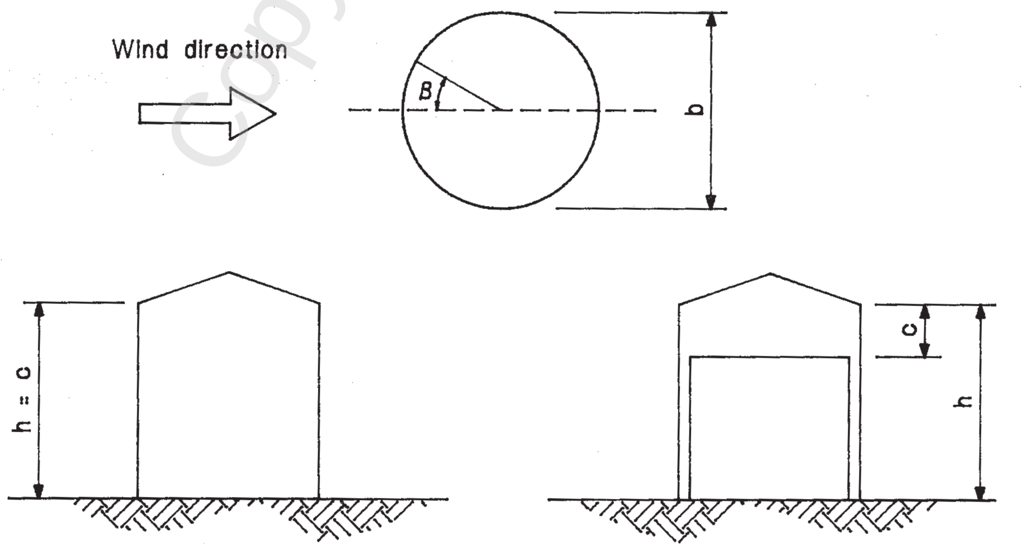
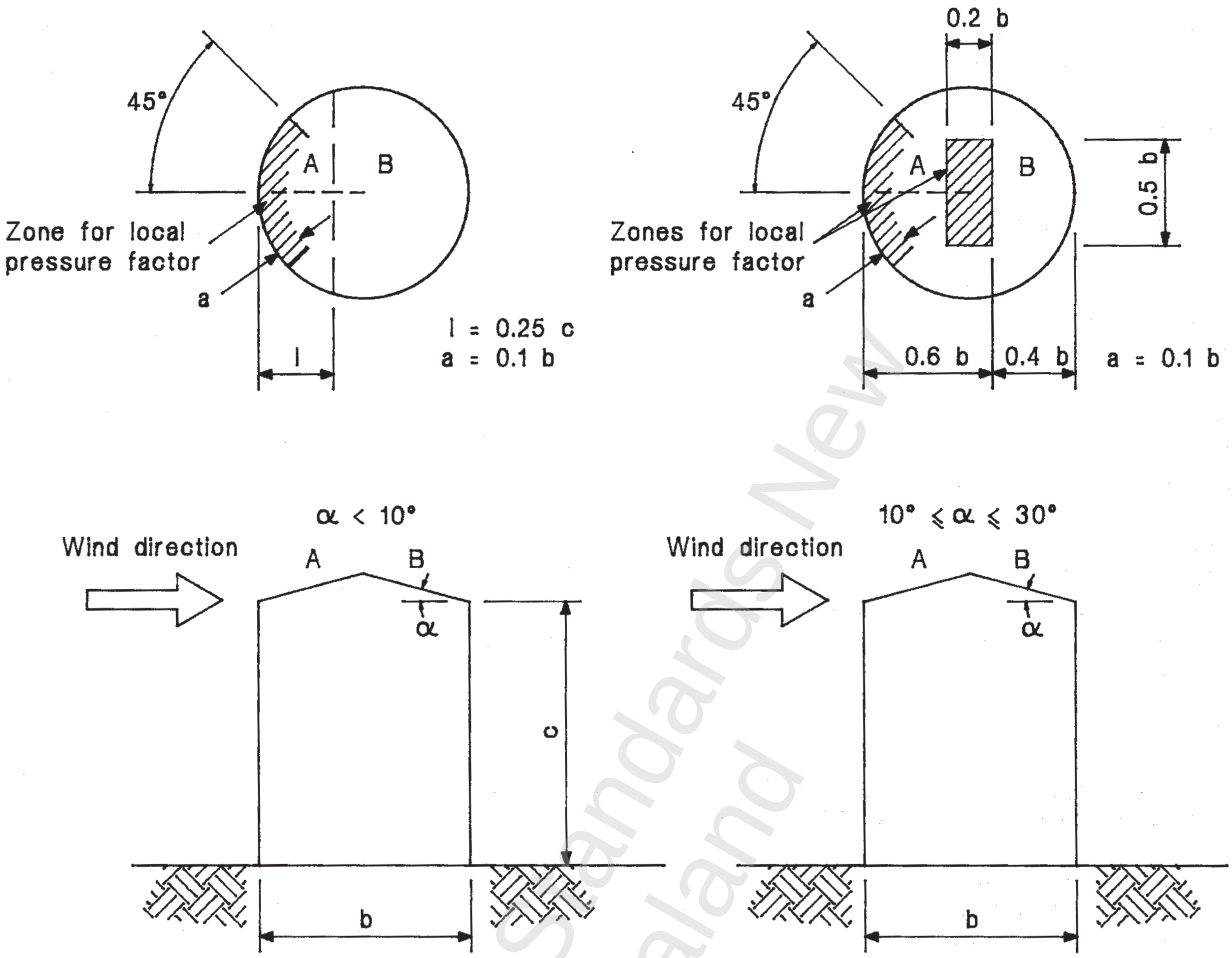


Figure 5.A4.1 – External pressure coefficients, C_{pe}, for walls of circular bins, silos and tanks ($0.25 \leq c/b \leq 4.0$)



External pressure coefficients (C_{pe}) for roofs of circular bins, silos and tanks

Zone A	Zone B
–0.8	–0.5

Figure 5.A4.2 – External pressure coefficients, C_{pe}, for roofs of circular bins, silos and tanks (0.25 < c/b < 4.0)

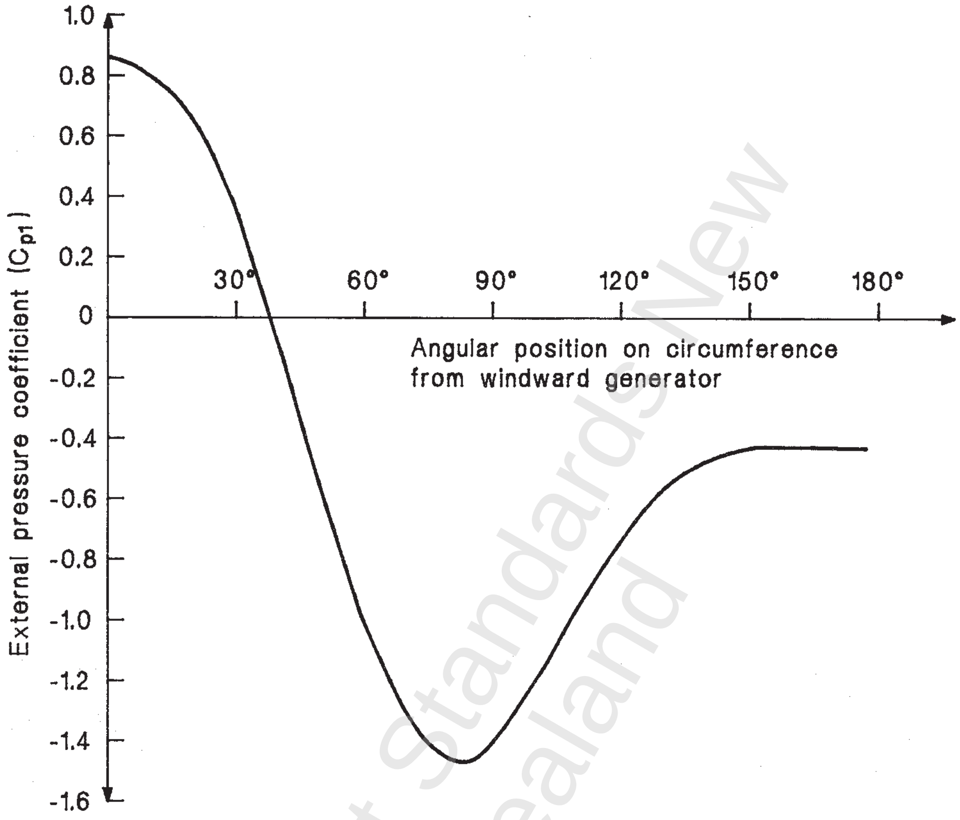


Figure 5.A4.3 – External pressure coefficients, $C_{p/}$, for walls of circular bins, silos and tanks of unit aspect ratio (c/b = 1)

APPENDIX 5.B
SECTIONAL DRAG AND FORCE COEFFICIENTS

(This Appendix forms an integral part of this Standard.)

5.B1 Introduction

5.B1.1
This Appendix specifies drag force and force coefficients, and aspect ratio correction factors for structures or parts of structures with uniform cross-section.

5.B1.2
Structures or parts of structures having aspect ratios, l/b, greater than 8 shall have tabulated drag force or force coefficients multiplied by the correction factor, K_{ar} , given in table 5.B6.1.

5.B1.3
This Appendix should not be used for low aspect ratio structures such as buildings. Pressures on these structures should be derived using 5.6.3.2 or 5.A1, 5.A2 or 5.A3 of Appendix 5.A.

5.B1.4
Forces on structural sections shall be calculated from the equations given in section 5.8, in which the area, A_z, is given by, bl, where b is the width dimension defined in tables 5.B2.1, 5.B3.1 and 5.B5.1, and l is the length of the member.

5.B2 Rounded cylindrical shapes

5.B2.1
Drag force coefficients, C_d, for long sections for which l/b is greater than or equal to 8, where l is the length of the member and b is defined in table 5.B2.1, shall be obtained from table 5.B2.1. For intermediate values of V_zb, linear interpolation is permitted. For values of l/b greater than 8, the aspect ratio correction factors, K_{ar}, shall be obtained from table 5.B6.1.

NOTE –

(1) The values shown in table 5.B2.1 are derived from wind tunnel tests described by Delaney and Sorensen [5.36]. (References at end of commentary.)

(2) Drag coefficients for curved cylindrical shapes depend on the Reynolds Number, Re, as follows:

$$Re = \frac{Vb}{\upsilon}$$

where

V = the wind velocity, m/s

b = the cross-section dimension, m

υ = the kinematic viscosity.

(3) For air at constant temperature and pressure, Re is proportional to Vb. In large scale turbulent flow the 'critical' Reynolds number range broadens and linear interpolation in this range is required.

(4) In general table 5.B2.1 should be used only for structures well away from the influence of the ground.

(5) Treat helically wound cables as smooth cylinders. Cables can also experience small cross-wind (lift) forces. For further information see Sachs [5.37] or specialist test results.

Table 5.B2.1 – Drag force coefficients, C_d, for rounded cylindrical shapes ($l/b \geq 8$)

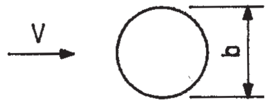
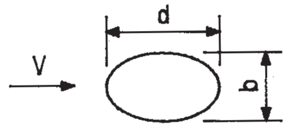
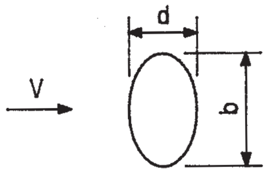
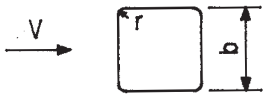
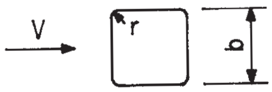
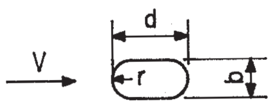
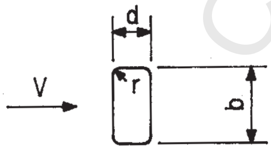
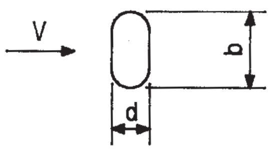
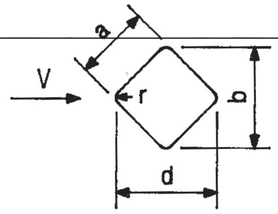
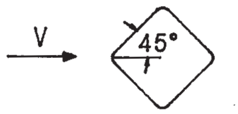
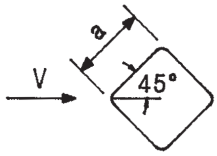
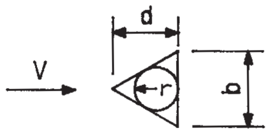
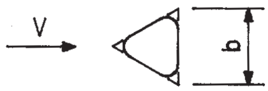
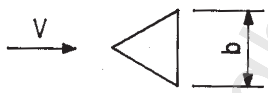
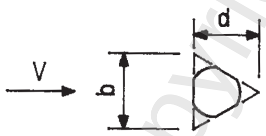
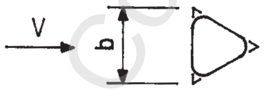
Cross-sectional shape		Drag force coefficients, C_d	
		$V_z b < 4$ m²/s	$V_z b > 10$ m²/s
	Rough or with projections	1.2	1.2
	Smooth (Note 1)	1.2	0.6
	Ellipse b/d = 1/2	0.7	0.3
	Ellipse b/d = 2	1.7	1.5
	b/d = 1 r/b = 1/3	1.2	0.6
	b/d = 1 r/b = 1/6	1.3	0.7
	b/d = 1/2 r/b = 1/2	0.4	0.3
	b/d = 1/2 r/b = 1/6	0.7	0.7
	b/d = 2 r/b = 1/12	1.9	1.9
	b/d = 2 r/b = 1/4	1.6	0.6

Table 5.B2.1 (continued)

Cross-sectional shape		Drag force coefficients, C_d	
		$V_z b < 4\ m^2/s$	$V_z b > 10\ m^2/s$
	r/a = 1/3	1.2	0.5
	r/a = 1/12	1.6	1.6
	r/a = 1/48	1.6	1.6
	r/b = 1/4	1.2	0.5
	r/b = 1/12	1.4	1.4
	r/b = 1/48	1.3	1.3
	r/b = 1/4	1.3	0.5
	1/12 > r/b >1/48	2.1	2.1

NOTE –

(1) Helically wound cables can be considered as smooth.

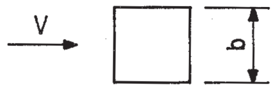
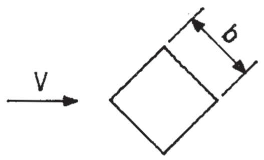
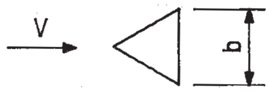
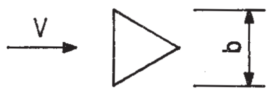
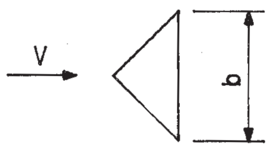
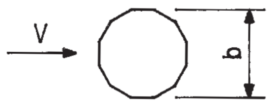
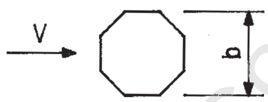
(2) For intermediate values, $4 < V_z b < 10\ m^2/s$, linear interpolation is permitted.

5.B3 Sharp-edged prisms

5.B3.1
Drag force coefficients, C_d, for sharp-edged sections except for rectangular prismatic sections shall be obtained from table 5.B3.1. For values of l/b greater than 8, the aspect ratio correction factors, K_{ar} , shall be obtained from table 5.B6.1.

NOTE – Drag force coefficients for sharp-edged sections are independent of the Reynolds number. Table 5.B3.1 gives values for the most common polygonal cross-sections, except that rectangular prismatic sections (other than square sections) are covered separately in 5.B4.

Table 5.B3.1 – Drag force coefficients, C_d, for sharp-edged prisms (l/b $\geq$ 8)

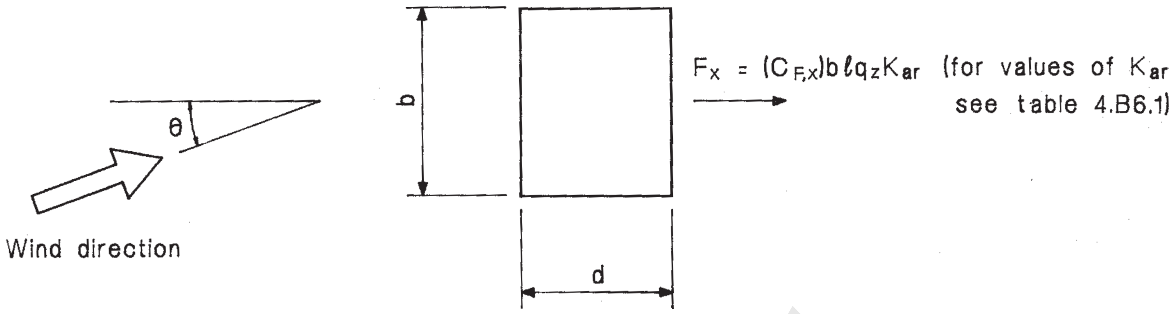
Sectional shape		Drag force coefficients, C_d
	Square with face to wind	2.2
	Square with corner to wind	1.5
	Equilateral triangle – apex to wind	1.2
	Equilateral triangle – face to wind	2.0
	Right angled triangle	1.55
	12-sided polygon	1.3
	Octagon	1.4

5.B4 Rectangular prismatic sections

5.B4.1
Force coefficients, $C_{F,x}$ and $C_{F,y}$, for sections with rectangular cross-sections and aspect ratios l/b greater than or equal to 8 shall be obtained from figures 5.B4.1 and 5.B4.2.

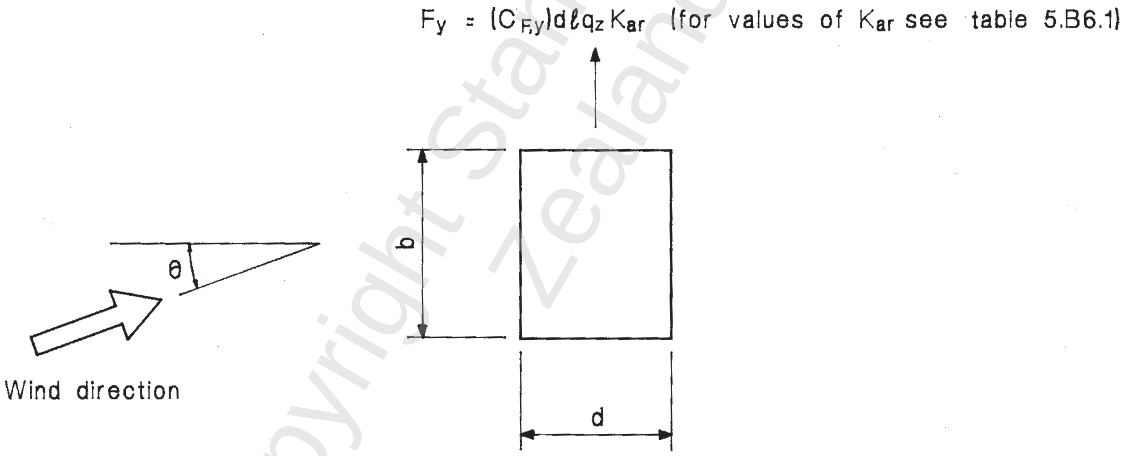
5.B4.2
For structures with d/b ratios greater than 1, inclined to the wind at an angle θ not exceeding 15°, the $C_{F,x}$ coefficients obtained from figure 5.B4.1 shall be increased by the factor [1 + (d/b)tanθ].

5.B4.3
For structures with d/b ratios less than or equal to 1, inclined to the wind at an angle not exceeding 15°, no increase in $C_{F,x}$ coefficients is required.



Aspect ratio, d/b	**Force coefficients, $C_{F,x}$**
0.1	2.2
0.65	3.0
1	2.2
2	1.6
4	1.3
≥ 10	1.1

Figure 5.B4.1 – Along-wind force coefficients for rectangular prisms (l/b > 8)



Aspect ratio, d/b	**Force coefficients, $C_{F,x}$**
0.5	+1.2
1.5	+0.8
2.5	+0.6
4	+0.8
≥ 20	+1.0

Figure 5.B4.2 – Cross-wind force coefficients for rectangular prisms (*l*/b > 8)

NOTE – For intermediate values of d/b, linear interpolation is permitted.

5.B4.4
For values of l/b greater than 8, the aspect ratio correction factors, K_{ar}, shall be obtained from table 5.B6.1.

NOTE –

(1) The data in 5.B4 are derived from data of Jancauskas [5.38]. The large value of $C_{F,y}$ that occurs for sections with d/b ratios about 0.65 was first reported by Nakaguchi, Hashimoto and Muto [5.39].

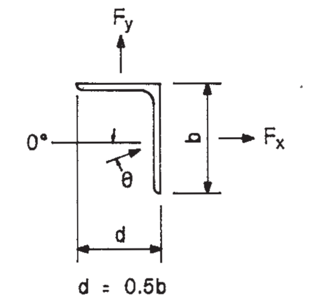
(2) Figure 5.B4.2 contains maximum values of $C_{F,x}$ for angles within 20° of the directions parallel to the faces of the rectangle. This sort of variation can occur in turbulent flow nominally parallel to one face.

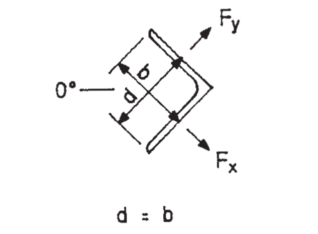
(3) For oblique wind directions greater than 20° from directions parallel to the sides of the rectangle, more detailed information, or specialist advice should be sought.

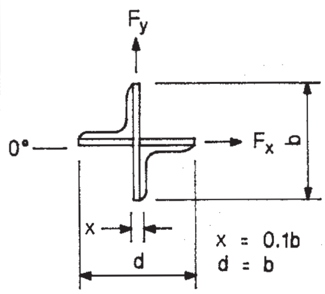
5.B5 Structural sections

5.B5.1
Force coefficients, $C_{F,x}$ and $C_{F,y}$, for structural sections similar to those shown, shall be obtained from table 5.B5.1 for long sections, where l/b is greater than or equal to 8, l is the length of the member and b is defined in this table. For values of l/b greater than 8, the aspect ratio correction factors, K_{ar}, shall be obtained from table 5.B6.1. In table 5.B5.1, the wind direction angle, θ, shall always be measured anticlockwise.

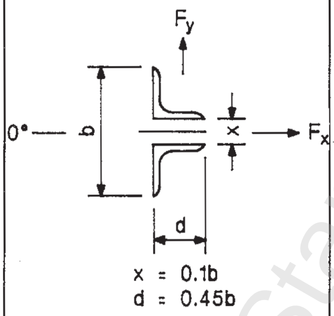
NOTE – These data are unpublished and first appeared in the Swiss Normen A160 [5.40]. Note that the dimension 'b' used in the definition of the force coefficients is not always normal to the flow direction.

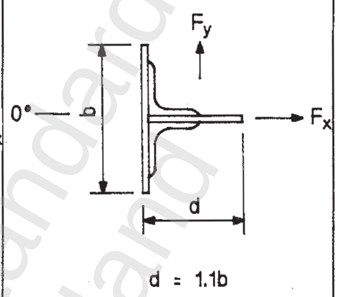


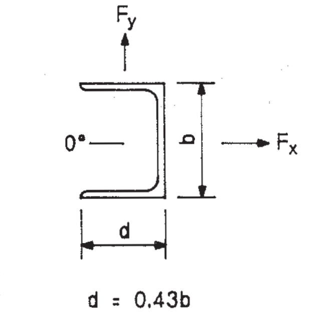




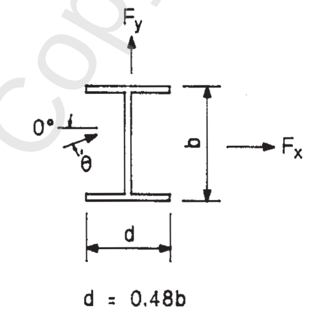
θ	$C_{F,x}$	$C_{F,y}$	$C_{F,x}$	$C_{F,y}$	$C_{F,x}$	$C_{F,y}$
0°	+1.9	+0.95	+1.8	+1.8	+1.75	+0.1
45°	+1.8	+0.8	+2.1	+1.8	+0.85	+0.85
90°	+2.0	+1.7	–1.9	–1.0	+0.1	+1.75
135°	–1.8	–0.1	–2.0	+0.3	–0.75	+0.75
180°	–2.0	+0.1	–1.4	–1.4	–1.75	–0.1

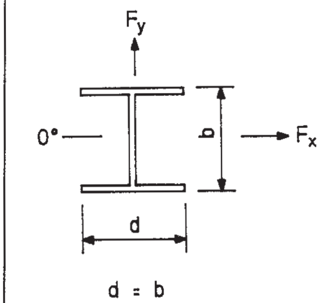


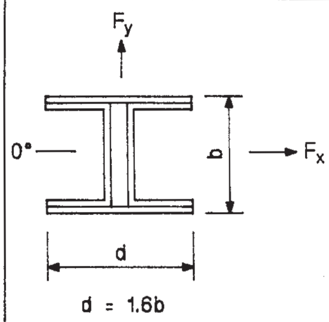




θ	$C_{F,x}$	$C_{F,y}$	$C_{F,x}$	$C_{F,y}$	$C_{F,x}$	$C_{F,y}$
0°	+1.6	0	+2.0	0	+2.05	0
45°	+1.5	–0.1	+1.2	+0.9	+1.85	+0.6
90°	–0.95	+0.7	–1.6	+2.15	0	+0.6
135°	–0.5	+1.05	–1.1	+2.4	–1.6	+0.4
180°	–1.5	0	–1.7	±2.1	–1.8	0







θ	$C_{F,x}$	$C_{F,y}$	$C_{F,x}$	$C_{F,y}$	$C_{F,x}$	$C_{F,y}$
0°	+2.05	0	+1.6	0	+1.4	0
45°	+1.95	+0.6	+1.5	+1.5	+1.2	+1.6
90°	±0.5	+0.9	0	+1.9	0	+2.2

5.B6 Aspect ratio corrections

5.B6.1

The aspect ratio factors, K_{ar}, shall be determined from table 5.B6.1, where the aspect ratio, l/b, of a structure or structural member is greater than 8.

NOTE – When the aspect ratio of a structure or structural member reduces, air flow around the ends of the structure or member, is facilitated. This additional air path reduces the magnitude of the average force on a section.

Table 5.B6.1 – Aspect ratio correction factors, K_{ar}

Aspect ratio, l/b	Correction factor, K_{ar}
8	0.7
14	0.8
30	0.9
40 or more	1.0

NOTE – For intermediate values of l/b, linear interpolation is permitted.

PART 6 PROVISIONS FOR SNOW LOADS, RAIN WATER PONDING LOADS, ICE LOADS, SOIL LOADS AND GROUND WATER LOADS

6.1 Notation

6.1.1

For the purposes of this Part of the Standard, symbols shall have the following meanings:

C_c Coefficient for building category (6.3.1.3)

C_e Coefficient for building exposure (6.3.1.4)

C_r Coefficient for roof slope (6.3.1.5)

M_h Hill shape multiplier (6.5.3 and refer Part 5)

S Basic snow load for sloping roofs, kPa (6.3.1.1)

S_g Open ground snow load, kPa (6.3.1.2)

$V_{d(h)}$ Wind speed at the top of a building for the serviceability limit state (6.3.1.4)

α Effective roof slope, degrees (6.3.1.5)

6.2 General requirements

6.2.1

The loadings, prescribed in this Part, shall be used in conjunction with the load combinations and other general requirements specified in Part 2.

6.3 Snow loads

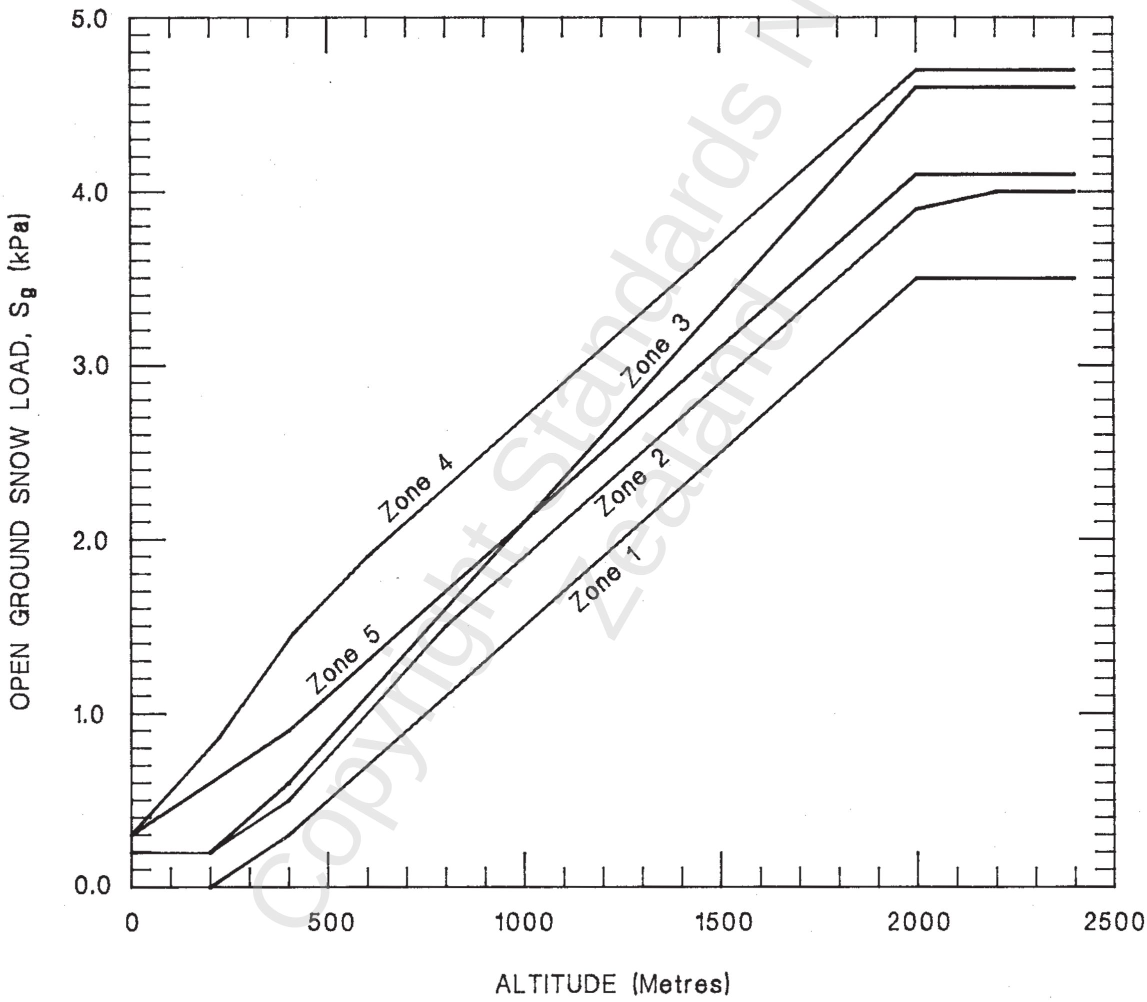
6.3.1 *Basic roof snow load*

6.3.1.1
The basic roof snow load, S (applied over the plan projection of the roof), shall be calculated from:

$$S = C_c \,.\, C_e \,.\, C_r \,.\, S_g \qquad \text{(Eq. 6.3.1)}$$

and shall be distributed in accordance with 6.3.2.

6.3.1.2
The open ground snow load, S_g, shall be as given in figure 6.3.1 for the snow zones shown in figure 6.3.2.



The snow zones shall be as given in figure 6.3.2.

Figure 6.3.1 – Open ground snow load, S_g

Figure 6.3.2 – Snow zones

6.3.1.3
The coefficient for building category, C_c, shall be taken from table 6.3.1.

Table 6.3.1 – Coefficient for building category for snow loads, C_c

Building category (As defined in Part 2)	C_c
I	1.2
II	1.1
III	1.0
IV	1.0
V	0.8

6.3.1.4
The coefficient for building exposure, C_e shall be determined from equation 6.3.2 for each wind direction and that leading to the most adverse snow load used.

$$1.2 \geq C_e = (70 - V_{d(h)}) / 50 \geq 0.8 \qquad \text{(Eq. 6.3.2)}$$

Where $V_{d(h)}$ is the design gust wind speed at the top of the building determined in accordance with Part 5 for the serviceability limit state.

6.3.1.5
Subject to 6.3.1.6 the coefficient for roof slope, C_r, shall be given by:

$$C_r \begin{cases} = 1.0 \text{ for } \alpha < 30^\circ \\ = 0.0 \text{ for } \alpha > 70^\circ \\ = (70 - \alpha)/40 \text{ otherwise} \end{cases}$$

For planar roofs, α shall be taken as equal to the actual roof slope.

For curved roofs, α shall be taken as equal to the average slope from the crown to a line where the actual roof slope exceeds 70˚, or where no such line exists, to the line of the eaves.

6.3.1.6
For regions where shedding of snow is prevented, C_r shall be taken as equal to 1.0.

6.3.2 *Distribution of snow loads on roofs*

6.3.2.1
Each roof shall be considered subdivided into regions bounded on each side by one of a ridge, crown, hip, valley, parapet or eaves. For each region so defined, consideration shall be given to uniform loading in accordance with 6.3.2.2 and to non-uniform loading in accordance with 6.3.2.3. Where S for any region is greater than 1.0 kPa, the additional requirements of 6.3.2.4 shall apply.

6.3.2.2
For uniform loading conditions (a) and (b) apply. For curved roofs, any part of the roof with actual slope greater than 70˚ may be considered free of snow.

(a) Each region defined in 6.3.2.1 shall be considered loaded by the basic roof snow load, S, determined from *a* for the region.

(b) Each region defined in 6.3.2.1 which has a leeward slope shall be considered loaded by S/C_e, where S is determined from *a* for the region. Other regions shall be considered free of snow.

6.3.2.3
For any region from which effective shedding of snow is prevented, a total snow load shall be calculated from the plan area of the region and S. This total load shall be considered non-uniformly distributed such that the intensity of the load along the highest level of the region shall be taken as zero, and the intensity elsewhere shall be taken as proportional to the depth below the highest level.

6.3.2.4
For any roof for which S exceeds 1.0 kPa on any region, effects of drifting or sliding onto lower roofs shall be considered.

6.4 Rain water ponding loads

6.4.1
The load due to rain water ponding shall be calculated from the quantity of water that can collect when primary drainage does not function.

6.4.2
Roofs, gutters and the like shall either:

(a) Be sloped for free run off, allowing for long term deflection due to dead load and deflection due to the nominal depth of rain water (assuming undeformed surfaces), in which case rain water loads may be calculated assuming such nominal depths, or

(b) Be designed for loads which take into account the additional rainwater that collects due to load-induced sag.

6.4.3
For the purposes of determining appropriate load factors and combinations with other loads, rain water load shall be treated as snow load, and substitute for it.

6.5 Ice loads

6.5.1
Ice loads shall be considered within the zones where snow loading is applicable (see figure 6.3.2). Snow loads and ice loads need not be considered as acting together.

6.5.2
Ice loads shall be taken as being composed of both the weight of ice coating around the members and the wind load on the coated members calculated using a wind speed with a five year return period. This wind speed may be assumed to be 0.9 $V_{d(h)}$, where $V_{d(h)}$ is specified in 6.3.1.4.

6.5.3
In the absence of site specific data ice coating thicknesses shall be taken as follows;

For M_h =1.0: 30 mm on all faces
For M_h >1.0: 100 mm on the windward face and 30 mm elsewhere

where M_h is the hill shape multiplier obtained from Part 5.

6.5.4
At altitudes above 1500 m the ice loads to be considered as present at the time of an earthquake, and contributing to the total seismic weight and to the dead load, shall be calculated assuming a uniform ice coating thickness of 100 mm on all roof surfaces.

6.5.5
For the purposes of determining appropriate load factors and combinations with other loads, ice load shall be treated as specified in 6.5.4 or as snow load, as appropriate. Where it is treated as snow load, ice load shall substitute for the snow load.

6.6 Soil loads

6.6.1
Lateral loads on earth retaining structures shall be obtained by applying appropriate load factors to the soil and water weights and the surface loads, applying a strength reduction factor to the measured soil strength, and using established methods of soil mechanics.

6.6.2
Soil, including earth surcharge, and ground water shall be regarded as dead load for the purpose of determining appropriate load factors. Surface loads shall be factored appropriately.

6.6.3
The strength reduction factor for the soil shall be taken as 0.6, except in applications involving sloping cohesionless backfill material, where, provided due allowance is made for likely variations in backfill slopes, the dependable strength of the backfill need not be taken less than the result of assuming that the angle of internal friction is equal to the backfill slope and that the strength reduction factor is unity.

6.6.4
Where, in the event of failure of an embankment which is part of a soil-retaining system, there is risk to life, or to adjacent buildings, or to services to buildings of Category I or II, the embankment shall be shown to be stable, as required by 2.5.3.4, under the most adverse likely conditions, using a strength reduction factor as specified in 6.6.3.

6.7 Ground water loads

6.7.1
The hydrostatic pressure of water acting on surfaces below ground level shall be assessed from the maximum likely water level. Where the level is affected by flooding, heavy rain, or variations in sea level or the like, the level shall have an annual probability of exceedance as given in table 6.7.1.

Table 6.7.1 – Building category factor for hydrostatic ground water load

Building Category	Annual probability of exceedance
I	1/500
II	1/300
III	1/150
IV	1/100
V	1/50

6.7.2
For the purposes of determining load factors, ground water load shall be treated as variable dead load.

APPENDIX A
NOTATION

The following summarizes, in alphabetical order, the notation employed in all parts of the Code of practice.

A Tributary area of floor, the area from which the total load supported by the member is accumulated – Part 3

A Surface area of an element or tributary area which transmits wind forces to the element – Part 5

A_z Area of a structure or a part of a structure, at height z, upon which the design wind pressure (p_z) operates – Part 5

a Dimension used in defining the extent of application of local pressure factors – Part 5

b Breadth of a structure normal to the windstream – Part 5

b_s Average breadth of shielding buildings, normal to the windstream – Part 5

C Lateral force coefficient for the equivalent static method – Part 4

C(T) Ordinate of the design spectrum (given in units of g) at period T – Part 4

$C_b(T,\mu)$ Basic seismic acceleration coefficient which accounts for different soil conditions, structural ductility factors, μ, and transitional periods of vibration, T_i – Part 4

C_c Coefficient for building category – Part 6

C_d Drag force coefficient for a structure or member in the direction of the windstream – Part 5

C_e Coefficient for building exposure – Part 6

C_F Frictional drag force coefficient which is resolved into $C_{F,x}$ and $C_{F,y}$ along the member axes – Part 5

C_{fi} Floor coefficient at level i – Part 4

C_{fn} Floor coefficient at the level of the uppermost principal seismic weight – Part 4

C_{fo} Floor coefficient at and below the base of the building – Part 4

C_{pe} External pressure coefficient – Part 5

C_{ph} Basic horizontal seismic coefficient for a part – Part 4

C_{pi} Basic horizontal seismic coefficient for a part at level i – Part 4

C_{pi} Internal pressure coefficient – Part 5

$C_{p\ell}$ Net pressure coefficient for the leeward half of a free roof; or external pressure coefficient for a bin, silo or tank of unit aspect ratio – Part 5

C_{pn} Net pressure coefficient for canopies, free standing roofs, walls, etc. – Part 5

C_{pv} Basic vertical seismic coefficient for a part – Part 4

C_{pw} Net pressure coefficient for the windward half of a free roof – Part 5

C_r Coefficient for roof slope – Part 6

c Net height of a hoarding, bin, silo or tank – Part 5

D Down-wind roof slope – Part 5

d Depth of a structure parallel to the windstream; or span of a curved roof – Part 5

d_a Along-wind depth of a surface, m, =d for walls and flat roofs – Part 5

d_t Depth of the transition zone over which the wind profile becomes established with changes in terrain roughness, m – Part 5

E Site elevation above mean sea level, m – Part 5

E_s Earthquake forces for the serviceability limit state –Part 2

E_u Earthquake forces for the ultimate limit state – Part 2

F Wind force normal to the element surface – Part 5

F_d Wind induced drag force parallel to the windstream – Part 5

F_f Wind induced frictional force parallel to the windstream – Part 5

F_i Equivalent static lateral force at level i ; or inertial force at level i found from combination of modal inertial forces – Part 4

F_n Inertial force at the height of the uppermost principal seismic weight, h_n, used for the design of the building structure, either the equivalent static force or the force from the combination of modal forces, as appropriate to the method of analysis used for the building structure – Part 4

F_{ph} Horizontal seismic force on a part – Part 4

F_{pv} Vertical seismic force on a part – Part 4

F_x Wind force component resolved along the x-axis of a body – Part 5

F_y Wind force component resolved along the y-axis of a body – Part 5

G Dead load – Part 2

g Acceleration due to gravity. To be taken as 9.81 m/s^2 – Part 4

H Height of the crest of a hill, ridge or escarpment above a valley floor up-wind of the feature – Part 5

h Effective height of a structure – Part 5

h_c Height from ground to the attached canopy, etc. – Part 5

h_e Vertical height from the eaves of a building to the ground immediately below – Part 5

h_i Height of level i above the level where the ground provides lateral restraint to the structure – Part 2 and 4

h_n Height from the base of the building to the level of the uppermost principal seismic weight – Part 2 and 4

h_s Average height of shielding buildings – Part 5

h_t Vertical height from the apex of a building to the equivalent ground surface immediately below – Part 5

i A level or storey of a building. NOTE – Level i is immediately above storey i – Part 4

K_a Area reduction factor – Part 5

K_{ar} Aspect ratio correction factor for individual member forces – Part 5

K_i Factor to account for the angle of inclination of the axis of members to the wind direction – Part 5

K_l Local pressure factor – Part 5

K_m Modal analysis scaling factor – Part 4

K_p Reduction factor for porous cladding – Part 5

K_{sh} Shielding factor for multiple open frames – Part 5

k_c Multiplier for C_{pe} (on circular tanks, bins and silos) – Part 5

L_s Limit state factor for the serviceability limit state – Part 4

L_u Limit state factor for the ultimate limit state – Part 4

L_u Horizontal distance up-wind from the crest of a hill, ridge or escarpment to a level half the height below the crest – Part 5

l Length of a frame member; or length of a cantilevered roof beam – Part 5

l_s Average spacing of shielding buildings – Part 5

M_c Channelling multiplier – Part 5

$\overline{M}_c$ Channelling multiplier used with the dynamic wind design procedure – Part 5

M_e Site elevation multiplier – Part 5

M_h Hill-shape multiplier – Parts 5 & 6

$\overline{M}_h$ Hill-shape multiplier used with the dynamic wind design procedure – Part 5

M_{ls} Limit state multiplier – Part 5

M_L Lee multiplier – Part 5

$\overline{M}_L$ Lee multiplier used with the dynamic wind design procedure – Part 5

M_r Structure risk multiplier – Part 5

$\overline{M}_r$ Structure risk multiplier used with the dynamic wind design procedure – Part 5

M_s Shielding multiplier – Part 5

M_t Topographic multiplier (the maximum of M_p, M_L and M_c) –Part 5

$\bar{M}_t$ Topographic multiplier used with the dynamic wind design procedure – Part 5

$M_{(z,cat)}$ Site terrain/height multiplier – Part 5

$\bar{M}_{(z,cat)}$ Site terrain/height multiplier used with the dynamic wind design procedure – Part 5

n Maximum value of i – Part 4

n Number of spans of a multi-span roof – Part 5

n_s Number of up-wind shielding buildings within a 45° sector of radius 20 h_t and with height $h>h_t$ – Part 5

p_e External wind pressure, kPa – Part 5

p_i Internal wind pressure, kPa – Part 5

p_n Net wind pressure, kPa – Part 5

p_z Design wind pressure at height z, kPa – Part 5

Q Reduced live load – Parts 2, 3 & 4

Q_b Basic live load – Parts 2 & 3

Q_s Live load for the serviceability limit state – Part 2

Q_u Live load for the ultimate limit state for combinations of load involving dead load and live load with other loads – Parts 2 & 4

q_h Design wind pressure at height h, kPa – Part 5

q_{hc} Design wind pressure at height h_c, kPa – Part 5

$q_{(z)}$ Design wind pressure at height z resulting from the design wind speed $V_{(z)}$, kPa – Part 5

R Risk factor for a structure, – Part 4

R_p Risk factor for a part – Part 4

r Corner radius of a structural shape, m; or the rise of a curved roof, m – Part 5

S Basic snow load for sloping roofs – Parts 2 & 6

S Sidewall – Part 5

S_g Open ground snow load, kPa – Part 6

S_m,S_{m1},S_{m2} Response spectrum scaling factors – Part 4

S_s Snow load, ice load, or rain load for the serviceability limit state – Part 2

S_u Snow load, ice load, or rain load for the ultimate limit state (for ice load, includes wind load on coated members) – Part 2

s Shielding parameter – Part 5

T, T_i Translational periods of vibration – Part 4

T_p Fundamental translational period of vibration of a part – Part 4

T_{pe} Equivalent period for a part – Part 4

T_1 Fundamental translational period of vibration – Part 4

U Up-wind roof slope – Part 5

u_{el} Elastic component of the deflection of a building at the level of the uppermost principal seismic weight – Part 4

u_i Lateral displacement of the centre of mass at level i – Part 4

V Basic directional wind speed for the region and direction under consideration, m/s – Part 5

V Horizontal seismic shear force acting at the base of a structure – Part 4

$V_{base(1)}$ Combined modal shear at the base of a building determined for the direction being considered taking $S_m = 1$ – Part 4

$V_{d(h)}$ Wind speed at the top of a building for the serviceability limit state – Part 6

$V_{d(z)}$ Design wind speed at height z derived from $V_{(z)}$ to suit the building orientation, m/s – Part 5

$V_{d(z)}$ Hourly mean design wind speed, m/s – Part 5

V_i Storey shear in storey i – Part 4

$V_{(z)}$ Site wind speed at height z for the direction under consideration, m/s – Part 5

$V_{(z)}$ Hourly mean site wind speed, m/s – Part 5

W Windward wall – Part 5

W_i Seismic weight at level i – Part 4

W_n Seismic weight at height h_n – Part 4

W_p Seismic weight of a part – Part 4

W_s Wind load for the serviceability limit state – Part 2

W_t Total seismic weight of a structure – Part 4

W_u Wind load for the ultimate limit state – Part 2

w_c Width of canopy, etc., from the face of a building – Part 5

x Horizontal distance from a site to the crest of a hill or escarpment, m – Part 5

Z Zone factor – Part 4

z Distance or height above the ground, m – Part 5

α Angle of slope of a roof, degrees – Parts 5 & 6

β Angle from the wind direction to a point on the wall of a circular bin silo or tank – Part 5

δ Actual solidity ratio equal to the ratio of solid wall area to total wall surface area – Part 5

δ_e Effective solidity ratio for an open frame – Part 5

δu_i Interstorey deflection at storey i divided by the storey height – Part 4

θ Angle between the wind direction and the axis of a building; or angle between the wind direction and the reference axis of a structural section – Part 5

λ Spacing ratio for open frames – Part 5

μ Structural ductility factor – Part 4

μ_o Structural ductility factor calculated using the overstrength values – Part 4

μ_p Structural ductility factor used for the design of a part – Part 4

ν Kinematic viscosity – Part 5

ϕ Upwind slope of a hill, ridge or escarpment – Part 5

ϕ_d Average down-wind slope measured from the crest of a hill, ridge or escarpment to the ground level at a distance of 5H – Part 5

ψ_a Area reduction factor for live load – Parts 2 & 3

ψ_l Long term factor for the serviceability limit state live loads, snow, ice, or rain loads – Parts 2 & 4

ψ_s Short term factor for the serviceability limit state live loads, snow, ice, or rain loads – Part 2

ψ_u Live load combination factor for the ultimate limit state – Parts 2, 3 & 4

INDEX

NZS 4203:1992

Code of practice for
GENERAL STRUCTURAL DESIGN AND DESIGN LOADINGS FOR BUILDINGS

Volume 1 Code of Practice

CORRIGENDA

January 1994

Figure 4.6.1(a) (page 41)
Delete % sign on vertical axis of graph.

Figure 4.6.1(b) (page 42)
Delete % sign on vertical axis of graph.

Figure 4.6.1(c) (page 43)
Delete % sign on vertical axis of graph.

Figure 5.6.1 (page 75)
Top right hand diagram
Delete "$\theta = 0^o$" and **replace** with "$\theta = 90^o$".

Figure 5.7.6 (b) (page 88)
Delete dimension "c" and **replace** with "h".

Figure 5.B4.2 (page 109)
Under heading "Force coefficients" **delete** "$C_{F,x}$" and **replace** with "$C_{F,y}$".

Volume 2 Commentary
Clause **C5.3.1** (page 67)
In example C5.3.1 **delete** line "Importance multiplier $M_i = 1.0$".

Clause **C5.4.2.1** (page 73)
Delete last sentence of first paragraph:
"It should be noted that in region V, the south westerly, southerly and south easterly winds have been magnified by 10 % to allow for channelling effects known to occur in this region".

STANDARDS NEW ZEALAND
PRIVATE BAG 2439
WELLINGTON 6020

NOTES

Approved by the Standards Council on 16 October 1992 to be a New Zealand standard pursuant to the provisions of section 10 of the Standards Act 1988.

First published: 14 December 1992

Reprinted with Corrigenda appended: 31 January 1994

Reprinted: May 2015

The following references relate to this standard:

Project No. P 4203

Draft for comment No. DZ 4203

Typeset by: Standards New Zealand

STANDARDS® NEW ZEALAND
PAEREWA AOTEAROA

NZS 4203:1992

Corrigenda appended

New Zealand Standard

Code of practice for general structural design and design loadings for buildings

(Known as the Loadings Standard)

Volume 2: Commentary

Superseding NZS 4203:1984

NZS 4203:1992

COMMITTEE REPRESENTATION

This standard was prepared under the direction of the Building and Civil Engineering Board, (30/-) for the Standards Council, established under the Standards Act 1988 (1965).

The first draft for comment of the Standard was prepared by an ad hoc committee consisting of:

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Mr G.W. Butcher

Dr D.L. Hutchison

Mr D. Kolston

The second draft for comment and this Standard were prepared by the Loadings Code Revision Committee, (42/12) consisting of:

Mr A B King	nominated by the Building Research Association of New Zealand. Chairman
Mr G C Clifton	nominated by New Zealand Heavy Engineering Research Association
Dr R C Fenwick	nominated by New Zealand Vice Chancellors Committee. Withdrew from committee before the Standard was completed.
Mr I A Fraser	nominated by the Association of Consulting Engineers New Zealand
Mr R D Jury	co-opted
Dr G McVerry	nominated by the Department of Scientific and Industrial Research
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ACKNOWLEDGEMENT

Standards New Zealand gratefully acknowledges the contribution of time and expertise from all those involved in developing this standard.

Published by Standards New Zealand, the trading arm of the Standards Council, Private Bag 2439, Wellington 6140. Telephone: (04) 498 5990; Fax: (04) 498 5994; Website: www.standards.co.nz.

AMENDMENTS

No.	*Date of issue*	*Description*	*Entered by, and date*

CONTENTS **PAGE**

(Contents continued overleaf)

(Contents continued)

PART 5 PROVISIONS FOR WIND FORCES

PART 6 PROVISIONS FOR SNOW LOADS, RAIN WATER PONDING LOADS, ICE LOADS, SOIL LOADS, AND GROUND WATER LOADS

(Contents continued overleaf)

Table

Figure

RELATED DOCUMENTS

Reference is made in this document to the following:

NEW ZEALAND STANDARDS

NZS 3404:1989 The steel structures code

NZS 3603:1990 Code of practice for timber design

NZS 4219:1983 Specification for seismic resistance of engineering systems in buildings

AUSTRALIAN STANDARDS

AS 1170:- - - - Minimum design loads on structures
Part 1:1989 Dead and live loads and load combinations
Part 2:1989 Wind loads

AS 1418:- - - - Cranes (including hoists and winches)

BRITISH STANDARDS

BS 6399:- - - - Loading for buildings
Part 1:1984 Code of practice for dead and imposed loads
Part 3:1988 Code of practice for imposed roof loads

AMERICAN NATIONAL STANDARDS

ANSI /ASCE 7-1988 Minimum design loads for buildings and other structures (Revision of ANSI A58.1-1982)

BUILDING INDUSTRY AUTHORITY

New Zealand Building Code, 1992

NOTES

COMMENTARY

PART 1 SCOPE AND INTERPRETATION

C1.1 Scope

C1.1.1 *General*
Buildings and parts covered in this Standard are defined in the Building Act. However the Standard does not apply to special structures such as vehicular bridges, towers, dams, major storage tanks, mechanical plant, or industrial equipment. The general structural requirements for buildings, as distinct from detailed design appropriate to particular construction materials, are specified in the New Zealand Building Code (NZBC).

C1.1.2 *Unusual buildings*
A special study may be made at the initiative of the designer for any building, subject to satisfying the requirements of the NZBC. Any special study may make use of the provisions of this Standard, either directly or as a general guide, provided they are used in context.

C1.1.3 *Non-brittle behaviour*
The seismic design acceleration coefficients and other requirements in Part 4 include allowances for overstrengths, redundancies and inherent ductility and damping which are present in typical building structures, but are not necessarily accounted for in the design process. Most common building materials, such as reinforced concrete, structural steel and timber framing, exhibit some level of ductility, even in buildings designed as "elastically responding structures".

The design spectra adopted in this Standard have been factored down from the uniform hazard spectra for the appropriate limit state, to reflect the likely better performance of typical buildings than assumed using standard design procedures (refer also C2.2.2 and C4.6). For this reason the seismic design provisions of this Standard may not be applicable to elements or structures which exhibit brittle behaviour, such as exhibited by many ceramic components unless a higher value of the "structural performance factor" is adopted.

Under certain conditions brittle behaviour may also be exhibited by more common building materials. Examples of these include cross-grain tension failure of timber members, tension failure of light gauge steel nailplates, flexural and consequent shear failure of plain concrete foundations, and failure of some configurations of welds. A cautious evaluation of the applicability of the seismic design provisions of susceptible elements is required, especially where they are used in "bare structure" configurations with little or no redundancy.

C1.2 Interpretation
This Commentary is intended to be read in conjunction with the Code of practice in Volume 1. It not only explains the provisions of the Code of practice, but in certain cases it suggests approaches which satisfy the intent of the Code of practice. Commentary clauses are not mandatory. At the end of each commentary section a list of references is provided.

Clause numbering of the Commentary is identical to that of the Code of practice except that clauses are prefixed with the letter ‘C’. A cross-reference such as 5.4.1.2 refers to that clause in the code of practice, while C5.4.1.2 refers to the corresponding commentary clause.

PART 2 GENERAL REQUIREMENTS

C2.1 Notation

C2.1.1
The following symbols appearing in this Part of the Commentary are additional to those used in Part 2 of the Code of practice.

C_h (T,μ),$C_h(T_i,\mu)$	Basic seismic hazard acceleration coefficient which accounts for different soil conditions, structural ductility factors, μ, and translational periods of vibration. T_i or T (C2.5.4.6)
K_j	Load factor applicable to loads producing component j of the stabilizing moments (C2.5.3.4)
M^I	Moment about the centre of rotation, of the loads tending to cause instability (C2.5.3.4)
M^S	Moment about the centre of rotation, of the loads tending to resist instability, and scaled by a load factor of 0.9 (C2.5.3.4)
S_{m1}	Design spectrum sealing factor (C2.5.4.6)
T_1	Fundamental period of the building vibrating predominantly in translation (C2.5.4.6)
μ	Structural ductility factor (C2.5.4.4).

C2.2 Application

C2.2.1 *General*
Consistency is required in the formulation of loads and forces, in the manner in which they are combined, in the modelling of the structure, and in the analysis.

The principal objectives under the various limit states are the achievement of the following:

(a) All loads likely to be encountered during the life of the building will be sustained with an adequate margin of safety;

(b) Deformations of the building will not exceed acceptable limits;

(c) In events that occur occasionally, such as moderate earthquakes and severe winds, structural damage will be avoided and other damage will be minimized;

(d) In events that occur rarely, such as major earthquakes and extreme winds, collapse and irreparable damage will be avoided, and the probability of injury to or loss of life of people in and around the building will be minimized.

Towards the achievement of the principal objectives, some more general requirements are included in this Part. Others, more specific to the type of loading are included in other Parts of this Standard. In addition, requirements peculiar to the materials of construction (e.g. permissible crack widths in reinforced concrete) are included in the material standards. All requirements must be met.

C2.2.2 *Special studies*
As a guide for special studies, the aim of the designer should be that for the ultimate limit state loads there is not greater than a 5 % probability of exceedence in a 50 year period (975 year return period) or in the assumed life of the structure.

The probability of exceedence may be increased to 10 % for ultimate limit state earthquake forces (approximately 450 year return period).

In this Standard, the uniform risk spectra, resulting from a risk analysis utilising the attenuation of peak response values, have been reduced by a "structural performance factor" to provide the design values. Values of the "structural performance factor" may lie in the range 1.0 to 0.4 or be even smaller and will depend on such factors as structural type, structural importance, structural redundancy and the performance reliability required. The value of the "structural performance factor" adopted in this standard (ie 0.67) is considered appropriate for structures "on average".

The choice of an appropriate "structural performance factor" for special cases will rely on the designer's judgment and therefore should be approached with caution. This is especially the case for systems that respond in an inherently brittle manner. In such cases a value of the "structural performance factor" approaching 1.0 may be appropriate. Refer also to C4.2.4.1 and C1.1.3.

For the serviceability limit state loads and forces, a probability of exceedence of approximately 5 % in any year (20 year return period) would usually be appropriate, but subject to significant variation depending on the particular effect considered. In this Standard the probability has been increased to 10 % for the serviceability earthquake forces based on the uniform risk approach outlined above.

C2.3 Classification of buildings and parts

In Parts 3, 4, 5 and 6, Categories for buildings and parts are associated with multipliers by which infrequent loads are factored. The purpose is to obtain an acceptably low rate of loss of building function and to prevent injury to people in and around the building.

C2.3.1 *Buildings*

Examples of buildings for each of the five Categories are as follows:

Category I

(a) Designated civilian emergency centres and civil defence centres, such as essential hospital and medical facilities, (operating theatres and related treatment areas and their support facilities); ambulance, fire and police stations and buildings housing emergency vehicles and their fuel supply;

(b) Public radio and television transmitting facilities and telephone exchanges;

(c) Power stations and sub-stations;

(d) Maximum security places of restraint, such as blocks in some prisons and mental institutions;

(e) Industrial plant of national economic importance, where production cannot readily be taken over by another plant;

(f) Containment buildings for toxic and deleterious substances, the release of which could disrupt production in rural, forest or horticultural areas or could cause serious illness.

Category II

Residential buildings containing more than 500 people, assembly halls, theatres, school classroom buildings, airport terminals and main railway stations.

Category III

Buildings with contents of particular cultural or historical significance, such as art galleries and archival record depositories.

Category IV

Other buildings such as office buildings, residential buildings, industrial buildings, or warehouses, where not included in any other Category.

Category V

Outbuildings, some farm buildings and temporary buildings such as offices on a construction site.

C2.3.2 *Parts of buildings*

Examples of Categories of various parts are given below. These should be read in the context of the general description of the Category. For example, a small inexpensive motor associated with a life support system should be considered as Category P.I, but normally such a piece of rotating equipment would be considered as Category P.III.

Category P.I

(a) Heavy prefabricated elements and their connections above an exitway or public place at which screening does not protect people should the element collapse; exterior walls and glazing, and interior partition walls of specific weight more than 10 kN/m^3;

(b) Equipment designed to function in an emergency; pressure vessels, furnaces, steam boilers, and equipment conveying combustible, high temperature or toxic substances;

(c) Electrical busbars and primary cable systems, and lifts.

Category P.II

Communications systems and equipment designed to function in an emergency; vessels and lines supplying energy (transformers, switchgear etc.) important to the function of the building.

Category P.III

Items not included in Categories P.I or P.II.

C2.4 Load combinations

C2.4.1 *General*

In this Standard, the combinations are principally of load (or forces) rather than of load effects. However, it is recognized that designers may prefer to work with load effects such as element actions. Indeed for some components of the combinations, load effects (rather than loads and forces) are the product of the method of analysis, notably where the modal response spectrum method is used to determine earthquake actions. It should be noted, however, that components of the combinations may differ in nature and cause, and that the inelastic methods of analysis permitted for the ultimate limit state do not admit superposition of load effects. To stress these points, the ampersand (&) is used to signify symbolic aggregation of loads and forces or of their effects rather than the plus sign (+) used hitherto which implied admissible direct addition of load effects.

Soil loads are derived from dead load, live load, and earthquake forces for use in these combinations of factored loads. Snow load may need to be replaced with ice load (which includes wind load on coated members) or rain load where these loads are important. Hydrostatic loads are treated as variant dead load. Refer to Part 6 for detailed treatment of these loads.

Designers should note the occasional need to include combinations of load other than those listed in the Standard, as required by 2.4.1 (see also 2.5.3.4).

C2.4.2 *Serviceability limit state*

C2.4.2.1

$\psi_s Q$ is the value of live load with a 5 % chance of exceedence in one year (20 years return period). $\psi_l Q$ is the average value of live load over the assumed 50 year life of the building, also with a 5 % chance of exceedence. Analyses for the derivation of these factors are contained in Reference 2.1.

As an example of the use of the short term and long term factors, consider the calculation of deflection in a structure in which there is no yielding but in which creep may be significant. Deflections could be approximately found from analyses for G & $\psi_l Q$, with creep effects included, and for $(\psi_s - \psi_l)Q$, without creep effects. Addition gives an approximation to the total deflection to be expected.

It is the long term live load, $\psi_l Q$, which is to be used when live load is considered in combination with other time dependent loading conditions such as wind or earthquake.

C2.4.2.2

The most common result of failure to meet basic serviceability criteria is the occurrence of excessive deflection. Suggested limits on deflection are given in table C2.4.1. Further recommendations on the maximum limits for foundation settlement for a variety of structures may be found in Reference 2.2.

In some instances for calculation of deflection limit, the span will require modification to accommodate the effects of adjacent stiff elements, for example:

(a) Long span one way floor systems with adjacent parallel walls supporting floor edges;

(b) Flexible roofs with stiff gable end walls.

The design of some elements may require satisfying more than one serviceability limit for deflection, for example, floors may need to be designed for the worst case of all the different criteria of table C2.4.1.

Table C2.4.1 – Suggested serviceability limits for deflection

Control phenomenon	Element	Explanatory comment	Deflection limit	Loading regime	Action being considered
Sensory: *1 Seen			*2		
	Roof systems	Rafters, purlins	Span/300	$G \& \psi_l Q$	Sag
		Cladding	Span/85	Q_b = 1 kN	Indentation
	Ceilings	Flat finish	Span/500	G	Ripple *3
		Textured finish	Span/250	G	Ripple *3
		Suspended systems	Span/360	G	Ripple *3
	Ceiling supports		Span/360	G	Sag *3
	Glazing systems		Span/400	W_s	Bowing *4
	Columns		Height/500	G; W_s	Side sway
	Floors		Span/300	$G \& \psi_l Q$	Ripple *3
			Span/180	$G \& \psi_l Q$	Sag
	Beams	Line-of-sight Along invert	Span/500	$G \& \psi_l Q$	Sag *5
		Line-of-sight Across soffit	Span/250	$G \& \psi_l Q$	Sag *5
Felt	Walls		Height/150	W_s	Face loading
			Height/200	Q_b = 0.7 kN	Impact *6
	Floors		<1 mm	Q_b = 1.0 kN	Transient vibrations *7
		Acceleration limit	<0.01 g	W_s	Sway
Functionality *1					
	Flat roof		Span/400	$G \& \psi_l Q$	Drainage *8
			Span/500	$G \& S_s$	Ponding *8
	Walls		Height/300	W_s ψ	In-plane
	Floors	Normal occupancy	Span/400	$G \& \psi_s Q$	Sag *9
		Specialist floors	Span/600	$G \& \psi_s Q$	Sag *9
	Lintels		Span/240	$G \& \psi_s Q$	Jamming
			or <25 mm	$G \& \psi_s Q$	
Protection of non-structural elements *1					
	Roofs	Brittle claddings	Span/150	W_s	Cracking *10
	Ceilings	Stopped systems	Span/200	$G \& \psi_s Q$; W_s	Cracking
	Walls	Brittle claddings	Span/150	W_s	Cracking
	Glazing elements	Facades, and curtain wall fixed glazing	Span/250	W_s; E_s	In-plane
			2 x glass clearance	W_s; E_s	In-plane
	Masonry walls		Height/600	W_s; E_s	In-plane
			Height/400	W_s; E_s	Face loading
	Plaster/gypsum walls		Height/400	W_s	In-plane
			Height/200	W_s; E_s	Face loading
			Height/200	Q_b = 0.7 kN	Impact *6
	Moveable partitions		Height/160	Q_b = 0.7 kN	Impact
	Walls (other linings)		Height/300	W_s; E_s	Face loading
	Portal Frames		Spacing/200	W_s; E_s	In-plane *11
	Floors	Supporting masonry walls	Span/500	$G \& \psi_s Q$	Wall cracks
		Supporting framed walls	Span/300	$G \& \psi_s Q$	Joint cracks

Notes to Table C2.4.1

This table identifies deflection limits beyond which serviceability problems have been observed. Such boundaries are imprecise and require a degree of engineering judgement in their application. Where short term live loads, $\psi_S Q$, are prescribed, refer to C2.4.2.1 for guidance as to how long term deflection phenomena such as creep are to be considered.

*1 Enter this table by determining the reason for limiting deflection (Sensory – seen or felt; Functionality – use; or Protection of non-structural elements). Different deflection limits may apply to elements depending on the reason for limiting the deflection, and the most stringent appropriate limit is to control. People's tolerances to sensory deflections are influenced by the environment of the observer. Where a lot of movement is occurring, the stated sensory limits may be exceeded without resulting complaint.

*2 Deflection criteria are usually stated in terms of span ratios for a given load. The span is to be considered as the clear spacing between points of support. For vertical elements, height has been substituted for span.

*3 The deflection limits for sheet lining materials (ceilings or floors) are heavily influenced by the surface finish. Ripple effects appear more pronounced when the surface is flat (and have a reflective gloss finish). Textured surfaces tend to disguise ripple effects. Surfaces which extend over a wide expanse reveal both ripple and sag effects when light is reflected from the surface.

*4 Reflective surfaces amplify apparent bowing as the reflected images move with the surface distortions. Observers are readily disturbed by such movements.

*5 Problems with visually sensed deflections are frequently dependent on the presence of a visual cue for the observer to gauge linearity. Deflection limits which might be acceptable are a function of the line of sight of the observer.

*6 Internal walls and partitions require stiffness control to minimize disturbance of elements or people often on the reverse side of the partition. The response of the wall to soft-body impact is greatly influenced by the nature and characteristics of the impacting body. The deflection criteria stated (span/200 from a concentrated load of 0.7 kN at mid-height) has been simplified for ease of application by designers. It is based upon a running person falling against a wall. Internal partitions may be subjected to differential pressures which result from wind. A net pressure coefficient of 0.8 may be considered appropriate when used in conjunction with the serviceability wind pressures.

*7 Floor vibration problems are very complex. Problem floors usually have low levels of elastic damping present. The limiting criteria stated (<1 mm under a 1 kN point load) should give a guide as to whether the floor may have vibration problems. When the criterion is not satisfied, a more detailed examination of the dynamic behaviour of the floor may be merited. (Refer also to 2.5.2.3).

*8 Flat roofs are susceptible to ponding when drainage is precluded. Where parapets are used, or with wide span flexible roofs, the weight of the water and snow may induce instability as a result of ponding deflections. (Refer also to 6.4.2).

*9 Floor sag may result in furniture which rocks, or is not firmly seated, or drainage surfaces which do not function adequately. Specialist floors are those upon which trolleys may move, sensitive equipment may be installed or special activities (e.g., bowls etc.) may be undertaken. More restrictive deflection limits are appropriate in such cases.

*10 Brittle cladding elements may become cracked when the substrate moves excessively during wind gusts.

*11 The limiting deflection of portal frame knee deflections is related to the behaviour of the cladding between the 'free portal' and a more rigid plane (typically the end wall of a structure). The deflection limit of such portals is based on the bay spacing and ability of the cladding to withstand in-plane shear distortion.

The above deflection limits are based on similar tables published in Reference 2.3. The values have been modified where appropriate by the deflection factor, discussed below, which addresses the changes to the level of the applied live loads and the serviceability wind pressure outlined elsewhere in this document.

C2.4.2.3
The self-equilibrating internal strain effects are those associated with foundation settlement, prestress, welding, shrinkage, creep, temperature effects, and the like. Several of the effects are interrelated and dependent on materials of construction (for example shrinkage and creep in concrete structures). The effects will differ for each load combination, and should be assessed in a manner which is consistent with the source of the other loads in the combination. For example, creep effects due to live load will be significant under the long-term component, but of little significance for an increase beyond that level.

The detailed specification of these effects is beyond the scope of straightforward rules in this Code of practice. Details are left to the knowledge, experience, and judgement of the designer who should consult the material standards for guidance. Additional information is contained in References 2.3 to 2.9.

The effects are most important in the serviceability limit state when they may lead to large deflections, cracking, or yield, but in general they disappear when plastic mechanisms form and have not therefore been included in the load combinations specified for the ultimate limit state. It is important to appreciate, however, that these effects may lead to reduced strength. For example, shrinkage may reduce the strength of bolted joints in restrained timber assemblies, or reduce shear strength in concrete members restrained longitudinally. Such reduction in strength must be taken into account, in accordance with the provisions of the material standards.

C2.4.3 *Ultimate limit state*

C2.4.3.1
The working stress method (called the Alternative method in the 1976 version of the Standard and in subsequent amendments), uses stresses derived from the ultimate strength of the materials divided by a factor of safety. This factor is intended to ensure adequate safety under loads which latterly have been derived from combinations of factored loads specified for the strength method of design.

For this revision of the Standard, the working stress method is no longer permitted for the ultimate limit state. Use of equivalent methods, however, is not excluded, but use will need to be made of the listed combinations of factored load and the dependable limit stresses derived from the 5 % exclusion limit stresses and the strength reduction factors appropriate to the material. The material most affected is timber, for which NZS 3603 is (1992) in the process of revision. Changes of emphases and associated format have also been proposed for other material standards, such as NZS 3404 for steel.

The same sets of combinations of factored load are used for whatever method of analysis is employed. For the plastic method, which traditionally has assumed proportional loading, this is a significant change. To determine the deflection history of a structure designed by the plastic method, if desired, an elastic-plastic method should be employed, as always, but usually it will now be necessary to assign fixed (e.g. 1.2 G) and variable (e.g. Q) components of load with progressive incrementation on the variable load component only (e.g. a live load factor progressively incremented to 1.6).

C2.4.3.2
The specified values of live load are meant to be the peak values of live load for a 50 year life time with a 5 % chance of exceedence. The live load combination factor for the ultimate limit state is used in combinations involving rare events to account for the reduced probability that maximum live load will act simultaneously with peak values of these rare event loads.

C2.4.3.3
Combination (1) is to ensure adequate reliability of structures dominated by dead load: the reliability decreases for small values of the ratio Q/G [2.10]. Consideration must also be given to the possibility that some components of dead load, the so-called superimposed dead load, may be absent.

C2.4.3.4

(a) Requirements for structural stability during fire are detailed in the New Zealand Building Code, Clauses C2, C3 and C4. Under the listed load combination the reduced strength of members exposed to high temperatures must be taken into account, in accordance with the material standards.

(b) Designated elements are typically external walls or vertical fire separations. In the approximate method, the specified face load is a simple means of applying a destabilizing action.

C2.4.3.5
"Skip" loading is usually not considered appropriate when wind or earthquake forces are acting. For fire emergency conditions more complex phenomena, such as impact of collapsing elements and thermally induced deformations, are likely to have more important effects.

C2.4.3.6
The live load Q_u is thus taken to be always present, for the calculation of period, in the assignment of strength to plastic hinges and other yielding members, and in applying capacity design.

C2.5 Analysis and design

C2.5.1 *General*

C2.5.1.1
Reliable procedures should be consistently used throughout the processes of analysis, design and specification of construction requirements. Guidance on the choice of member properties should be sought in the material standards.

C2.5.1.2
Second-order effects need not involve classical buckling instability to be of importance. Consideration of second-order effects needs to take account of deflections along a member and relative deflections between its ends, both of which promote further increases in deflection and increased strength demands. These effects need to be considered for all load cases and both limit states. Guidance should be sought in the material standards.

P-delta effects considered for the building as a whole will normally be significant only in the ultimate limit state for load cases involving wind or earthquake, but occasionally will also be significant where sidesway under gravity load occurs.

C2.5.1.3
As an example, a stiff element, though weak, can attract seismic forces to itself. This may have adverse effects on the structure. Other elements may be considered not to contribute to lateral resistance of the building, but they should nevertheless be designed to fulfil their role while subject to lateral deflection imposed by the response of the building.

C2.5.1.4
Foundations, including piles, and the supporting soils with which they interact should be treated as part of the overall building structure and analysed as such.

Flexibility of foundations affects the response characteristics of the building by affecting period, drift, and the like, and affects the relative participation between dissimilar systems in the resistance of lateral forces, such as between structural walls and frames.

C2.5.2 *Serviceability limit state*

C2.5.2.1
Significant yielding is not appropriate in the serviceability limit state and therefore use of the elastic method is recommended. However, adjustments from linear elastic assumption may be appropriate to account for creep, shrinkage, cracking, or yielding of compressive reinforcement in reinforced concrete, and scaling procedures are available to provide approximations to service load deflections where the plastic method has been used for design [2.11].

C2.5.2.2
Care should be exercised to ensure that the damage associated with any adjustments from the elastically derived actions is not cumulative – the structure should "shake-down" to a stable state under repeated application and removal of the variant loads. Guidance should be sought in the material standards.

C2.5.2.3
It is important to ascertain the appropriate level of applied live load to use in the assessment of the in-service vibration characteristics of a floor system, as vibration problems are normally encountered under relatively light live loading.

In the calculation of the fundamental frequency, it is recommended that, typically, the short-term live load (ψ_S Q) be applied. This value may be unconversatively high in a subsequent dynamic analysis, which should involve a realistic minimum level of design live load appropriate to the nature of the activity. Guidance is available [2.12, 2.13].

C2.5.3 *Ultimate limit state – General*

C2.5.3.1
Provided care is exercised to ensure that the required strength can be realised when large inelastic strains are implied, designers need not be constrained to use elastic methods. With that proviso, any statically or kinematically admissible analysis technique may be used, but elastic-plastic analyses are generally recommended since these include direct computation of deformations. It is intended (1992) that the material standards be expanded to include specification of maximum permissible deformations, but in the interim designers will need to rely on published results of testing or reliable theoretical appraisals.

During an earthquake, inertia forces continually change in magnitude, direction, and pattern. Therefore, accurate prediction of inelastic deformations likely to occur during an earthquake cannot be made using an elastic-plastic analysis which assumes undirectional earthquake loading (see C4.7.3.1). Nevertheless, such analyses provide information on the likely order of these deformations, and lead to better estimates of structural performance than do methods relying on limited redistribution of elastically derived actions. This Standard therefore permits the use of such analyses.

C2.5.3.2
One of these methods is likely to be chosen for routine design. Examples of where the plastic method is permitted include steel frames of limited number of storeys and yield line analysis of plates and slabs; and assumptions of plastic equilibrium are commonly employed for soil retaining structures. Where redistribution from the actions found from an elastic analysis is undertaken, care should be taken to ensure that the structure remains in a statically admissible state – all actions should be adjusted to be consistent with the adjustments made to the redistributed actions.

C2.5.3.4
Production of generally applicable rules for specifying adequate stability is fraught with difficulty, not only as to the means by which stability is to be measured, but also in quantifying adequacy.

Some established methods are suspect. For retaining walls, for example, it has been common practice to measure overturning stability by the ratio of resisting moment to overturning moment, both related to the toe of the wall. This clearly produces inconsistent results, related to the bearing pressures which may force yield before the predicted limit loads are reached, at which stage the centroid of the bearing pressures forms the pivot for overturning.

One common method used to force adequate structural proportions relies on factoring stabilizing loads by a smaller multiplier than for the destabilizing loads. This leads to physically unrealistic conditions and often to numerical complexity. This method is scarcely improved by scaling actions (or load effects) instead of the loads directly, recognizing that load effects are not linearly related to the loads or combinations of loads that produce them, although this approach is usually more straightforward in application.

The method specified in this Standard uses a blend of the two methods outlined above.

(a) It encourages designers to investigate the manner in which instability may occur;

(b) It allows the factored loads used for structural strength evaluation to be used for stability calculations. This is a major concession where strength is interactive with applied load, or where a pivot for a rotational mechanism depends on the intensity and position of applied load resultants.

(c) The stabilizing and destabilizing components will usually be obvious on study of the postulated mechanism. What might be expected to be a destabilizing load may be subsequently revealed as stabilizing.

(d) The same results can be accomplished by recourse to the principle of virtual work, usually much more simply, but the specified method has the advantage of more familiar appearance.

(e) Destabilizing actions are factored in the same way as the applied loads and forces. No scaling is therefore required for them.

(f) This might be construed as implying that stabilizing loads carry a load factor of 0.9. However, the loads and their factors are not varied. Stabilizing actions are factored by $0.9/K_j$. The resulting ratio may be considered a "strength" reduction factor applied to the stabilizing action. The effect is to add to the destabilizing actions a further action of the same kind, given, for moments, by:

$$dM^I = \Sigma_j \left[(1 - 0.9/K_j) M^S \right] \qquad \text{(Eq. C2.5.1)}$$

In the case of load combination (4) of 2.4.3.3, dM^I for dead load is zero, because $K_j = 0.9$ for dead load in that combination.

(g) Often there will be no structural strength mobilized.

(h) This test of stability will usually be employed as a check, rather than for direct design.

Stability under earthquake (e.g., rocking) is in a different class of stability problem and is dealt with elsewhere in this Standard.

Figure C2.5.1 illustrates some examples of typical circumstances in which the provisions may control. Refer also to C6.6.1 and to example C6.6.1.

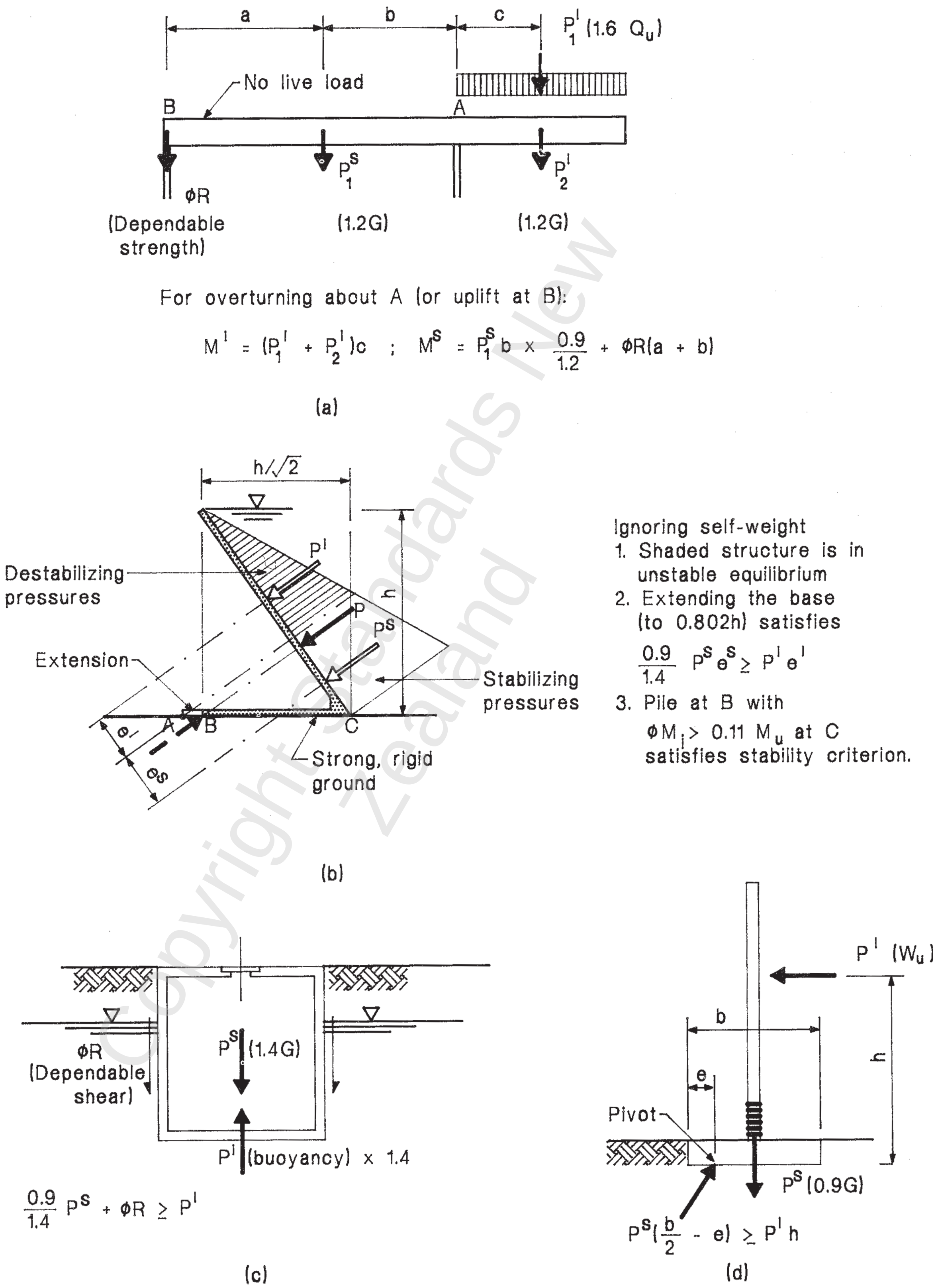

Figure C2.5.1 – Examples of stability considerations

C2.5.4 *Ultimate limit state – Additional requirements involving earthquake effects*

C2.5.4.1
Rarely will the actual strength of the structure, as detailed, coincide with the strength level associated with a pre-chosen ductility factor. The actual strength can, however, be established, and from this the required ductility factor can be found. It may be desirable to determine its value, as it is the global ductility factor which sets levels of inelastic deflections, and member deformations determined from it are used for the determination of detailing standards.

C2.5.4.3
For long fundamental periods, the deflection at the top of the structure is equal to that calculated for elastic response (“equal displacement” concept applies), and is therefore independent of the ductility factor. For shorter periods, this displacement is a function of the ductility factor and is greater than that corresponding to elastic response.

C2.5.4.4
For effects of foundation flexibility, refer to C2.5.1.4

The designer should determine whether the foundations are to be part of the energy dissipating system or are to remain essentially elastic through the application of the principles of capacity design. While this Standard does not prohibit yielding of foundation elements at low levels of load consistent with the choice of large ductility factors for the building as a whole, it is recommended that yielding should be inhibited by assigning greater strengths to foundation elements relative to elements of the superstructure.

Foundations of ductile structures should generally be designed to remain elastic, but it is widely held that foundations designed for loads corresponding to $\mu = 3$ for the building as a whole will be sufficiently protected against early yield to ensure adequate protection in moderate earthquakes.

Rocking of individual footings is not essentially different from yielding of foundation elements. Exceptions to this similarity are the effects on building deformation, especially where footings are wide, the effects of possible reduced energy dissipation, and reduced potential for corrosion. Uplift of a footing is also permitted, but the effect should be recognized in the analysis and design of the building as a whole. For example, uplift of an exterior column will reduce the resistance of the entire frame by limiting the development of potential strength in the exterior bay, but this may be acceptable if the other bays are adequately strong. In this case the designer should consider the effects of reduced energy dissipation, the effects of unidirectional hinging and the associated deformation of the frame, and modified approaches in the determination of design actions – it is inappropriate to use established methods of capacity design for the axial loads in the column when the column axial tension cannot be sustained at footing level. Refer also to C4.11.1.2.

C2.5.4.5
The deflection limits reflect that demands on element integrity and deformation may become excessive when interstorey slopes are large. At large interstorey slopes, difficulties can be expected in ensuring adequate performance of elements under reversing load with likely severe strength degradation and irreparable damage. The need to limit deflections increases with increasing building height.

Limits on interstorey deflection for the equivalent static and modal response spectrum methods are smaller than for the numerical integration time history method. This is because the former, being elastic analysis methods, determine less accurately the deflection profile of an elastic

C2.5.4.6
Capacity design provides an assurance that the behaviour of the building will be predictable, and that the failure mechanisms will be those capable of providing significant energy dissipation through ductile yielding in the event of a severe earthquake.

To date (1992), methods developed for the detailed application of capacity design are material dependent. This recognizes that modes of failure which may be unacceptable in one material may be acceptable in another (e.g. shear failure in reinforced concrete is generally unacceptable, but shear flow in structural steel is an acceptable form of energy dissipation in, for example, eccentrically braced frames). Guidance in the application of capacity design should therefore be sought in the material standards.

(a) Guidance on the calculation of overstrength, which may be a function of inelastic strains to be expected, should be sought in the material standards.

(b) Since structures may deform in all directions for significant periods, hinging of several beams (or diagonal braces or the like) framing into columns or walls is likely. This is the concurrency effect to be considered in design of columns, joints, and foundations, for flexure, axial load, and shear.

(c) Dynamic response may produce departures from the patterns of moments and forces derived in the analysis for the loads specified in Part 4. The modes of deformation sustained during an earthquake differ from those predicted by an elastic analysis due to higher mode effects associated with plastic hinge formation. In effect, this sub-clause recognizes that patterns of actions which differ from those specified in Part 4 develop during earthquakes.

(d) Actions resulting from the application of capacity design are expected to be extreme values. Consequently the use of strength reduction factors, which partly allow for overload and for possible inelastic behaviour, need not be used.

Member actions generated during large inelastic response to earthquake excitation may depart markedly from those calculated by simply scaling actions derived from an elastic analysis for loading specified in Part 4. Capacity design recognizes this, and is therefore most important where large ductility factors are used in design. Where the design ductility factor is small, so that the forces specified in Part 4 are a significant fraction of those corresponding to nominally elastic response, this aspect of capacity design becomes less important. Although it remains important to suppress undesirable failure modes by providing increased strength in those modes, it is acceptable to locally scale actions found from an analysis for the forces specified in Part 4 by the factor $C_h(T_1,1)/C_h(T_1,\mu) = S_{m1}$ or, for subsoil types (a) & (b) where $T_1 \leq 0.45$s, by the factor $C_h(0.45,1)/C_h(0.45,\mu)$, or for subsoil type (c) where $T_1 \leq 0.6$s, by the factor $C_h(0.6,1)/C_h(0.6,\mu)$, combine the results with gravity load actions, and provide dependable strength to resist the combined actions. For details for applying this approach, refer to the relevant material standard.

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PART 3 DEAD AND LIVE LOAD PROVISIONS

C3.2 General requirements

C3.2.1
A loading notice is not required for slabs on grade within a building. For a building on piles the ground slab should be considered as suspended if the designer has assumed that the live loading from the slab is transferred to the piles.

C3.3 Dead loads

C3.3.1
BRANZ Bulletin 227: "Weights and measures of building materials" may be used to determine component and material masses.

C3.4 Live loads

C3.4.1 *General*
The floor loadings apply to all floors including those on grade.

C3.5 Balustrades and parapets

C3.5.1 *Serviceability limit state*
The deflection of handrails and balustrades may be considered acceptable if they do not exceed $h/60 + l/240$ (horizontally)

where

h = height of balustrade above support
l = span between vertical supports

C3.5.2 *Ultimate limit state*
Values of load are based on those given in BS 6399:Part 1.

C3.6 Vehicle barriers for parking areas

C3.6.1 *Ultimate limit state*

C3.6.1.2
Energy considerations can readily be developed for use in a special study. Determination of a credible vehicle velocity and maximum values of elastic or plastic deformation values for vehicle and barrier are crucial (Ref. BS 6399: Part 1).

C3.7 Machinery live loads

Machinery live loads are complex and the loadings given are the minimum to be used. In general the designer should refer to specialist standards such as the Australian Standard AS 1418,SAA Crane Code for cranes, or the Power Lift Rules (Marine Division, Ministry of Transport) for lifts. While in most cases the load effects need to be ascertained only for immediate supporting members (e.g. corbels supporting crane rails, beams supporting lift machinery), the load path needs to be pursued further in cases such as when the operating weight of machinery is the primary load on a structure. It should be appreciated however, that treating these dynamic loads as static loads will often produce deflections which are unrealistically large.

PART 4 EARTHQUAKE PROVISIONS

C4.1 Notation

The following symbols appearing in this Part of the Commentary are additional to those used in Part 4 of the Code of practice.

A	Angle of the effective earthquake force measured from the X-axis (C4.9.2.3).
c	cos(A) (C4.9.2.3).
e	Eccentricity of the effective earthquake force, measured in a perpendicular direction to the mass centroid (C4.9.2.3).
F_t	Additional equivalent static lateral force applied at the top level of a structure (C4.4.2).
I_o	Polar moment of inertia of the level under consideration about the co-ordinate origin (C4.9.2.3).
M	Translational mass (C4.9.2.3).
s	sin(A) (C4.9.2.3).
V_θ	Horizontal seismic shear force acting at the base of a structure in a direction θ (C4.8.1.1).
V_x, V_y	Horizontal seismic shear force acting in orthogonal directions x and y (C4.8.1.1).
X,Y	Coordinates of the mass centroid from the origin (C4.9.2.3).
θ	Angle of skew measured from the X-axis (C4.8.1.1)

C4.2 General requirements

C4.2.1 *Structural system*
The Applied Technology Council [4.1] investigated building performance in earthquakes and concluded that configuration, in general, and detailing, in particular, play key roles in providing reliable earthquake resistance. One of the aims in design should be to produce symmetrical structures but not without regard to the efficient operation of the buildings [4.2]. Irregular buildings, and/or buildings which contain re-entrant corners of significant dimensions, have been found to perform poorly in severe earthquakes compared with regular structures.

Evaluation of damage sustained in major earthquakes [4.3, 4.4, 4.5, 4.6] has shown that seismic performance is improved by:

(a) Reducing the eccentricity between the centres of mass and lateral force resistance of the building;

(b) Increasing the stiffness of the structure;

(c) Increasing the lateral strength of the structure;

(d) Increasing the ductile capacity of the regions where inelastic deformation may occur and ensuring that non-ductile failure mechanisms cannot develop, and

(e) Avoiding the use of long flexible diaphragms.

C4.2.2 *Limit states*
The serviceability limit state requirements are intended to protect the building from structural damage and to limit damage sustained by non-structural components in moderate earthquakes. The ultimate limit state requirements are intended to protect life and to ensure that the structure will not collapse in a major earthquake.

In many cases, especially where a large ductility factor is assumed for the ultimate limit state, the serviceability limit state requirements may set the required stiffness and strength of some members.

C4.2.3 *Ductility and structural type*
The ability of a structure to survive a major earthquake depends upon a combination of its strength, ductility and the ability to dissipate energy. Ductile structures may be provided with a lower strength than elastically responding structures, but greater attention to detailing is necessary to ensure the associated inelastic deformation and energy dissipation can be met. Elastically responding structures require higher strength, but detailing standards are less onerous to achieve. Structures of limited ductility are cases between these two forms.

C4.2.4 *Structural performance factor and structural ductility factor*

C4.2.4.1 *Choice of structural performance factor*
The uniform hazard spectra derived by the SANZ Seismic Risk Subcommittee and discussed in C4.6.2 are indicative of the likely recurrence of peak acceleration response, but not necessarily of damage and therefore of structural performance.

A single peak response of short duration will not necessarily lead to damage. It is perhaps more appropriate to consider a number of cycles of motion of sustained high response. Studies by Perez [4.7] indicate that the level of sustained shaking likely to damage structures may be as low as 60 % of the recorded peak response. For structures which are designed and detailed for inelastic behaviour, cumulative ductility demand may be given a similar interpretation in assessment of damage potential and in assessment of likely performance.

It has been found in various studies [4.8, 4.9] and in past earthquakes that buildings, on average, perform better than can be predicted by calculation using simplified analyses. The better actual performance compared with assessments made in design is due to a number of factors, including: higher material and member strengths and better member performance; greater redundancy of the structure than assumed (for example participation of non-structural members and cladding, and twisting of floor slabs in *in situ* concrete construction); beneficial effects arising from geometric changes (for example changes brought about by stiffness centroid shifts, particularly in structures with large members); increases in damping and period after the onset of damage; energy dissipation from elements not considered in the design as contributing (for example yielding of diaphragms and damping from secondary elements); and damping from radiation of energy associated with interaction of the structure with the ground.

These effects depend to a large degree on the form of the building and on the materials used in its construction. The value of the structural performance factor, S_p, given in this Standard (i.e. 0.67) is considered to be a reasonable average value taking into account all of the above effects. It should be appropriate for the majority of structures and material types in use in New Zealand.

There may, however, be structural and material types which will require special consideration. In such circumstances appropriate values of S_p will be specified in the appropriate limit state material standard.

C4.2.4.2 *Choice of structural ductility factor*
The preferred source of the structural ductility factor for a given seismic-resisting system is the relevant limit state format material standard, written specifically for compatibility with this Standard. The choice of the structural ductility factor carries with it requirements for design and detailing of the system and these requirements must be met to ensure that the anticipated level of inelastic demand can be dependably achieved.

It is recognized that, pending publication of all material standards in limit state format, guidance on structural ductility factors and material design requirements will need to be sought from existing material standards or other published sources. In this instance, table 4.2.1 gives the structural ductility factor that may be used with the most ductile form for each category. This factor may be varied by the relevant source document and must be applied in conjunction with the appropriate design and detailing requirements from that source. For example, a squat reinforced concrete structural wall is not as ductile as a slender one and consequently the structural ductility factor shown in table 4.2.1 cannot be applied to such a wall without modification.

The assessment of appropriate structural ductility factors for the design of structural forms not covered by either a limit state material standard or table 4.2.1, as appropriate, may be determined in accordance with the procedure given in Appendix C4.A.

Where lateral resistance is provided by a combination of lateral force-resisting systems, each of which has a different level of structural ductility factor, it is important to ensure the less ductile systems can sustain the displacements associated with the more ductile systems without loss of strength.

C4.2.5 *Capacity design*
In capacity design, a strength hierarchy is established, with yielding regions identified in such a manner that acceptable mechanisms may form. The structure is designed so that yielding is confined to acceptable mechanisms in the chosen regions (see also C2.5.4.6).

Ideally, capacity design should also be used for structures of limited ductility. It is recognized, however, that some structures of limited ductility have a form for which the strict application of capacity design is inappropriate. For these cases this Standard and the material standards permit alternative approaches (with the same objective as the formal capacity design approach) in which undesirable failure modes are suppressed by specified strength increase in these modes (see also C2.5.4.6).

C4.2.6 *Seismic weight*

For all uses specifically listed in tables 2.4.1 and 2.4.2, the ψ_l and ψ_u values are equal. Consequently the live load contribution to the seismic weight, which is an "arbitrary-point-in-time" value, is the same for both limit states. For roofs, the seismic weight may need to include the weight of ice (see 6.5.4 and C6.5.4).

C4.3 Methods of analysis

C4.3.1 *General*
There are limits on the applicability of the equivalent static method, but it is likely to remain the most commonly used of the three permitted methods.

C4.3.2 *Limitation on the use of the equivalent static method*
For structures where none of these criteria is met, the modal response spectrum or time history method of analysis should be used. For such structures, the higher mode actions, which are not effectively modelled in the equivalent static method, can have significant influence on the seismic response of the structure.

C4.3.3 *Use of dynamic analyses*
For horizontally irregular structures, or structures with plan or diaphragm irregularities, there can occur significant torsional response which may be inadequately modelled using a two-dimensional method of analysis where torsion is applied using a static method. For such cases a three-dimensional method of analysis should be used.

C4.4 Structural regularity
Irregularities in the lateral force resisting system have been found to increase the damage sustained in earthquakes. It is important that the designer can recognize the existence of and be aware of the effects of such irregularities. Wherever possible these irregularities should be avoided.

The assessment of irregularity must be interpreted with sound engineering judgement.

It must be recognized that the use of three-dimensional modal response spectrum or numerical integration time history methods of analysis does not reduce the need for care and engineering judgement in the assessment of the analysis results.

C4.4.1 *Horizontal regularity*
The purpose of this clause is to prevent the use of both the equivalent static method and two-dimensional dynamic methods on those structures where these methods are likely to be in appreciable error compared with more sophisticated methods. Such circumstances arise where there is significant torsional response or where changes in diaphragm stiffness affect the distribution of lateral force to resisting elements.

The criteria given in both (a)(i) and (a)(ii) are set to ensure that the response of the structure is principally one of translation without significant torsion. One only of the criteria (a)(i) or (a)(ii) needs to be satisfied.

The criterion in (a)(i) is intended to verify the applicability of a two-dimensional lateral force analysis where such an analysis has already been performed.

For the purposes of 4.4.1, the shear centre is defined as the point through which the resultant shear at any level will pass when the structure is subjected to translation only, i.e. rotation in plan is restrained.

The plan position of the shear centre may vary over the height of the structure and is a function of the structural form and the pattern of applied forces considered. The position of the shear centre at each level may be found from a two dimensional analysis of the structure using the following procedure:

1. Apply the specified pattern of lateral forces to a two dimensional model of the structure for each principal direction.

2. For each principal direction, include only those lateral force-resisting systems aligned in that direction. The lateral force-resisting systems shall be constrained to the same displacement at levels where they are interconnected by stiff diaphragms.

3. For each principal direction and each level, establish the line along which the resultant storey shear acts.

4. The intersection of the lines of resultant storey shear for each principal direction defines the shear centre at each level.

The criterion in (a)(ii) is illustrated in figure C4.4.1.

Where there are abrupt discontinuities, major variations in in-plane stiffness, or major re-entrant corners in diaphragms, the assumption of a rigid diaphragm may not be valid. In some cases investigation of these effects may require the stiffness of the diaphragm to be modelled in the analysis to ensure that a realistic distribution of lateral force has been obtained.

C4.4.2 *Vertical regularity*
The seismic design forces specified in the equivalent static method are derived from an assumed deflected shape, which is an approximation to the first mode response of a reasonably regular

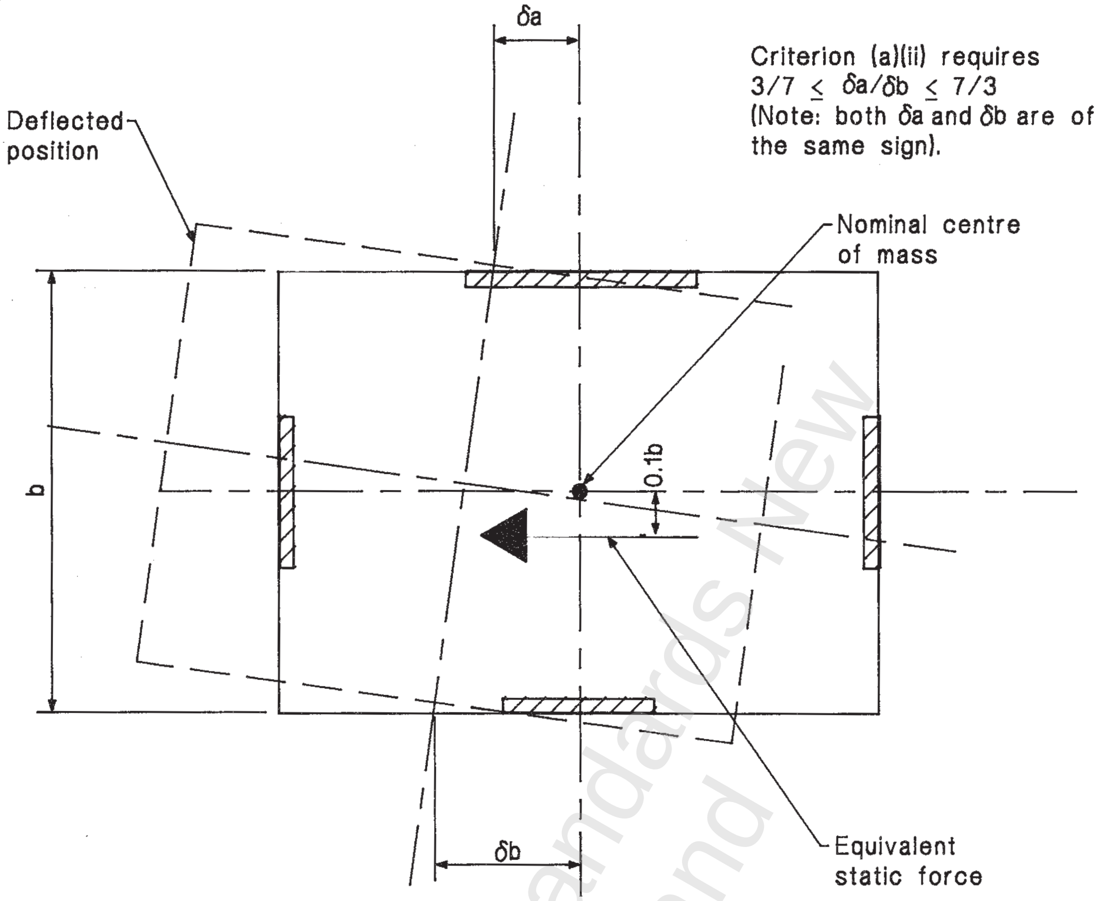


Figure C4.4.1 – Plan on structure illustrating criterion (a)(ii) of clause 4.4.1

structure. In addition a nominal allowance is made for the contribution of higher elastic mode actions. Where higher modes may make a major contribution to the response, such as may occur in structures that have a fundamental period of 2 seconds or more, or where the deflected profile is not in reasonable agreement with the assumed one (i.e., varying approximately linearly with height above the base), the equivalent static method should not be used.

The following procedure, which has been adapted from reference 4.1, may be used to check the requirement that the lateral displacement is reasonably proportional to the height above the base.

(a) Apply the equivalent static forces defined in 4.8.1 to the corresponding centres of seismic weight and determine the lateral displacement, u_i , at each level.

(b) Calculate a revised set of equivalent static forces from equation C4.4.1.

$$F_i = F_t + 0.92\,V \frac{W_i u_i}{\Sigma(W_i u_i)} \quad \text{.......... (Eq. C4.4.1)}$$

where F_t = 0.08 V at the top level and zero at all other levels

(c) Calculate the storey shears corresponding to the revised set of equivalent static forces. If these values lie within the margin of ±20 % of the storey shears corresponding to the equivalent static forces (applied in step (a)), the criterion is satisfied.

For structures which satisfy horizontal and vertical regularity requirements and have a fundamental period of 2 seconds or less, the set of equivalent static forces given by equation C4.4.1 provides a better match to the dynamic characteristics of the structure than the values defined in 4.8.1, and they may be used in the design of the structure if desired.

C4.5 Period of vibration

C4.5.1

Tests on full-size buildings show that it is difficult to accurately calculate their dynamic characteristics [4.10]. The proper choice of material and member properties for the calculation of stiffness is important. Guidance in this choice should be sought in the material standards.

In calculating the periods of vibration, the influence of the flexibility of the supporting soils should be considered. Ignoring foundation flexibility will generallly lead to a conservative assessment of the seismic forces, but it is likely to result in a low estimate of the seismic deformations. Particular caution is required where different foundation types or supporting soils occur in the same building. In such cases the differing foundation stiffnesses have a direct influence on the distribution of seismic forces in the structure.

For common ductile structures, the same properties are appropriate for both the ultimate and serviceability limit states. In some structures, particularly where reinforced concrete or timber is used and the structural ductility factor for the ultimate limit state is large, the use of larger section or material parameters may be justified for the serviceability limit state than for the ultimate limit state, to reflect the lower level of cracking or strain levels.

C4.6 Seismic design actions

C4.6.1 *General*

C4.6.2 *Design spectra and lateral force coefficients*

The design coefficients and design spectra in this Standard are based on 5 % damped uniform hazard spectra which are modified to give values which are appropriate for design.

For the ultimate limit state, uniform hazard spectra with an assessed return period of 450 years (approximately 10 % probability of exceedance in 50 years) have been adopted. The design forces for the serviceability limit state are based on uniform hazard spectra with an annual probability of exceedance of about 10 % (a return period of about 10 years).

The uniform hazard spectra adopted are based on those derived by the SANZ Seismic Risk Sub-committee using modified Katayama attenuation relationships [4.11].

The SANZ Seismic Risk Sub-committee established that, for New Zealand, the normalised shapes of the uniform hazard spectra are largely independent of geographical location (seismic zone) and of return period. This permits the use of the simplifying assumption employed in this Standard that the hazard spectrum ordinates are linear in Z, R and L_s or L_u and therefore can be defined by the product of these factors and a basic seismic hazard acceleration coefficient, $C_h(T,\mu)$.

For intermediate subsoil sites (site subsoil classification (b)) the basic seismic hazard acceleration coefficients for elastic response, $C_h(T,1)$, are the 5 % damped normalised hazard spectral values derived by the SANZ Seismic Risk Sub-Committee.

For structural periods greater than 1.0 s the normalised spectral ordinates have been taken as inversely proportional to period, as recommended by the SANZ Seismic Risk Sub-committee. This corresponds to an assumption of constant spectral velocity and leads to less rapid reduction of the spectral ordinates with increasing structural period than given by the uniform hazard results. This adjustment was adopted to reflect a number of concerns, including:

(a) Possible underestimation of the long-period ordinates due to scarcity of strong-motion recordings of large earthquakes at short epicentral distances; and

(b) The need to provide greater weighting to higher long-period ordinates of large magnitude earthquakes which, because of their longer duration, have a greater damage potential than events of moderate magnitude.

The basic seismic hazard acceleration coefficients for rock or very stiff soil sites (subsoil category (a)) have been derived by scaling the values for intermediate soil sites by the ratio of Katayama's [4.12] Class I and Class III ground condition factors, and then increasing these values by 50 % (approximately). The 50 % increase is considered reasonable as it is felt that there is insufficient data from New Zealand earthquakes at rock sites to justify full reduction.

The basic seismic hazard acceleration coefficients for flexible or deep soil sites (subsoil category (c)) have been derived by scaling the values for intermediate soil sites by approximately 1.25 times the ratio of Katayama's [4.12] Class IV and Class III ground condition factors. The Katayama Class IV is for sites with natural periods exceeding 0.6 s. The 25 % increase has been applied because, although Katayama Class IV includes sites with "soft" materials, the Katayama ratio is thought to be unconservative for sites with such material.

The basic inelastic seismic hazard acceleration coefficients for the equivalent static method and for the calculation of lateral forces on parts (4.12), have been derived from the elastic coefficients as follows:

(a) For $T_1 > 0.7$ s, the equal displacement principle is applied:

$$C_h(T_1, \mu) = \frac{C_h(T_1, 1)}{\mu} \qquad \text{(Eq. C4.6.1)}$$

(b) For $0.45 \text{ s} \leq T_1 \leq 0.7$ s for site subsoil categories (a) and (b) and for $0.6 \text{ s} \leq T_1 < 0.7$ s for site subsoil category (c)

$$C_h(T_1, \mu) = \frac{C_h(T_1, 1)}{\left[\dfrac{(\mu - 1)\,T_1}{0.7} + 1\right]} \qquad \text{(Eq. C4.6.2)}$$

(c) For $T_1 < 0.45$ s for site subsoil categories (a) and (b).

$$C_h(T_1, \mu) = \frac{C_h(0.45, 1)}{\left[\dfrac{(\mu - 1)\,0.45}{0.7} + 1\right]} \qquad \text{(Eq. C4.6.3)}$$

(d) For $T_1 < 0.6$ s for site subsoil category (c).

$$C_h(T_1, \mu) = \frac{C_h(0.6, 1)}{\left[\dfrac{(\mu - 1)\,0.6}{0.7} + 1\right]} \qquad \text{(Eq. C4.6.4)}$$

The denominator on the right hand side of equation C4.6.2 is as proposed by Berrill et al [4.13] and allows a gradual variation from equal displacement theory for periods greater than 0.7 s to equal energy theory at a period of approximately 0.35 s. The validity of the approximation has been checked by evaluating the response of single degree of freedom models with varying levels of ductility using numerical integration time history analyses [4.14].

Equations C4.6.1 to C4.6.4 also form the basis for the S_{m1} factors used to scale the elastic design spectra for the modal response spectrum method.

The zone factor, Z, is an indicator of relative hazard. In this Standard the values of Z are equivalent to the 450 year return period, elastic 5 % damped uniform hazard contours for a structural period of 0.2 s calculated by the SANZ Seismic Risk Sub-committee. However, the value of the zone factor has been limited to the range 0.6 (applying near Dunedin and north of Hamilton) to 1.2 (applying at Whakatane). About 40 % of the value of Z gives a fairly close estimate of the 450 year return period peak ground acceleration (in terms of g).

For important structures and structures of a secondary nature, the probability of exceedance of the hazard spectra is varied using a risk factor, R. The relationship between R and the return period, or probability of exceedance as derived from the results of SANZ Seismic Risk Subcommittee, is given in figure C4.6.1.

The hazard spectra for the serviceability limit state have been estimated by extrapolation of the results of the SANZ Seismic Risk Sub-committee. The ratio between the serviceability and ultimate limit state hazard spectra is approximately one-sixth, i.e. $L_s = {}^1/_6$.

The majority of New Zealand buildings have a fundamental period less than 0.45 s. For many of these structures particularly in domestic construction, the dynamic characteristics, which depend on soil response and soil-structure interaction, cannot be readily established. Experience also indicates that these structures tend to have sources of both strength and deformation, not readily quantified in a simplified analysis, which increase their ability to survive major earthquakes.

Furthermore, the spectral displacements at short periods are rather small (in the order of 10 mm for a period of 0.2 s). The effects of such small displacements are such that collapse is unlikely in the region of maximum acceleration response for normally proportioned members, so reduced strength is admissible. For these reasons, the spectra for rock and intermediate soils have been truncated at a period of 0.45 s.

No reduction in design load for the ultimate limit state is permitted for structures with very short initial periods designed using either the equivalent static or modal response spectrum methods. This reflects the possible reduction in stiffness and lengthening of the period of such structures as they yield, leading to a higher response.

The soil descriptions and the associated properties used for the site categories correspond to those given in reference 4.15.

In general, the classification of the site will be dependent on the surface soils even where vertical piles or piers extend down to a harder underlying stratum. However, with raking piles or with stubby vertical piles or piers, the possible adverse effects of the stiffer foundations should be considered.

The depths of materials listed for category (c) sites are likely to produce periods of about 0.6 s. Where a site consists of layers of several types of material, the contribution of each layer to the natural period may be estimated by multiplying 0.6 s by the ratio of its thickness to that listed for its soil type. The total period may then be estimated by summing the contribution for each layer.

None of the hazard spectra for site subsoil categories (a), (b) or (c), satisfactorily account for resonant subsoil response. Such response has been a factor in causing severe damage in a number of earthquakes around the world. It occurs at sites where there are sharp impedance contrasts between the near-surface soil layers and underlying material. A further factor that appears to be associated with resonant response is the presence of cohesive soil with a high plasticity index. This situation is likely to occur where recent marine, lake or swamp deposits overlie coarser gravel or rock. These sites may occur in coastal towns and cities and on floodplains of rivers. They should be treated with caution, and the designer should be aware that earthquake motions at these sites for some spectral periods could far exceed those implied by the spectra in this Standard. Similarly, motions in the vicinity of surface fault rupture could considerably exceed those provided for in this Standard.

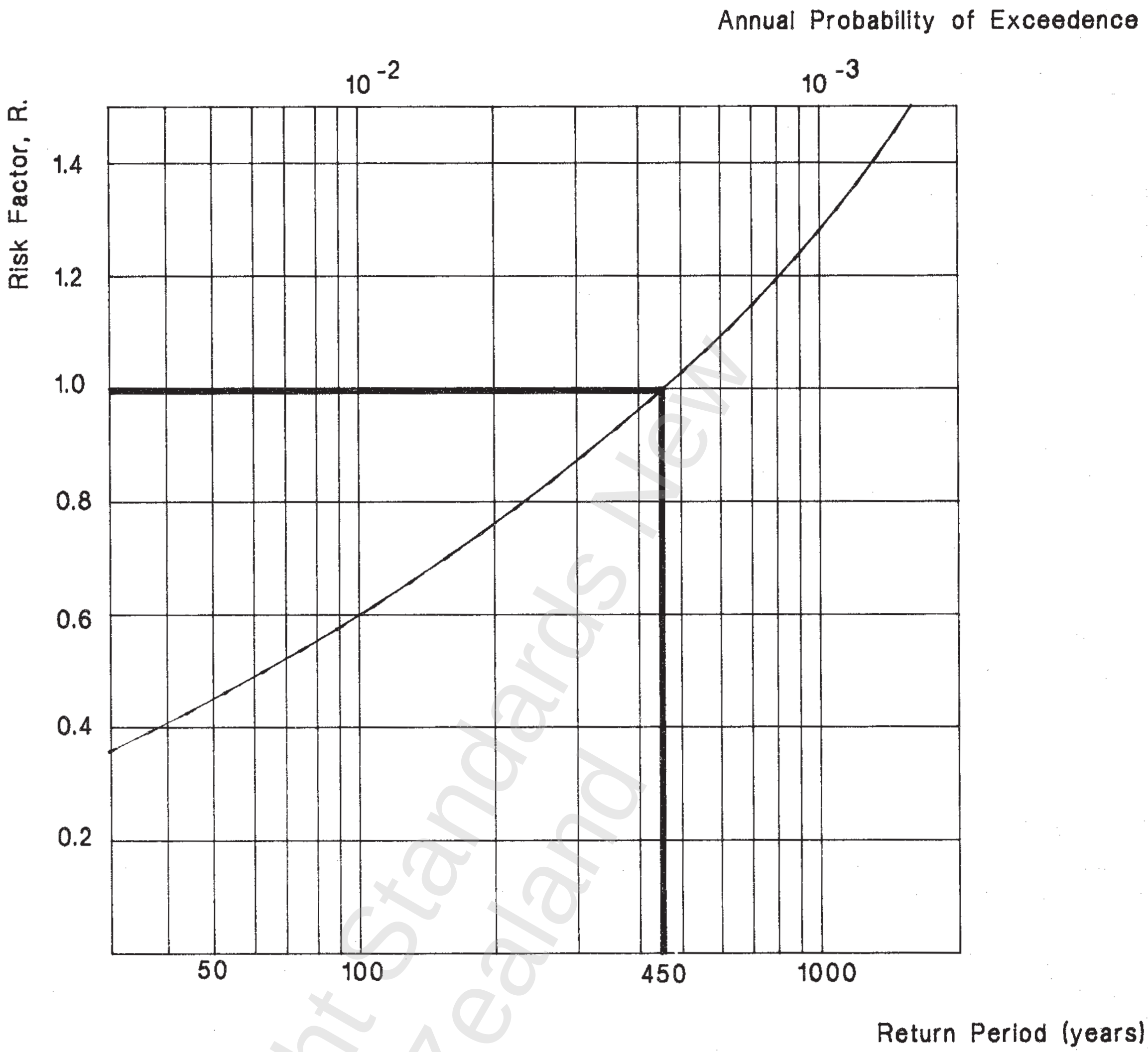


Figure C4.6.1 – Relationship between Risk Factor, R, and return period or annual probability of exceedence

C4.6.2.7, C4.6.2.8 and C4.6.2.9

For the serviceability limit state, the seismic actions are found from the scaled elastic design spectrum or scaled elastic seismic acceleration coefficients. A scaled elastic design spectrum is also used for the ultimate limit state where the modal response spectrum or numerical integration time history methods are used.

For the equivalent static method, actions and displacements for the ultimate limit state may be obtained by scaling actions and displacements for the serviceability limit state.

It should be noted that for a fundamental period greater than 0.7 s and a structural ductility factor equal to 6, the scale factor is generally equal to unity. This illustrates that, provided P-delta actions are small, the serviceability limit state actions may control minimum strength levels.

Modal response spectrum method

For the modal response spectrum method, the design spectra for both limit states are of similar shape, provided the same member properties are used for both limit states. This enables the seismic effects for the serviceability limit state to be scaled to give the design lateral force effects for the ultimate limit state. The P-delta effects may need to be added to these to give the design values.

The value of the design spectrum scale factor, S_{m1} is based on equations C4.6.1 to C4.6.4. However, the need to modify the results for minimum levels of base shear may produce a scale factor which is different from this value.

Modal analysis, being a procedure assuming elastic behaviour, uses the elastic spectrum. The results are then scaled to accord with the inelastic spectral acceleration at the first mode. This is the intended role of S_{m1}.

The use of the modal response spectrum method usually gives smaller design base shears and overturning moments than the equivalent static method. For regular structures, extensive experience with the equivalent static method shows that method to be satisfactory for these structures. The scale factor, S_{m2}, is to ensure that the results of the modal response spectrum analysis are not excessively smaller than the results of an equivalent static method. Consequently for regular or elastically responding structures, scaling up of all analytical results will be required when the combined modal base shear is less than 80 % of the value from an equivalent static analysis.

As irregular structures have been found to perform poorly in earthquakes compared to regular structures, a possible reduction in strength due to use of modal method is not appropriate. Accordingly K_m has been set equal to 1.0 for these structures.

Under no circumstances for either regular or irregular buildings may the base shear and associated seismic actions found from a modal response spectrum analysis be scaled down to the limits given in 4.6.2.8(b)(ii).

Designers should be careful to select an appropriate value of T_1 in these scaling procedures. The largest period may be for a torsional mode, whereas modes associated with translation are intended. T_1 should correspond to a mode with significant effective mass in the direction under consideration. Where there is doubt due to complex mode shapes and modal participation, T_1 should be found as for the equivalent static method as provided in 4.5.2.

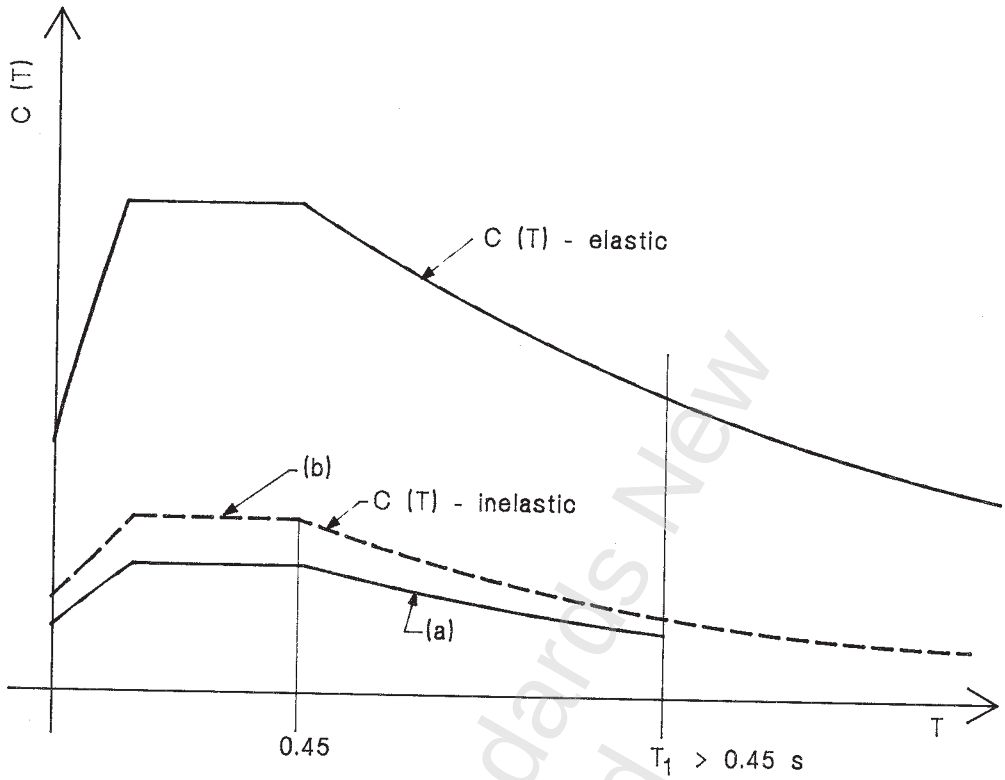
While it would suffice to scale the results of modal combinations, such scaling would lead to ambiguity in the contribution to be assumed from each mode. It is intended that scaling be applied to each mode (and therefore to the combined results). This is equivalent to producing a modified spectrum which is a scaled version of the elastic spectrum. Typical spectra resulting from such scaling are illustrated in figure C4.6.2.

Numerical integration time history method

The design spectra for the serviceability limit state for the numerical integration time history method are the same as for the modal response spectrum method. Comparative analyses indicate good agreement between the levels of strength and deformation predicted by both methods using the same spectra, although much depends on the method adopted for scaling the earthquake records (refer also C4.10.2).

For the ultimate limit state the design spectrum is the hazard spectrum (i.e. S_p is taken as equal to 1.0). For the equivalent static and modal response spectrum methods, the hazard spectra are scaled by S_p to reflect several effects including a sustained number of damaging cycles rather than a single peak. This may not be appropriate for assessment of ductility demands using the numerical time history method where the earthquake motions are scaled so that the peak response over the period of interest matches the design spectrum.

The value of S_p has been set conservatively at 1.0 for the ultimate limit state until further study or experience indicates a lesser value is appropriate.



(a) T_1 > 0.45 s. S_m depends on T_1 and μ unless minimum base shear controls

(b) $T_1 \leq$ 0.45 s. S_m depends on μ only unless minimum base shear controls

For (a) and (b), ordinates are S_m times ordinates of elastic spectrum.

Figure C4.6.2 – Derivation of the inelastic design spectrum for use in the modal response spectrum analysis – ultimate limit state (intermediate soil site assumed)

C4.6.3 *Direction of forces and accidental eccentricity*

C4.6.3.1
The intention is that designers are required to identify the axis of loading which produces the most serious condition in any major sub-assemblage, such as a frame or wall. For buildings in which the seismic resisting elements are oriented in two perpendicular directions, application of the specified forces (or inertial effects) in a direction other than those parallel to the framing lines may produce the most adverse effects in some cases, but it will usually be sufficient to apply the forces along each of the orthogonal axes.

The requirement to consider the direction of the horizontal forces should not be confused with the concurrency requirement which is concerned with capacity design (see C2.5.4.6(b)).

C4.6.3.2
The provisions for eccentricities for arbitrary directions of loading remove otherwise rather abrupt steps. Eccentricity is intended to allow for several effects (see C4.8.2).

Where a diaphragm or other means of distribution of seismic loads is not present or not continuous, the allowance for an accidental eccentricity of the applied inertial forces or of the mass as a proportion of the total width of the building may not be appropriate. For such cases the structure could be divided into several substructures which would then be considered separately for the purposes of applying 4.6.3.2. The value of b would then be the width of the tributary area affecting each substructure.

C4.7 Seismic deflections and P-delta effects

C4.7.1 *General*

C4.7.1.1
Refer to commentary for Part 2 for the criteria to be met and limits on seismic deflections.

C4.7.1.2
Consistent sets of material and section properties should be used when determining the period and deformation of the structure for each limit state.

C4.7.1.3
Refer to C4.8.1.5.

C4.7.2 *Serviceability limit state deflections*
Additional deflection is associated with the redistribution of flexural actions. Where this is assumed to occur in the serviceability limit state, a shake down analysis should be carried out as required in 2.5.2.2 (C2.5.2.2) to determine the additional deflection. Designers should also be alert to the occasional need to include additional deflections due to sidesway caused by gravity loads.

C4.7.3 *Ultimate limit state lateral deflections*

C4.7.3.1
The clause requires the designer to consider the lateral deflection envelope created by combining the results from the elastic analysis scaled up by μ and the estimates of the deflection profiles resulting from any possible (including beam) mechanisms.

Where storey mechanisms can form, deflections at levels above this storey increase markedly. These may lead to unacceptable damage to the structure or its parts and will greatly increase vulnerability to P-delta effects. Construction of sidesway deflection profiles is illustrated in figure C4.7.1.

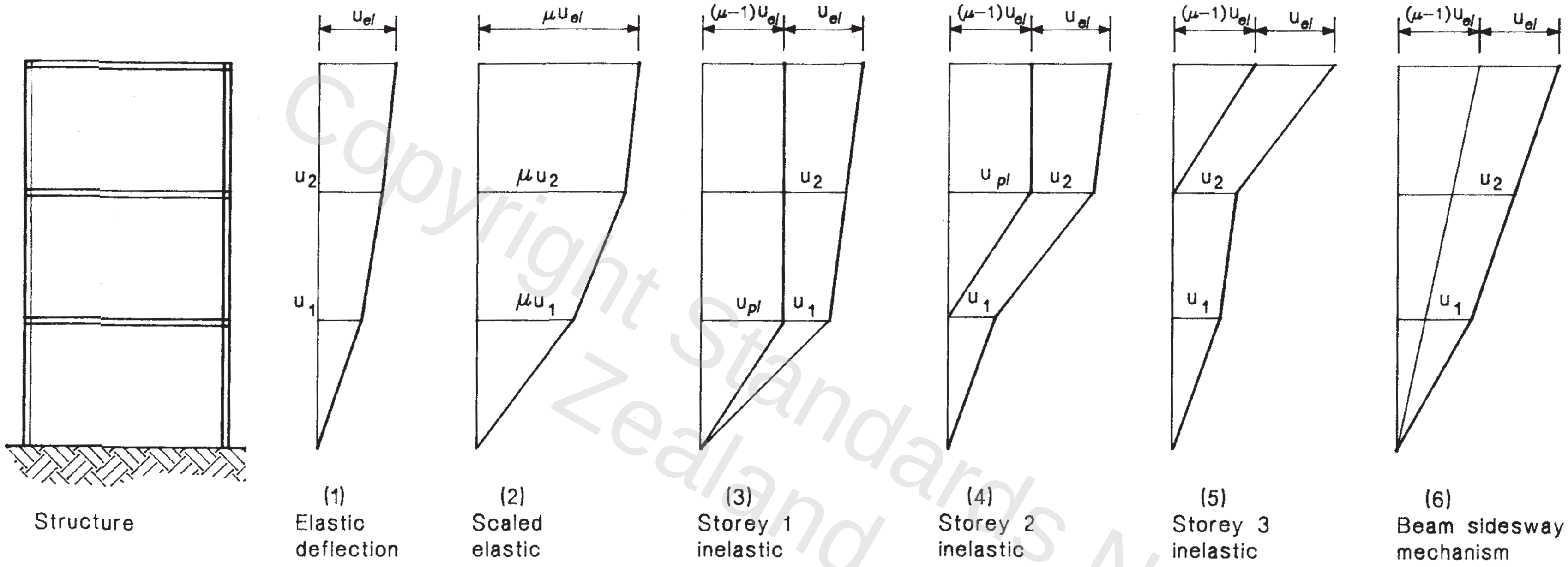
It should be noted that only sidesway mechanisms which are not specifically suppressed through the application of capacity design procedures need be considered in 4.7.3.1(b).

The designer should be aware that recent research [4.16, 4.17, 4.18] indicates that the procedure outlined above may under-estimate the lateral deflections in lower levels of ductile tall multistorey frames as predicted using the numerical time history method given in 4.7.3.2.

It is considered that in the majority of cases the deflection limits in Part 2 of the Standard will prevent significant departures from the specified deflection profiles.

C4.7.4 *Ultimate limit state interstorey deflections*
A series of analyses of frame structures has shown that taking the interstorey deflections as the difference in the combined modal deflections of adjacent levels under-estimates the maximum interstorey deflection in most storeys of regular buildings by a few percent. However, the under-estimate may be as large as 40 % in the upper storeys of these buildings.

As discussed in C4.7.3.1 the equivalent static or modal response spectrum methods may under-estimate the interstorey deflection in the lower storeys of ductile moment resisting frames. This arises as the elastic based methods of analysis do not necessarily allow for the ‘higher mode effects’ which can be associated with the formation of plastic hinges in the structure.



For $\mu \geq 2$

Level	Control profile for lateral deflection	Storey	Control profile for interstorey deflections
3	All	2-3	(5)
2	(4)	1-2	(4)
1	(3)	0-1	(3)

NOTE – Assumes that sidesway mechanisms are not suppressed by application of capacity design procedures

Figure C4.7.1 – Construction of deflection profiles for use with 4.7.3.1(b)

C4.7.5 *P-delta effects – Ultimate limit state*
This clause refers to P-delta effects induced in a building as a whole. An investigation for the effects is required only in the ultimate limit state.

C4.7.5.1
An analysis for P-delta effects is not required in most low rise buildings. The criteria (a) to (d) are intended in general to require an assessment of P-delta effects to be carried out in structures where 25 % or more of the lateral strength of the structure is required to resist P-delta effects. This criterion is based on analyses for earthquake records with a duration of intense ground shaking of the order of 15 to 25 seconds.

Research has shown that P-delta effects increase with:

(a) The flexibility of the structure;

(b) The structural ductility factor and

(c) The duration of intense shaking of the earthquake ground motion.

P-delta effects have been found to be small in buildings which respond elastically or with a small ductility demand [4.19, 4.20, 4.21, 4.22].

For frame structures, if the critical ratio of interstorey deflection to interstorey height occurs in the first storey, equation 4.7.1 simplifies to:

$$\gamma_i \leq \frac{C}{7.5}$$

where C is the lateral force coefficient given in 4.6.2.7(b).

One way of overcoming P-delta actions is to design for a reduced level of ductility demand, i.e. a lower structural ductility factor. This has the effect of increasing the value of V_i and hence increasing the allowable deflection.

C4.7.5.2
A method of assessing P-delta effects is outlined in Appendix C4.B to this Commentary. The approach was developed from references 4.20, 4.21 and 4.22. Additional background to the method is given in reference 4.23.

Caution should be exercised in basing analyses for P-delta actions on methods described in other codes of practice. The methods contained in some of these (see references 4.1, 4.24 and 4.25) do not allow for the influence of ductility on the response, and as a result the actions can be greatly under-estimated. For certain structures these codes of practice indirectly compensate for this by specifying higher seismic design forces than those specified in this Standard.

C4.8 Equivalent static method

C4.8.1 *Equivalent static forces*

C4.8.1.1
Earthquake forces skew to two orthogonal principle axes may be assessed using
$V_\theta = V_x \cos^2 \theta + V_y \sin^2 \theta$

where θ is the angle of skew measured from the X-axis
V_x, V_y are the base shears in two principle directions x and y.

C4.8.1.2
The mass of the structure at or below the level where the ground provides effective horizontal restraint (the base of the structure) is assumed not to contribute to the horizontal seismic shear force at the base of the structure nor at any levels above the base. However, all parts at and below the base should be designed to resist the inertial forces due to their masses and the ground acceleration (see C4.12), and the reactions from levels above the base.

C4.8.1.4
The lateral force of 0.08 V, which is applied to the top of the structure, is to compensate for the effects of the higher modes in the upper few storeys. It has been shown that ideally the magnitude of this force should increase with the fundamental period [4.1, 4.25]. However, keeping this at a constant value of 0.08 V reduces the complexity and gives sufficiently accurate values for the period range up to 2 seconds [4.16].

Where the top storey of a building is of light weight construction compared with the floor below this level, or where a penthouse is used, the 0.08 V lateral force may be distributed between these upper levels. Alternatively the lightweight storey may be treated as a part (see C4.12).

C4.8.1.5
With regular multi-storey structures, the equivalent static method overestimates the deflections compared with the modal response spectrum method. In analyses of such structures with the modal method, it is found that 95 % or more of the combined deflection envelope is contributed by the first mode values. The proportion of mass that contributes to the first mode is close to 0.75. With the equivalent static method, 100 % of the mass is assumed to act in a manner similar to the first mode. A consequence of this is that the deflections are about 1.33 times the values predicted by the modal method [4.16, 4.17].

The 0.85 scale factor is generally conservative where used for predicting interstorey deflections in regular frame structures especially when predicting overall deflections in both frame and wall structures (on average 0.75 is closer).

For a single storey structure, the equivalent static and modal analyses are identical. Consequently for structures with a few floors only, the scale factor needs to be interpolated between 0.85 and 1.0.

The scaling of deflections by a factor which depends on the method of analysis that is used is not new to this Standard. It was practiced in the previous loadings standard [4.26] (see NZS 4203:1984).

C4.8.2 *Points of application of equivalent static forces*

C4.8.2.1
The 0.1 b eccentricity is to allow for variations in the structural properties, the distribution of mass, participation of cladding and partitions in lateral resistance, and the effect of rotation of the ground about a vertical axis.

For an arbitrary direction of loading, the force may be assumed to lie on an ellipse with semi-major axes equal to 0.1 b calculated for each orthogonal axis (see also 4.6.3.2 and C4.6.3.2).

C4.9 Modal response spectrum method

C4.9.1 *General*

C4.9.1.1
It is not considered necessary to analyse the structure for the effects of both horizontal translational components of the ground motion acting simultaneously.

C4.9.1.2
Sufficient modes are to be considered so that the summation of effective mass over all modes considered is at least 90 % of the total mass. While this results in structural actions and displacements which are slightly less than would be obtained if all modes were included, for practical purposes the difference is negligible. Designers should consult standard texts on structural dynamics (e.g. [4.27]) for calculation of effective mass.

C4.9.2 *Torsion*

C4.9.2.2 *Static analysis for torsional effects*
For some regular structures, where the nominal centres of mass and stiffness are close, it is possible to conduct 2D (two-dimensional) modal response spectrum analyses. In this case there are possible alternatives for allowing for torsional effects, two of which are;

(a) The mass may be increased at each level, or

(b) A separate static analysis may be made for the torsional actions with the resultant structural actions being added directly to the corresponding combined modal values.

The procedure (b) is covered by this clause.

C4.9.2.3 *Three-dimensional analysis*
For a three-dimensional modal response spectrum analysis, larger actions may occur near the centre of the building when the centre of mass is not moved, and this needs to be considered.

(a) It should be noted that the shift of the centre of mass is intended to account for variation in a number of parameters, not just variations in mass distribution (see C4.8.2.1). Nothing is specified about the relative shift of the individual components of the mass (i.e. of the dead and live load contributions), and this should not be assumed by the designer. If the live load only, for example, is assumed shifted then the 0.1 b shift of the total mass may not be physically plausible (it may imply a shift of the total live load outside the confines of the building).

To modify the rotational inertia requires a further assumption about the distribution of mass for each level. If a rigid floor diaphragm is not assumed, the designer will need to make some assumptions about the mass distribution, and, for most cases, a variation assumed linear with the perpendicular distance from the axis of the earthquake would be adequate.

(b) Where the diaphragm may be assumed to be rigid, the refinement of adjusting the rotational inertia appears in most cases to be unnecessary. The procedure in (i) permits this assumption to be made. However, it should nevertheless be recognized that any shift of the mass centroid implies a change to the dynamic characteristics of the building, even when the moment of inertia about the mass centroid is adjusted to leave the moment of inertia about the origin of co-ordinates unchanged. This is because the off-diagonal terms of the mass matrix are affected by the shift (the translational forces applied at the centre of mass contribute to the torsional moments about the origin). Therefore it is necessary to find a new solution for the periods and mode shapes, and the concessions made in the permitted procedure are seen to be slight. The procedure is included because of its long standing use in existing commercially available software.

The procedure described in (ii) is much simpler to apply in practice, and, recognizing that the effort required to solve for the periods and mode shapes represents the majority of the total effort, requires much less calculation than the alternative procedure in (i). In effect the procedure allows the dynamic characteristics of the system to remain fixed, whatever the orientation of the effective earthquake forces, and whatever their eccentricity from the mass centroid. The only variable is the earthquake influence vector (see for example reference [4.27]) which now contains terms typified by the following for each level, where the terms are ordered the same as for the 3 degrees of freedom at each level (u in the direction x, v in the direction of y, and β rotation about the vertical axis z):

$$\left|\begin{array}{c} \ldots \\ c \\ s \\ M(X.s - Y.c - e)/I_o \\ \ldots \end{array}\right|$$

where

c	=	cos(A)
s	=	sin(A)
M	=	translational mass
I_o	=	polar moment of inertia of the level under consideration about the co-ordinate origin
X,Y	=	the co-ordinates of the mass centroid from the origin
e	=	eccentricity of the effective earthquake force, measured in a perpendicular direction to the mass centroid
A	=	angle of the effective earthquake force measured from the X-axis.

The designer needs to include sufficient modes to ensure that at least 90 % of the total moment of inertia about the vertical axis is participating for each direction analysed.

C4.9.3 *Combination of modal effects*

When two or more modes which contribute to effects in the design direction have frequencies close together, the response of the structure to their combined effects may not be adequately calculated using the square root of the sum of the squares (SRSS) method. Although it is mandatory to take the closeness of the modes into account only when the ratio of their frequencies lies in the range specified in this clause, it is recommended that the more sophisticated methods be routinely used. The recognized and suggested methods given in references 4.28 and 4.29 degenerate to the SRSS results when the modes are not close. The 15 % tolerance is based on an assumed acceptable error of 10 % relative to the results of the CQC method when the SRSS method is used and the damping ratio is taken as constant at 0.05 in all modes.

It is acceptable to combine the closely spaced modes by summation of absolute values, and combine the results with other modes using SRSS. Recommended methods are those in references 4.28 and 4.29.

C4.9.4 *Design actions*

The actions in each mode are in equilibrium. However, combining the modal quantities, gives envelopes of design actions each of which may occur at different times during the passage of the design earthquake. The consequences of having design earthquake envelope values rather than a set of simultaneously occurring actions are:

(a) The envelope values are not in equilibrium with each other;

(b) An envelope of values cannot be manipulated in the same way as simultaneously occurring actions. For example integrating the storey shear forces gives spurious values for the storey bending moments. It is not possible to determine a simple set of static forces which satisfies all the combined quantities [4.30] (for instance deflection with storey shear and overturning moment, or bending moments with shears and axial forces).

Once the earthquake effects have been obtained from the modal response spectrum method, it remains to combine these effects into the load combinations specified in Part 2.

The combined modal values are not suitable for use in "capacity design" as equilibrium is not satisfied. Capacity design procedures for reinforced concrete were established on the basis of scaling the results of an equivalent static analysis. As these values correspond closely to first mode behaviour the equivalent static results can be replaced by first mode actions with no loss of accuracy [4.30]. The dynamic magnification factor, ω, is applied to the equivalent static or first mode values where appropriate to allow for higher mode effects. It should be noted that these "higher modes" include elastic higher modes as well as complex modes which arise as a result of plastic hinges forming in certain regions of the structure. Higher mode effects of the latter type will not be observed in any elastic analysis.

C4.10 Numerical integration time history method

C4.10.1 *General*

C4.10.1.1
In general, it would be necessary to conduct an analysis for each of the selected earthquake records for each of the conditions investigated. This represents a very large computational task, and it would be more usual to use this method for a few of the conditions only, relying on simpler methods, such as the modal response spectrum method, for the remainder.

C4.10.1.2
The under-estimation of structural stiffness may lead to unsafe results. Members should be assigned a strength appropriate to the aspect of design being examined, and the effects of strain hardening and possible stiffness degradation should be included where appropriate.

Damping values should be realistic. Large damping values may lead to spurious effects. Suggested damping values are given in reference 4.31.

C4.10.1.3
At least two of the three earthquake records should be of a form which could be reasonably expected at the site. The designer will need to include simultaneous vertical excitation where required to do so by the material standard or where it may be expected to have a significant effect on the structure or structural element.

C4.10.2 *Scaling of input earthquake records*
The validity of the results of a time history analysis will be very dependent on the characteristics of the earthquake strong motion records used. Scaling is required to ensure that over the period range of interest the severity of the records chosen matches the design requirements specified in section 4.6 as closely as possible. It is recognized that it may prove impractical to precisely match the response spectra of individual events to the uniform risk design spectra. For this reason it may be necessary, in some cases, to use more than three records to provide confidence that a representative result has been obtained.

C4.10.3 *Length of input earthquake records for ultimate limit state*
The length of the record included in the analysis is particularly important in studying the effects of P-delta in inelastic structures. The length of record, specified as a multiple of the fundamental period, also ensures that there are sufficient reversals to test cyclic response of the structure. The length of record is considered of less importance for the serviceability limit state and in cases where a structure remains essentially elastic during the ultimate limit state.

C4.10.4 *P-delta effects*
It is recommended that direct modelling for P-delta effects be included in the dynamic analysis. Where this is not feasible, the additional strength to satisfy P-delta requirements should be assessed, for example by the method of Appendix C4.B, and it may then be necessary to conduct analyses which variously include and exclude the enhanced strength so that ranges of inelastic deformation and overstrength actions are properly appraised.

C4.10.5 *Design using numerical integration time history method*

C4.10.5.1
While the strength requirements of yielding members will generally be found using simpler methods, the approach outlined could be appropriate when, for example, damping in the structure at yield varies from the 5 % value used as the basis for the derivation of the design spectrum. S_p is included for similar reasons to those outlined for the serviceability limit state in C4.6.2.9.

C4.10.5.2
When assessing inelastic demands and capacity actions, the properties of the yielding members should be based on probable (i.e. mean) strengths and the analysis should include the effects of strain hardening where appropriate.

C4.11 Foundations and soil retaining structures

C4.11.1 *Foundations*

C4.11.1.1
The provisions of Part 2 are intended to be quite general. Specific clauses relating to foundation design are: 2.5.1.4, and 2.5.4.4. Refer to the corresponding commentary of Part 2.

C4.11.1.2 *Rocking*
These provisions do not apply to rocking or uplift of individual footings covered by 2.5.4.4 unless dissipation of energy is primarily in that mode. As a general guide, a regular frame of three or more bays in which uplift of an exterior column footing occurs need not be made the subject of a special study referred to in this clause (see also C2.5.4.4).

Where the provisions do apply, careful modelling of foundation stiffness and a reliable means of simulating inelastic dynamic response will be necessary. The analysis will need to take into account the interchange between potential and kinetic energy and associated energy losses on rocking. Rocking as a means of dissipating seismic energy is thought to be appropriate only where the superstructure is stiff and remains essentially elastic.

C4.11.2 *Soil retaining structures*

C4.11.2.1
All retaining structures designed for loads other than earthquake will possess some resistance to earthquake. For the excepted structures, it is implied that this inherent resistance is acceptably large and specific design, requiring more complex calculations and yet with considerable uncertainty, is not warranted.

C4.11.2.2
The provisions of section 6.6 are intended to be quite general. They should be met, with the additional requirements of this clause also applying. It is to be appreciated that compliance with the provisions of 6.6.4 concerning embankment stability, for banks composed of cohesionless material, will be difficult unless the surface slope is small.

C4.11.2.3
The 0.25 factor approximates $C_h(0,1)$ in figure 4.6.1, corresponding to elastically responding structures of zero period, multiplied by a structural performance factor of 0.67. R and Z are the risk and zone factors. The product 0.25 ZRL_s or 0.25 ZRL_u may be considered to be the effective ground acceleration divided by the acceleration of gravity. The distribution and magnitude of the pressures are dependent on the soil type and the deformation of the soil retaining structure. For the common case of a rigid basement wall supporting a cohesionless backfill with level surface at the top of the wall, the pressure may be assumed to vary linearly from $(K_o+C/2)wh$ at the bottom of the wall to 1.5 Cwh at the top, where K_o is the earth pressure coefficient "at rest", C is the seismic coefficient (0.25 ZRL_s or 0.25 ZRL_u), w is the weight of a unit volume of soil, and h is the height of the wall. Further reference may be found in reference 4.13.

C4.11.2.4
As the seismic coefficient increases it becomes more difficult to economically resist lateral pressures arising from earthquake motions with an isolated soil retaining structure, unless cohesion is significant. The technique of using displacement to limit forces is described in reference 4.32, and is tested and discussed in reference 4.33. Further reference may be found in reference 4.13. In general, sliding is the preferred mode of deflection control, but in assessing the upper limit of resistance, allowance should be made for the likely changes in soil properties with time due to changes in moisture content, and consolidation arising from construction. Structural yield will not often be viable because the resulting deformations may exhaust the usable structural strains or lead to buckling where gravity loads are supported.

C4.12 Requirements for parts

C4.12 1 *General*

C4.12.1.1
Non-structural components comprise architectural systems and components and include such items as non-load-bearing walls and partitions, ceilings, veneers, canopies, roofing units, windows, and wall attachments.

Permanent services equipment comprise all mechanical, plumbing, and electrical services and machinery within the buildings. The seismic forces prescribed for permanent services equipment in this Standard are applicable only to the design of the supports and attachments to the equipment. The designer should consult NZS 4219 for the design of the equipment itself, but, pending the revision of NZS 4219, it is recommended that the equipment be designed to a minimum level of earthquake forces evaluated in accordance with this Standard paying due regard to ductility and the avoidance of brittle failure.

Proprietary equipment covered by this Standard should be capable of withstanding the serviceability limit state seismic loading evaluated in accordance with the method prescribed without malfunction. Critical proprietary equipment covered by this Standard should also be capable of withstanding the ultimate limit state seismic loading without malfunction.

C4.12.1.2 and C4.12.1.3
When the support or the component is demonstrably unsuited for the direct application of the seismic coefficient method described in 4.12.2, the designer may use established structural dynamics techniques to derive specific seismic coefficients. Typically this will involve the use of time history analysis or floor response spectrum techniques.

C4.12.1.4
The value of C_{pi} is derived principally from considerations of horizontal motion. Accordingly the coefficient is applied directly for horizontal force determination.

C4.12.1.5
Amplification implied by the specified value of C_{pv} is about 2.5 times the ground vertical acceleration, assumed to be two thirds of the horizontal. This value is the maximum response that is likely to occur in an elastic member when the spectral content of the vertical component of ground motion closely matches that of the horizontal component. Accordingly the usual prohibition on using dead load to offset the upward forces has been removed.

C4.12.1.7 and C4.12.1.8
These provisions require that the deflections of the part be considered when determining separations and other protective measures.

C4.12.2 *Basic horizontal coefficient for parts*

C4.12.2.1 *General*
Response of parts is determined from the horizontal acceleration of the floors and a response function assumed to be of the same form as the spectrum used for the design of the building structure. The objective is to allow reasonably simple specification of requirements while ensuring adequately cautious appraisal of forces on parts.

C4.12.2.2 *Floor coefficient*
C_{fo} approximates the maximum ground acceleration multiplied by Sp, and may be considered to be the maximum effective acceleration to which parts resting on the ground would be subject. While the ground acceleration is the activator of building response, it is not directly considered in the design of the building. The equivalent static method and most modal response spectrum techniques, for instance, indicate zero acceleration at the base of the structure. For the design

of parts, however, ground acceleration must be considered. Its effect is most conspicuous near the base of the building.

$C_{f\,n}$ is an estimate of the maximum acceleration of the structure at the top of the building, divided by g. The modal response spectrum method produces reasonable estimates of the likely maximum acceleration at that level. The equivalent static method overestimates $C_{f\,n}$ for short period structures, a result arising from the forces specified at that level which are known to be too large for short period structures. Within the limits specified for the equivalent static method, the method produces reasonable estimates for longer period structures.

If the structural strength exactly matched the design strength requirements of the modal response spectrum method, the maximum acceleration at the top of the structure would be about $F_n.g/W_n$.

The ratio $C_h(T_1,\mu_o)/C_h(T_1,\mu)$ in the expression for $C_{f\,n}$ is intended to reflect increased acceleration of the supports of the part with overstrength of the structure, according to which F_n may approach a magnitude associated with elastic response. A smaller value may be appropriate, but this must be demonstrated. Overstrength is included in the expression through the parameter μ_o. With the default value for μ_o , $C_h(T_1,\mu_o)/C_h(T_1,\mu)$ is the ratio of the spectral ordinate for an elastic structure to the ordinate used in the evaluation of F_n .

Maximum floor accelerations at intermediate levels are difficult to quantify where the equivalent static method is used. Figure C4.12.1 shows the values of F_i/W_i for both the equivalent static method and the modal response spectrum method, for example structures. For short period structures, both methods produce results in close agreement. For longer period structures, results differ markedly. The modal response spectrum method is usually in close agreement with results of time history analysis for modest structural ductility factors. Accordingly, that method may be used for an estimate of the maximum accelerations of floors.

The accurate determination of F_i using the modal response spectrum method will usually require consideration of a large number of modes. While floor deflections can be accurately found from a few modes, modal accelerations are found from modal deflections multiplied by the square of the frequency for each mode, and this leads to a significant contribution to acceleration from each mode. The intention is, however, that the F_i determined from just those modes which need to be considered to satisfy 4.9.1.2 may be used for calculating floor accelerations. The higher modes, while contributing to peak accelerations at supports, are considered to have little effect on the performance of parts and need not, therefore, be considered.

Equation 4.12.5 provides an approximation for floor accelerations where the equivalent static method is used. The equation provides for a linear variation of acceleration with height. The construction is shown in figure C4.12.2. Considering that periods are likely to increase with increasing structural height, the envelope values provided by the approximation are reasonable upper bound estimates for frames (“shear beams”) and for short period walls (“flexural beams”), but are less satisfactory for long period slender flexural cantilevers.

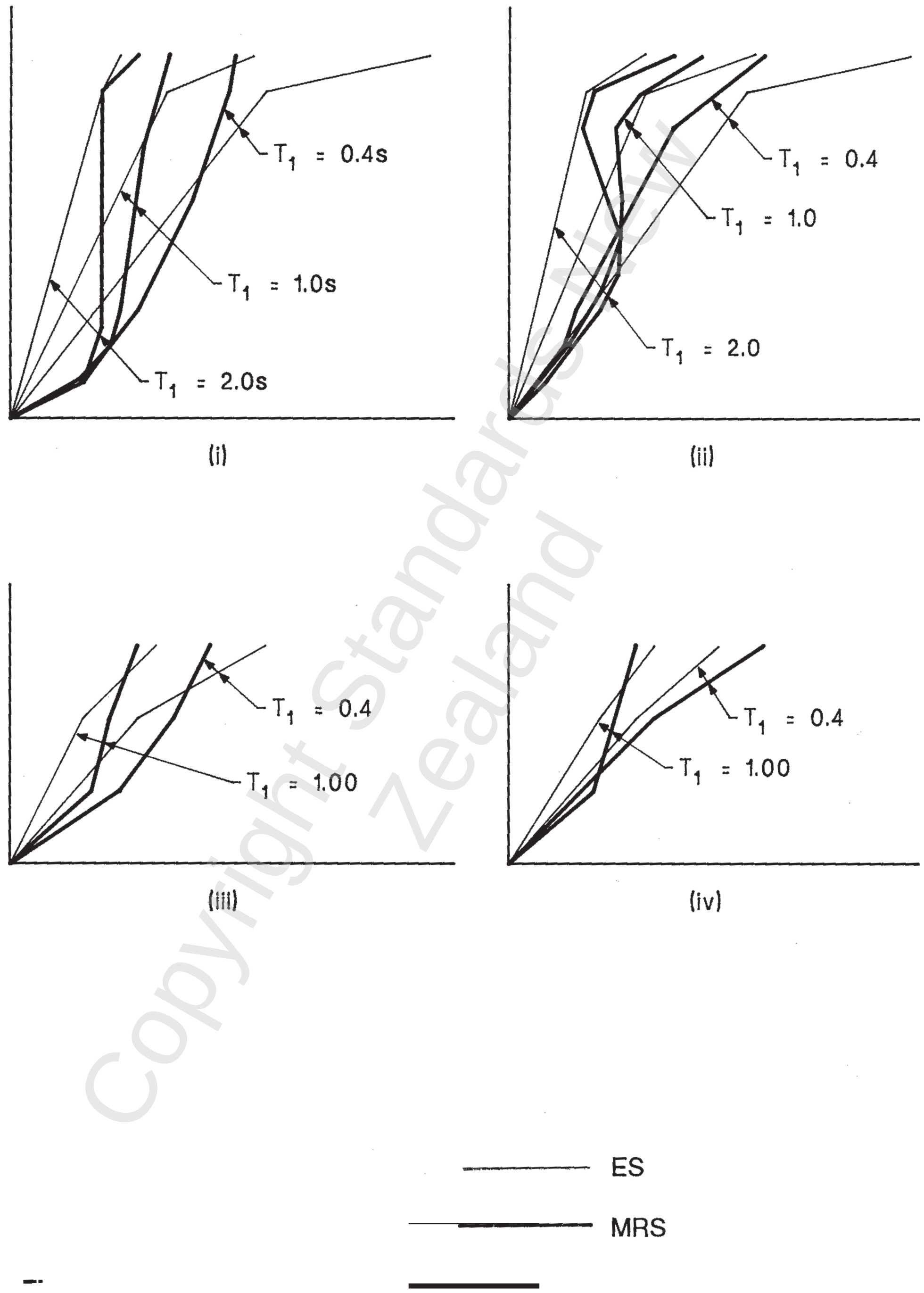
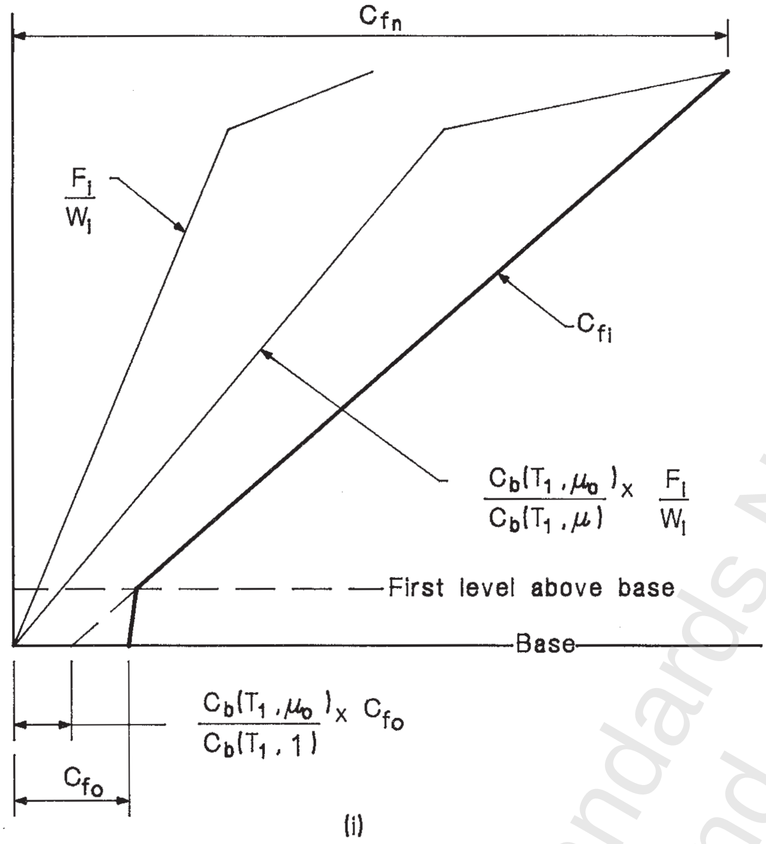


Figure C4.12.1 – Comparisons of F_i /W_i for the equivalent static and modal response spectrum methods for example structures:

(i) 10 storey shear cantilever
(ii) 10 storey flexural cantilever
(iii) 3 storey shear cantilever
(iv) 3 storey flexural cantilever



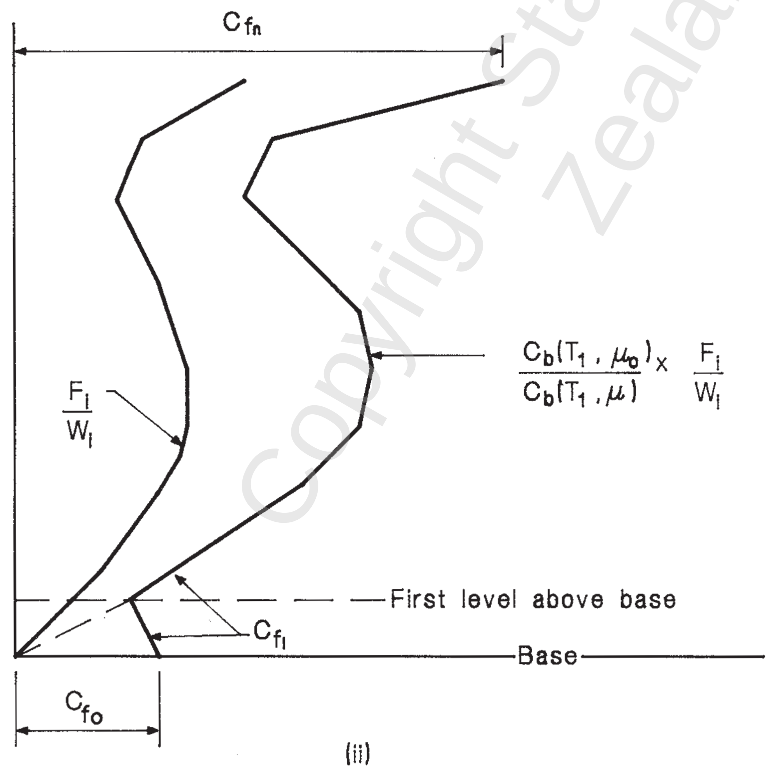


Figure C4.12.2 – Construction of $C_{f\,i}$ curves for:

(i) Equivalent static method
(ii) Modal response spectrum method

NOTES

C4.12.2.3 *Basic horizontal coefficient*

Assumptions embodied in equation 4.12.7 are complex. First, the response function $C_h(T,\mu)$ is assumed to be applicable, even given the differences between the spectral content of the floor motions and the ground motion. Resonance effects, which can be expected to occur when the period of the part is close to a dominant period of the structure (usually T_1) [4.34], have then been accounted for by shifting the peak response from 0.2 s to T_1. This is achieved by normalizing the function with 0.2 T_p/T_1 as abscissa rather than T_1. Then, noting that $C_h(T,\mu)$ has been drawn for an implied maximum ground acceleration of approximately 0.4 g, normalization of the shape of the spectrum is completed by dividing by 0.4.

These devices allow the same suite of spectra to be used for parts as for the structure and allow amplification and near resonance to be modelled by a smooth function. Furthermore, effects of ductility of parts can be accounted for without undue complexity. Ductility markedly reduces the degree of resonance found in studies on elastically responding parts [4.32].

The smallest value for the equivalent period of the part, T_{pe}, is 0.45 s (the factor 0.2 in the expression for T_{pe} carries dimensions of seconds) to reflect the movement of stiff parts into regions of increased response with period increase on stiffness reduction. The period T_p may therefore be assumed zero if the designer so chooses — a reasonable choice in most circumstances — and need never be calculated.

Pending publication of appropriate parameters in the material standards, the values for μ_p in Table C4.12.1 are suggested for common architectural and structural parts. Connections for machinery, switchgear and the like should generally be designed assuming μ_p = 1.0.

Table C4.12.1 – Ductility factors for parts

Item	Description of part	μ_p
1	Partitions, in-fill panels, prefabricated panels	
	(a) Connected so that instability is prevented if stiffness or strength degrades or if integrity is impaired	
	(i) Reinforced concrete or masonry	5.0
	(ii) Steel designed for ductility	6.0
	(iii) Timber or light-gauge steel framing	3.0
	(b) Other (e.g. vertical cantilevers)	
	(i) Reinforced concrete or masonry	
	Doubly reinforced	3.0
	Singly reinforced	2.0
	(ii) Steel or timber framing	1.0
2	Ornamentations, tied veneers, appendages	1.0
3	Floors and roofs acting as diaphragms, and other primary parts distributing seismic forces.	
	(a) Transfer diaphragms	1.0
	(b) Other	
	(i) Designed for limited ductility	3.0
	(ii) Otherwise	1.0
4	Substructures supported at floors, such as frames supporting containers (full), or top light-weight storeys not designed as part of the overall structure.	Table 4.2.1

Notes to Table C4.12.1

Item 1(a) Design should prevent instability if severe damage occurs, such as where a vertical cantilever yields in flexure.

Item 2 It is assumed that modest ductility only is available. Connections, including veneer ties and adhesives for tiling or ornamental stonework, should satisfy 4.12.1.6.

Item 3(a) Transfer diaphragms should be designed with the additional forces arising from redistribution of storey shears.

Item 4 It is recognised that "parts" may constitute "structures within structures". The base shear for the parts should be derived using the structural ductility factor shown in Table 4.2.1 and using the method of section 4.12. Such structures resting on the ground floor may be designed as independent structures, and the results will be similar.

Table C4.12.2 lists normalized values of C_{ph} calculated for the following assumptions:

1. Each structure has equal storey heights and weights, W_i.

2. The fundamental period, T_1, is not less than the greater of 0.6 s and 0.10 n.

3. Design has been by the equivalent static method. This has set the value of F_n/W_n. All structures are capable of responding elastically.

4. The parts, with their connections, are stiff ($T_p = 0$, $T_{pe} = 0.45$ s), but have varying structural ductility factors, μ_p.

5. The structures are sited on flexible or deep soil sites.

6. The table is based on the basic seismic acceleration coefficients provided for in Appendix 4.A.

Table C4.12.2 may be used to read off default value for C_{ph} under a wide range of conditions meeting these assumptions. Structures should generally comply with the limitations for application of the equivalent static method. It is unlikely that buildings will have periods which are less than those noted in the table (for $T_1 > 0.6$ s, $T_1 \geq 0.10$ n). Where the storey height varies, the table remains approximately correct provided n is determined by dividing the total height to the top of the building by the maximum storey height and provided the level, i, is given a corresponding interpretation. Where the site is of rock or intermediate soil type, and where the μ_o is greater than 1.0, the table may produce results which are unacceptably large.

Example C4.12.1 (following table C4.12.2) illustrates the use of table C4.12.2 and the provisions of 4.12.

Table C4.12.2 – Values of $C_{ph}/(RZL_s)$ or $C_{ph}/(RZL_u)$ for parts with $T_{pe} \leq 0.45$ s and varying structural ductility factors, μ_p, in elastically responding regular structures of period T_1 = MAX(0.6 s, 0.10 n) (Flexible or very deep soil sites)

i	0	1	2	3	4	5	6	7	8	9	10
n	$\mu_p = 1$										
1	0.50	1.34									
2	0.50	1.18	1.86								
3	0.50	1.06	1.61	2.17							
4	0.50	0.98	1.45	1.93	2.40						
5	0.50	0.92	1.34	1.75	2.17	2.59					
6	0.50	0.88	1.25	1.63	2.00	2.38	2.76				
7	0.50	0.82	1.14	1.46	1.78	2.10	2.41	2.73			
8	0.50	0.77	1.05	1.32	1.59	1.86	2.14	2.41	2.68		
9	0.50	0.73	0.96	1.19	1.42	1.65	1.89	2.12	2.35	2.58	
10	0.50	0.70	0.90	1.10	1.29	1.49	1.69	1.89	2.09	2.29	2.49
n	$\mu_p = 2$										
1	0.31	0.82									
2	0.31	0.72	1.14								
3	0.31	0.65	0.99	1.33							
4	0.31	0.60	0.89	1.18	1.47						
5	0.31	0.56	0.82	1.07	1.33	1.59					
6	0.31	0.54	0.77	1.00	1.23	1.46	1.69				
7	0.31	0.50	0.70	0.89	1.09	1.28	1.48	1.67			
8	0.31	0.47	0.64	0.81	0.97	1.14	1.31	1.48	1.64		
9	0.31	0.45	0.59	0.73	0.87	1.01	1.16	1.30	1.44	1.58	
10	0.31	0.43	0.55	0.67	0.79	0.91	1.04	1.16	1.28	1.40	1.52
n	$\mu_p = 3$										
1	0.22	0.59									
2	0.22	0.52	0.81								
3	0.22	0.46	0.71	0.95							
4	0.22	0.43	0.63	0.84	1.05						
5	0.22	0.40	0.58	0.77	0.95	1.13					
6	0.22	0.38	0.55	0.71	0.88	1.04	1.21				
7	0.22	0.36	0.50	0.64	0.78	0.92	1.06	1.20			
8	0.22	0.34	0.46	0.58	0.70	0.82	0.94	1.05	1.17		
9	0.22	0.32	0.42	0.52	0.62	0.72	0.83	0.93	1.03	1.13	
10	0.22	0.31	0.39	0.48	0.57	0.65	0.74	0.83	0.91	1.00	1.09

Table C4.12.2 (continued)

i	0	1	2	3	4	5	6	7	8	9	10
n					$\mu_p = 4$						
1	0.17	0.45									
2	0.17	0.40	0.63								
3	0.17	0.36	0.54	0.73							
4	0.17	0.33	0.49	0.65	0.81						
5	0.17	0.31	0.45	0.59	0.73	0.87					
6	0.17	0.30	0.42	0.55	0.68	0.80	0.93				
7	0.17	0.28	0.38	0.49	0.60	0.71	0.81	0.92			
8	0.17	0.26	0.35	0.45	0.54	0.63	0.72	0.81	0.91		
9	0.17	0.25	0.32	0.40	0.48	0.56	0.64	0.71	0.79	0.87	
10	0.17	0.24	0.30	0.37	0.44	0.50	0.57	0.64	0.70	0.77	0.84
n					$\mu_p = 5$						
1	0.14	0.37									
2	0.14	0.32	0.51								
3	0.14	0.29	0.44	0.60							
4	0.14	0.27	0.40	0.53	0.66						
5	0.14	0.25	0.37	0.48	0.60	0.71					
6	0.14	0.24	0.34	0.45	0.55	0.65	0.76				
7	0.14	0.23	0.31	0.40	0.49	0.58	0.66	0.75			
8	0.14	0.21	0.29	0.36	0.44	0.51	0.59	0.66	0.74		
9	0.14	0.20	0.26	0.33	0.39	0.46	0.52	0.58	0.65	0.71	
10	0.14	0.19	0.25	0.30	0.36	0.41	0.47	0.52	0.57	0.63	0.68
n					$\mu_p = 6$						
1	0.12	0.32									
2	0.12	0.28	0.44								
3	0.12	0.25	0.38	0.52							
4	0.12	0.23	0.34	0.46	0.57						
5	0.12	0.22	0.32	0.42	0.52	0.62					
6	0.12	0.21	0.30	0.39	0.48	0.57	0.65				
7	0.12	0.19	0.27	0.35	0.42	0.50	0.57	0.65			
8	0.12	0.18	0.25	0.31	0.38	0.44	0.51	0.57	0.64		
9	0.12	0.17	0.23	0.28	0.34	0.39	0.45	0.50	0.56	0.61	
10	0.12	0.17	0.21	0.26	0.31	0.35	0.40	0.45	0.50	0.54	0.59

EXAMPLE C4.12.1

The four storey building partly shown in figure C4.12.3 is on a soft soil site in Wellington (where Z = 1.2). The building has T_1 = 0.6 s, has been designed using μ = 3 and the equivalent static method. It is required to determine the forces on the parts shown (parapet, penthouse frame, and the precast wall panel) for the ultimate limit state (L_u = 1.0). Capacity design has been applied and reveals μ_o = 1.0 (or this default value has been assumed). All parts are assumed to be stiff, with T_p = 0 and T_{pe} = 0.45 s. In this example, use is made of table 4.A2.1 and table C4.12.2.

(a) **Using a general approach**

For the building as a whole,

$$\begin{aligned} C &= S_p\, C_h\,(0.6,3)\ \ RZL_u \\ &= 0.67 \times 0.37 \times 1.0 \times 1.2 \times 1.0 \\ &= 0.297 \\ W_t &= 4W \\ V &= CW_t = 1.190\ W \end{aligned}$$

For use of equation 4.8.2, noting that W_i is constant, $W_i\, h_i\, /\, \Sigma(W_i h_i)$ is the same as $i/\Sigma(i)$, and $\Sigma(i)$ = 10

$$\begin{aligned} \text{Hence } F_n &= (0.92 \times 4/10 + 0.08) \times 1.190\ W \\ &= 0.533\ W \end{aligned}$$

$$\text{Therefore } C_{fn} = \frac{C_h\,(0.6,1)}{C_h(0.6,3)} \times \frac{F_n}{W_n} = \frac{1.00}{0.37} \times \frac{0.553}{1.000}$$
$$= 1.441$$

$$\begin{aligned} \text{and}\quad C_{fo} &= 0.25\ RZL_u = 0.25 \times 1.00 \times 1.20 \times 1.00 \\ &= 0.300. \end{aligned}$$

For linear interpolation between the base and the top of the structure, we also need the scaling factor for C_{fo} of equation 4.12.5

$$\frac{C_h\,(T_1,\mu_o)}{C_h\,(T_1,\,1)} = 1.00 \text{ for this case } (\mu_o = 1.0)$$

The resulting values of C_{fi} at all levels are shown alongside the building elevation.

(1) For the parapet, μ_p = 2 (singly reinforced).

$$\begin{aligned} C_{pi} &= C_h\,(T_{pe},\,\mu_p) \times C_{fi}/0.4 \text{ (equation 4.12.7)} \\ &= C_h(0.45,2) \times C_{fi}/0.4 \\ &= 0.49/0.4 \times C_{fi} \text{ (from table 4.6.1(b)).} \\ &= 1.225\ C_{fi}. \end{aligned}$$

At parapet support level, $C_{fi} = C_{fn}$ = 1.441.

$$\begin{aligned} \text{Hence}\quad C_{ph} &= 1.225 \times 1.441 \\ &= 1.77 \end{aligned}$$

(2) For the penthouse, μ_p = 6 (ductile steel frame).

$$
\begin{aligned}
C_{pi} &= C_h\,(T_{pe}, \mu_p) \times C_{fi}/0.4 \\
&= C_h(0.45,6)C_{fi}/0.4 \\
&= 0.19/0.4 \times C_{fi} \\
&= 0.475\ C_{fi}
\end{aligned}
$$

Hence $C_{ph} = 0.475 \times 1.441$
$= 0.68$

(3) For the precast panel, μ_p = 5 (supported laterally top and bottom, preventing instability).

$$
\begin{aligned}
C_{pi} &= C_h\,(T_{pe}, \mu_p) \times C_{fi}/0.4 \\
&= C_h(0.45,5) \times C_{fi}/0.4 \\
&= 0.22/0.4 \times C_{fi} \\
&= 0.550\ C_{fi}
\end{aligned}
$$

The average value of C_{fi} at the top and bottom of the panel is

$$
\begin{aligned}
C_{fi} &= (0.871 + 1.156)/2 \\
&= 1.014
\end{aligned}
$$

Hence $C_{ph} = 0.550 \times 1.014$
$= 0.56$

(b) **Using table C4.12.2**

NOTE – RZL_u = 1.2

(1) For the parapet

$$\frac{C_{ph}}{RZL_u} = 1.47\ (\mu_p = 2,\ n = 4,\ i = 4)$$

$$C_{ph} = 1.76$$

(2) For the penthouse

$$\frac{C_{ph}}{RZL_u} = 0.57\ (\mu_p = 6,\ n = 4,\ i = 4)$$

$$C_{ph} = 0.68$$

(3) For the precast panel

$$\frac{C_{ph}}{RZL_u} = 0.465\ (\mu_p = 5,\ n = 4,\ i = 2.5)$$

$$C_{ph} = 0.56$$

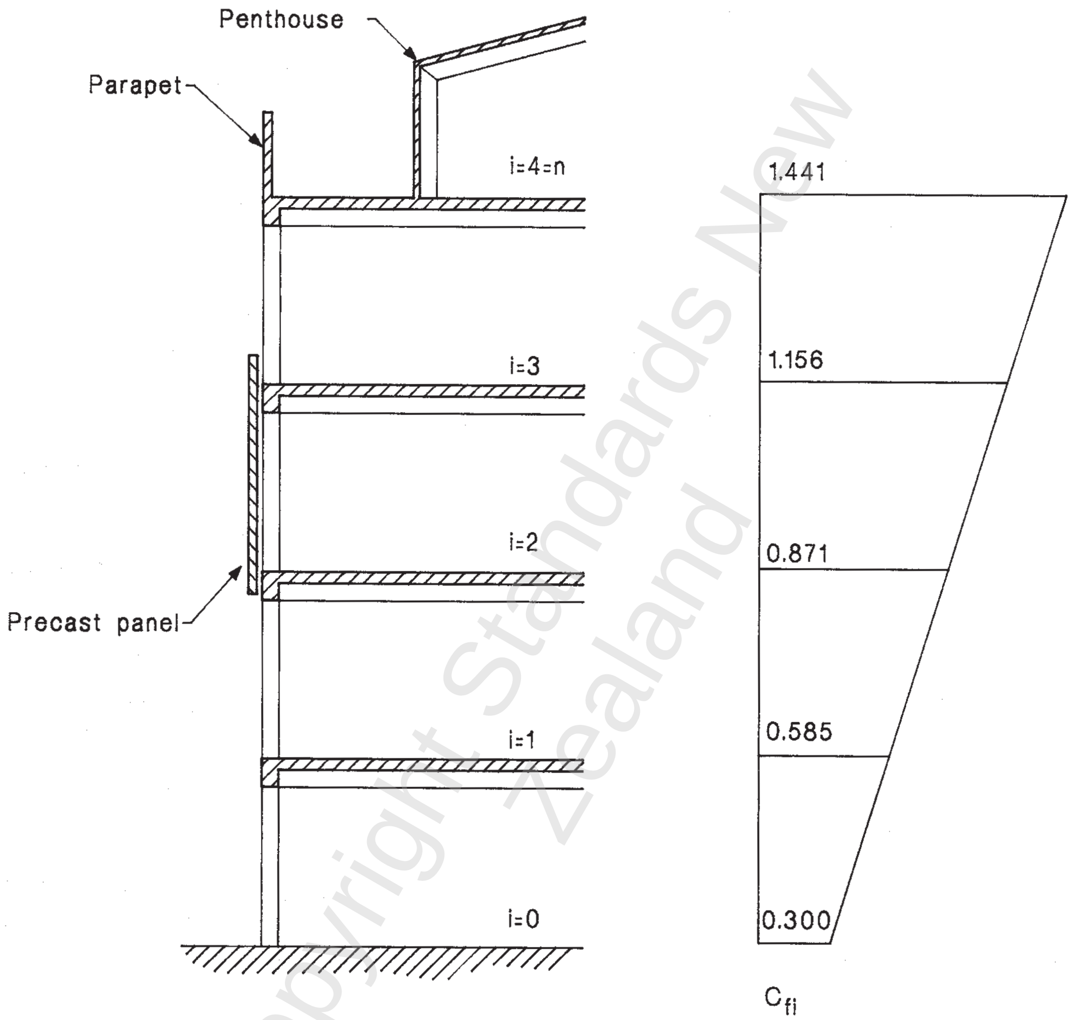


Figure C4.12.3 – Example C4.12.1

APPENDIX C4.A
GUIDELINES FOR ESTABLISHING A STRUCTURAL DUCTILITY FACTOR FOR A STRUCTURAL FORM OR MATERIAL THAT IS NOT COVERED IN TABLE 4.2.1

C4.A1

Structural ductility factors are used for assessing the appropriate response spectrum to use with the equivalent static or modal response methods of analysis. The following steps may be carried out to assess the appropriate structural ductility factor for materials or structural forms not covered in table 4.2.1. In no case should a structural ductility factor greater than 6 be used.

(1) Conduct an elastic analysis of the structure for earthquake effects using either the equivalent static or the modal response methods. The method employed should be chosen to satisfy the structural regularity requirements set out in section 4.4. In the analysis the response spectrum used corresponds to an assumed structural ductility factor.

(2) Determine the maximum lateral displacement (generally at the top of the structure) equal to the derived deflection found in step (1) multiplied by the assumed ductility factor.

(3) For the assumed structural ductility factor, determine the strengths required to resist the applied load combinations given in 2.4.3.3. The locations where energy is to be dissipated are to be identified and capacity design procedures used to ensure that energy dissipation is confined to those locations.

(4) Apply the gravity loads and incrementally apply a set of lateral forces to the structure until the displacement value found in step (2) is reached. In the analysis the inelastic force-deformation characteristics of the zones detailed for energy dissipation must be modelled. The set of forces used should be based on the equivalent static values where that method of analysis was used in step (1) or on the forces corresponding to the first mode shape where the modal response method was used.

(5) Reverse the direction of the set of lateral forces and apply them incrementally to the structure in that opposite direction until the displacement reaches minus the value determined in step (2).

(6) Repeat steps (4) and (5) until four complete displacement cycles to the displacement limits specified in steps (4) and (5) have been achieved.

(7) The maximum inelastic deformations obtained in the structure during steps (4), (5) and (6) should be multiplied by 1.25 to allow for dynamic magnification effects which may be expected to increase localized deformations in the structure above those generated by the set of lateral forces used in the analysis.

(8) The results of tests on subassemblages or individual members should be examined to establish that these can sustain not less than 80 % of their maximum measured strength when subjected to the displacement history obtained in steps (4), (5) and (6), with the peak deformations being increased as detailed in step (7).

(9) If the requirements in step (8) can be satisfied, the structural ductility factor assumed in step (1) may be taken as the appropriate value.

APPENDIX C4.B
P-DELTA EFFECTS IN BUILDINGS DUE TO EARTHQUAKE ACTION

C4.B1 Introduction

When a structure is displaced, the P-delta actions reduce the resistance of the structure to further displacement in the same direction. This becomes important in ductile structures, as each time the inelastic range is entered there is a tendency for the displacement to increase. Any structure which develops a sway mechanism in an earthquake will collapse if the duration of the severe ground motion is of sufficient duration. Consequently the P-delta effects in a structure increase with:

(a) An increase in the ductility demand on the structure;

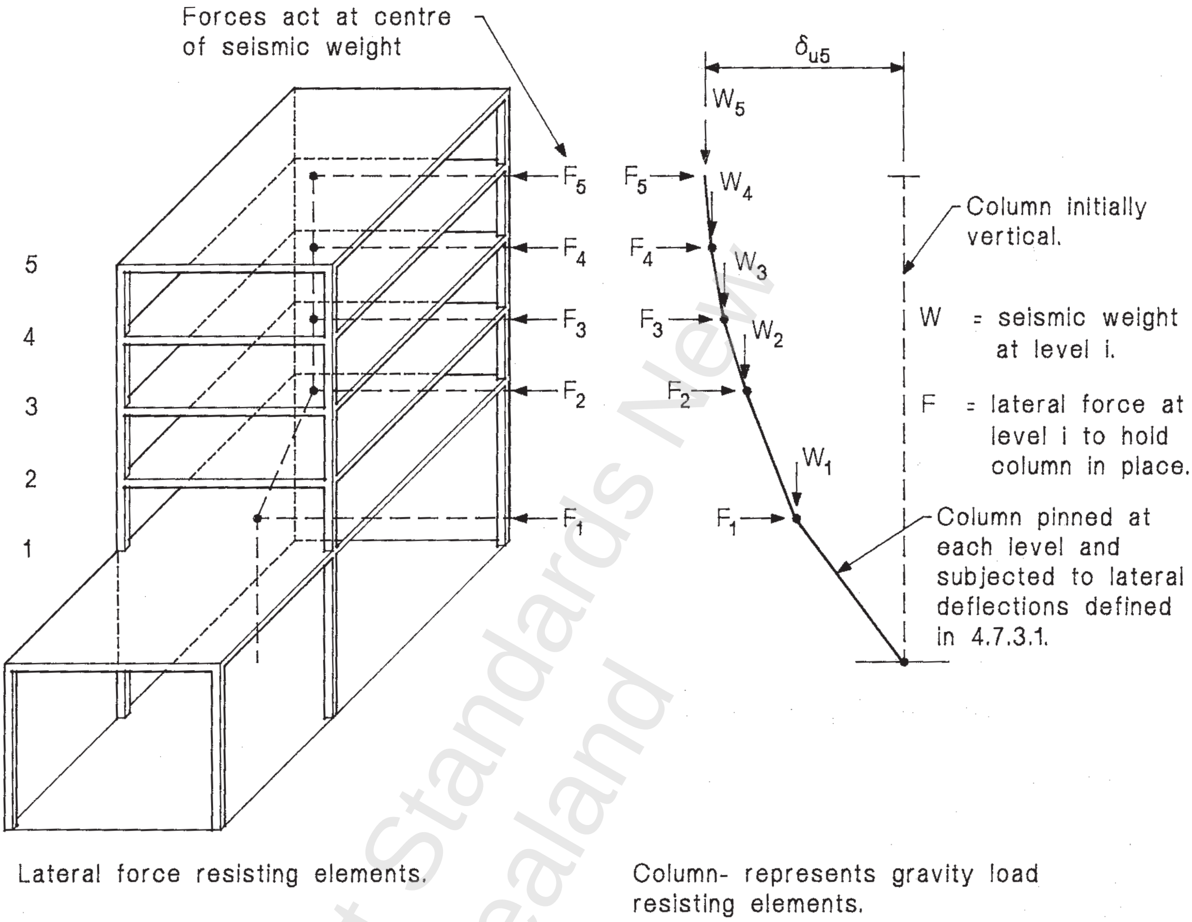
(b) The duration of the severe ground motion in the earthquake, and

(c) The inverse of the fundamental period of the structure, as this reduces the number of inelastic cycles sustained in a given earthquake.

Research has allowed an assessment to be made of the influence of ductility demand and the fundamental period on P-delta effects [4.20, 4.22, 4.23]. However, at present no numerical assessment can be made for the duration of severe ground shaking. The provisions in the proposed method outlined in C4.B2 are based on the analyses of earthquake ground motions with about 15 to 25 seconds of severe ground motion.

In assessing the influence of P-delta actions on the lateral force resisting elements in the building, it should be noted that the full gravity load supported by the structure contributes to those actions and not just the load supported by the frame or wall being considered. The P-delta effects arise due to the lateral displacement of the gravity load carrying members. The inclination of these induce shear in the lateral force resisting system. At each level this shear exactly balances the horizontal component of the inclined forces resisting the gravity loads. A simple model that can be used to assess P-delta actions is illustrated in figure C4.B1. In this model the gravity load carrying structure is visualized as being a column, which is pinned at every level. Initially it is assumed to be straight. When this column is deflected into the required profile a set of lateral forces, F_1, F_2, F_3 etc. are required to hold it in this position. The shear in each storey may be found from the inclination of the column and the force at each level may be determined by taking the difference in the shears in the adjacent storeys. The forces are scaled by an amplification factor, β, which makes an allowance for the ductility demand (structural ductility factor) and a factor K, which allows for the influence of the fundamental period and the foundation soil type. Those scaled forces, when applied to the structure, induce the additional structural actions to be used when allowing for P-delta effects.

P-delta actions will increase the displacements of the structure. The procedure outlined in C4.B2 aims at preventing unacceptable increases in the structural ductility demand, but it should be appreciated by designers that the displacements nevertheless will be larger than where P-delta affects are ignored. The displacements at yield increase. Consequently the total displacements also increase, even when the structural ductility factor is unaltered. This is addressed in Step 7 of the procedure outlined in C4.B2.



NOTE – The forces F_1, F_2 etc. are internal forces which act between the gravity load resisting elements and the lateral force resisting elements.

Figure C4.B1 – Model used to find equivalent lateral P-delta forces

C4.B2 Steps in the analysis of a building for P-delta effects

Step 1. Analyse the structure by the equivalent static or modal response spectrum method neglecting P-delta actions.

Step 2. Determine the lateral deflection of the centroid of the seismic weight at each level as defined in 4.7.3.1.

Step 3. From the deflected profile found in Step 2 and the model described in C4.B1 and figure C4.B1, find the horizontal components in each storey due to the inclined force in the gravity load resisting elements. From these values, find the set of lateral forces F_1, F_2, F_3 etc.

Step 4. Calculate the value of β, which makes an allowance for the ductility demand (as measured by the structural ductility factor) and K, which makes an allowance for the period and the foundation subsoil type.

$\beta = 2\mu K/3.5$ for $\mu \leq 3.5$

$= 2.0$ K for $\mu > 3.5$

For site subsoil categories (a) and (b)

K $= 1.0$ for $T_1 \leq 2.0$ seconds

$= \frac{(6-T_1)}{4}$ for $2.0 \leq T_1 \leq 4.0$

$= 0.5$ for $T_1 > 4$ seconds

For site subsoil category (c)

K $= 1.0$ for $T_1 \leq 2.5$ seconds

$= \frac{6.5 - T_1}{4}$ for $2.5 \leq T_1 \leq 4.5$

$= 0.5$ for $T_1 > 4.5$ seconds

Step 5. The forces F_1, F_2, F_3 (found in Step 3) are multiplied by β (found in step 4) and are applied to the lateral force resisting elements in the structure, and the resultant structural actions are found. The forces βF_1, βF_2 etc., pass through the respective centroids of the seismic weight on their respective levels.

Step 6. The structural actions and deflections found in Step 5 are added to the corresponding values found from the analysis of the structure in Step 1.

Step 7. The additional deflections at each level due to the P-delta actions are equal to the structural ductility factor times the corresponding deflections found from the elastic analysis in step 5.

The background to the method described above is described in reference 4.23.

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PART 5 PROVISIONS FOR WIND FORCES

C5.2 General requirements

C5.2.1 *Determination of wind forces*
The wind forces applied to structures are found by assessing the site wind speed which has a 5 % probability of exceedence either over 50 years (ultimate) or in one year (serviceability). The basic wind speed for each region has been determined from normalised anemometer records for different directions (as indicated by the direction multiplier, M_d). The basic wind speed is further modified by the passage of the wind over and around land forms and other features as it approaches the site. The degree of modification will usually be different from different directions and will depend upon ground roughness, various topographic effects and the degree of shielding in the vicinity of the site.

Either the design may use detailed directional effects, in which case the most severe site wind speed profile for each of the four orthogonal axes of the building are required to be used during the design process; or alternatively a simplified, more conservative, non-directional approach may be used. The conservatism is introduced by using a non-directional wind speed which approaches the maximum specific directional wind speed and by assuming the largest individual wind speed applies for all directions.

For the purposes of determining forces and pressures, wind tunnel tests, or similar tests employing a fluid other than air, shall be considered properly conducted only if the natural wind has been modelled for the appropriate terrain categories to take account of:

(a) The variation of wind speed with height,

(b) The scale and intensity of the longitudinal component of turbulence.

Notice shall be taken of:

(i) The effects of Reynolds Number where curved shapes are involved,

(ii) The appropriate frequency response of force and pressure measuring systems, and

(iii) Scaling of mass, length, stiffness, and damping, where measurements of dynamic response are involved.

Detailed information on appropriate wind tunnel testing procedures are provided by Reinhold [5.1].

Where properly conducted wind tunnel tests on a specific structure have been carried out, or where reference to such tests on a similar structure is used, the forces so determined shall be used instead of those determined through the provisions of this Part.

C5.2.2 *Limits of application*

C5.2.2.1
The term 'Static Analysis' in this context refers to a determination of design wind forces and moments based on quasi-steady assumptions. The static analysis procedure is based on the assumption that the wind loads on a structure or element can be determined by using a mean force coefficient in conjunction with a design gust wind speed. In so doing, time-dependent effects such as, the development of a pressure field, the correlation of pressures and the dynamic response of a structure are not taken into account.

Regardless of the wind sensitivity of the structure, all cladding elements and their supports are required to be designed using this "static analysis" procedure.

C5.2.2.2
Section 5.9 provides the dynamic wind design parameters applicable for New Zealand conditions which enable the use of the Dynamic Analysis procedures outlined in AS 1170.2, Section 4.

Most moderately large buildings in New Zealand will satisfy the criteria outlined for buildings which are not wind sensitive and may be designed using the 'static analysis procedure' detailed in this Part of the Standard. The results for such structures will usually be somewhat conservative compared to those obtained using the alternative 'dynamic design procedure'. Economies are possible by using the more precise dynamic design procedure for the structural actions, although the design of elements and components still requires the use of the static analysis procedure.

C5.2.2.3
The parameters used for enclosed buildings have been determined after a study of the performance of several multi-storey New Zealand buildings [5.2], and have been derived from a "best fit" approach to instances where the forces on the structure derived using the dynamic design procedure exceeded those derived using the static analysis procedure. The results apply to enclosed buildings only at their ultimate limit state.

Special structures such as chimneys (for vortex shedding) and pole houses (for lateral vibration) may also require more detailed examination for wind.

The above study [5.2] also identified that serviceability limits (i.e., lateral acceleration limits of 0.01 g for office and residential occupancy buildings) may be exceeded where:

$h^{1.3}/m>1.6$

where

h is the building height, m
m is the mass of the building plus long term live load for unit height of the building, t/m.

The designer should be satisfied that these levels of lateral acceleration are appropriate for the building being designed, and may then undertake a dynamic wind design to quantify the along and across wind accelerations if required.

Building accelerations increase inversely with building mass, directly with sway period, and the cube of wind speed. Because both wind speed and building period are functions of building height, a wind sensitivity criterion based only on structure height and mass has been derived to identify possible dynamic response problems. However, this criterion does not take account of wind speed variations from one locality to another. Therefore if a building design fails to meet the criterion, a more detailed analysis is required to determine the likely acceleration levels. This can be achieved either by using the analytical procedures presented in the Wind Design Guide [5.2] or through wind tunnel testing. Both approaches necessitate accurate assessment of site wind speeds as a 10 % error in wind speed may give a 30 – 40 % error in the predicted accelerations.

C5.2.2.4
While tornadoes are special phenomena for which insufficient data are available to allow their effects to be codified, the few measurements of wind speeds in tornadoes in New Zealand indicate that the ultimate wind speeds derived using the procedures outlined in this Part, should be sufficient in most instances.

C5.3 Design wind speed, $V_{d(z)}$

C5.3.1
This revision to the Standard is the first which incorporates information which enables the designer to consider directional wind effects. An alternative, conservative approach which ignores direction is also outlined. The 'Design Wind Speed, $V_{d(z)}$' for a structure includes both directional considerations and variations with height, which may differ depending on the shape

and configuration of the building. The design wind pressure, $q_{(z)}$, derived from $V_{d(z)}$, is used in conjunction with the published pressure coefficients to determine the wind forces on the building. It is the number of pressure coefficients available which dictates the width of the angular sector that must be considered when determining the design wind speed. Pressure coefficients published in this Part of the Standard are for wind from four orthogonal directions for different shapes of buildings. This requires that the design wind speed equals the most severe site wind speed within a 90˚ arc symmetrically orientated about the direction under consideration. Pressure coefficients may be available from wind tunnel testing or other sources for wind effects from more directions (typically from 8 or 16 directions) than those indicated by this Standard. Such coefficients are permitted to be used, in which case the width of the wind speed arc reduces to 360˚/(the number of pressure coefficients). However additional sets of orthogonal wind effects are required to be considered, and the four orthogonal design wind speeds would coincide with the most severe of these effects.

Basic directional wind speeds for eight directions and for seven wind regions are supplied (table 5.4.1). These are required to be modified by wind speed modification factors applicable for conditions up-wind from the site, and result in site wind speeds for each of the eight directions. The site wind speed for intermediate directions is determined by interpolating given site wind speeds with direction. An example of this is as follows:

EXAMPLE C5.3.1

Problem: Determine the ultimate wind speed for a 15 m high building located on the western boundary of Christchurch central business district.

Limit state multiplier M_{ls} = 0.93
Importance multiplier M_i = 1.0

Terrain/height multiplier, $M_{(z,cat)}$:

NE, E & SE Terrain category 3.5; $M_{(15,3.5)} = (0.89 + 0.75)/2 = 0.82$
S, SW, NW, N Terrain category 3; $M_{(15,3)} = 0.89$

West terrain category = 3 for 500 m; and = 2 beyond

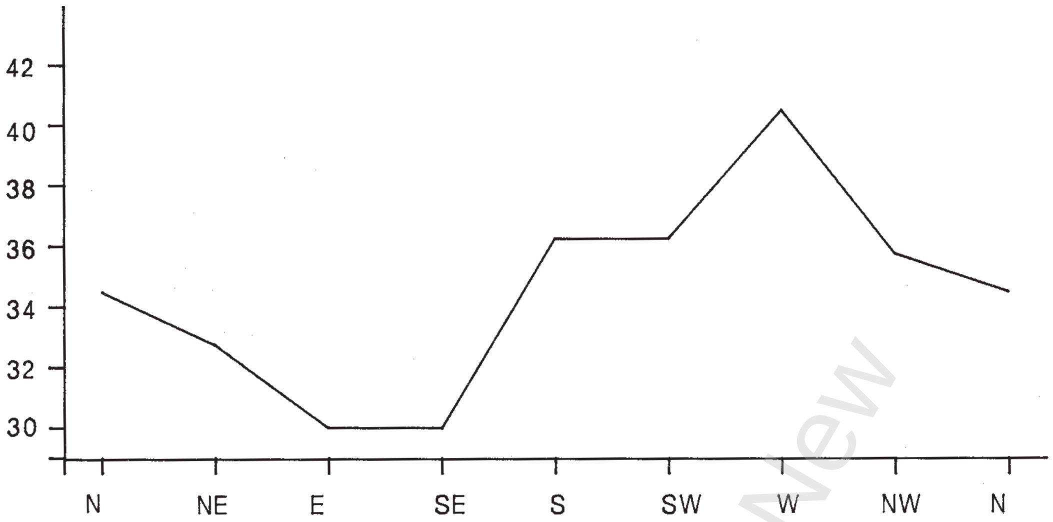
Transition zone depth = 50 x 15 m = 750 m
From westerly direction weighted average on distance of terrain
Terrain category = (500 X 3 + 250 X 2)/750 = 2.7
$M_{(15,2.7)} = 1.05 + 0.7\ (0.89 - 1.05)$
$= 1.05 - 0.11 = 0.94$
Shielding multiplier, M_s:

Easterly direction full shielding $M_s = 0.8$

NW & N Partial shielding $M_s = 0.9$
Elsewhere $M_s = 1.0$

Topographic Multiplier, $M_t = 1.0$

Direction		N		NE		E		SE		S		SW		W		NW		N	
$V_{(z)}$		46		43		39		39		44		44		46		48		46	
M_{ls}		.93		.93		.93		.93		.93		.93		.93		.93		.93	
$M_{(z,cat)}$		.89		.82		.82		.82		.89		.89		.94		.89		.89	
M_s		.90		1.0		.80		1.0		1.0		1.0		1.0		.90		.90	
$V_{d(15)}$		34.3		32.8		23.8		29.7		36.4		36.4		40.2		35.8		34.3	
$V_{d(15)}$*	35.1		33.6		28.3		26.8		33.1		36.4		38.3		38.0		35.1		33.6
					(=30.0)														



Building with north orientation	$V_{d(15)}$ =	35.8 (N);	32.8 (E);	36.4 (S);	40.2 (W)
Building with NE orientation	$V_{d(15)}$ =	34.3 (NE);	36.4 (SE);	40.2 (SW);	40.2(NW)
*Building NNE orientation	$V_{d(15)}$ =	35.1 (NNE);	33.1 (ESE);	38.3 (SSW);	40.2 (WNW)

C5.3.2

The force and pressure coefficients presented in this Part of the Standard are applicable to wind from two orthogonal directions only (Ref sections 5.6, 5.7, 5.8). This simplification, which neglects high corner pressures etc., requires the application of the largest corrected directional wind speed within a 90° segment symmetrically positioned about the orthogonal direction being considered.

To determine the applicable wind speeds for the site, a plot of wind speeds, including the appropriate multiplication factors, should be prepared. The orthogonal axes under consideration should be superimposed, and the maximum wind speeds within a 90° segment centred on these axes read from the plot.

C5.4 Site wind speed

C5.4.1 *Derivation of the site wind speed,* $V_{(z)}$

The basic directional wind speeds, V (from table 5.4.1) identify the directional variations of normalised anemometer readings observed from various stations within each region. The wind speed applicable to a specific site is required to be modified by the appropriate wind speed modification factors identified in equation 5.4.1.

The limit state multiplier, M_{ls}, and the importance multiplier, M_i , are functions of the limit state being considered and of the structure itself respectively. These two multipliers apply to winds from all directions.

The terrain/height multiplier, $M_{(z,cat)}$, the shielding multiplier, M_s, and the topographic multiplier, M_t , are each influenced by effects upwind of the site. These are likely to vary for winds from different directions, requiring that each multiplier be determined for each direction considered. Directional variations may be ignored by using the non-directional basic wind speed in table 5.4.1, together with the maximum values of $M_{(z,cat)}$, M_s and M_t which are applicable to the site for wind from any direction. Such maxima would usually be able to be determined by inspection.

C5.4.1.2
The designer may elect to either:

(a) Consider the wind speed from each of the 8 directions given, and determine the most severe design wind speed by interpolation of those 8 values, depending upon the building orientation, or

(b) Ignore directional effect and use non-directional wind effects, in which case the largest value of each multiplier applicable to the site shall be used to determine the site wind speed profile.

In many instances, where non-directional effects are considered, the most critical value of each multiplier may be determined by inspection only.

Flow charts given in figures C5.4.1 (a), (b) & (c) show the procedures for the determination of design wind speed.

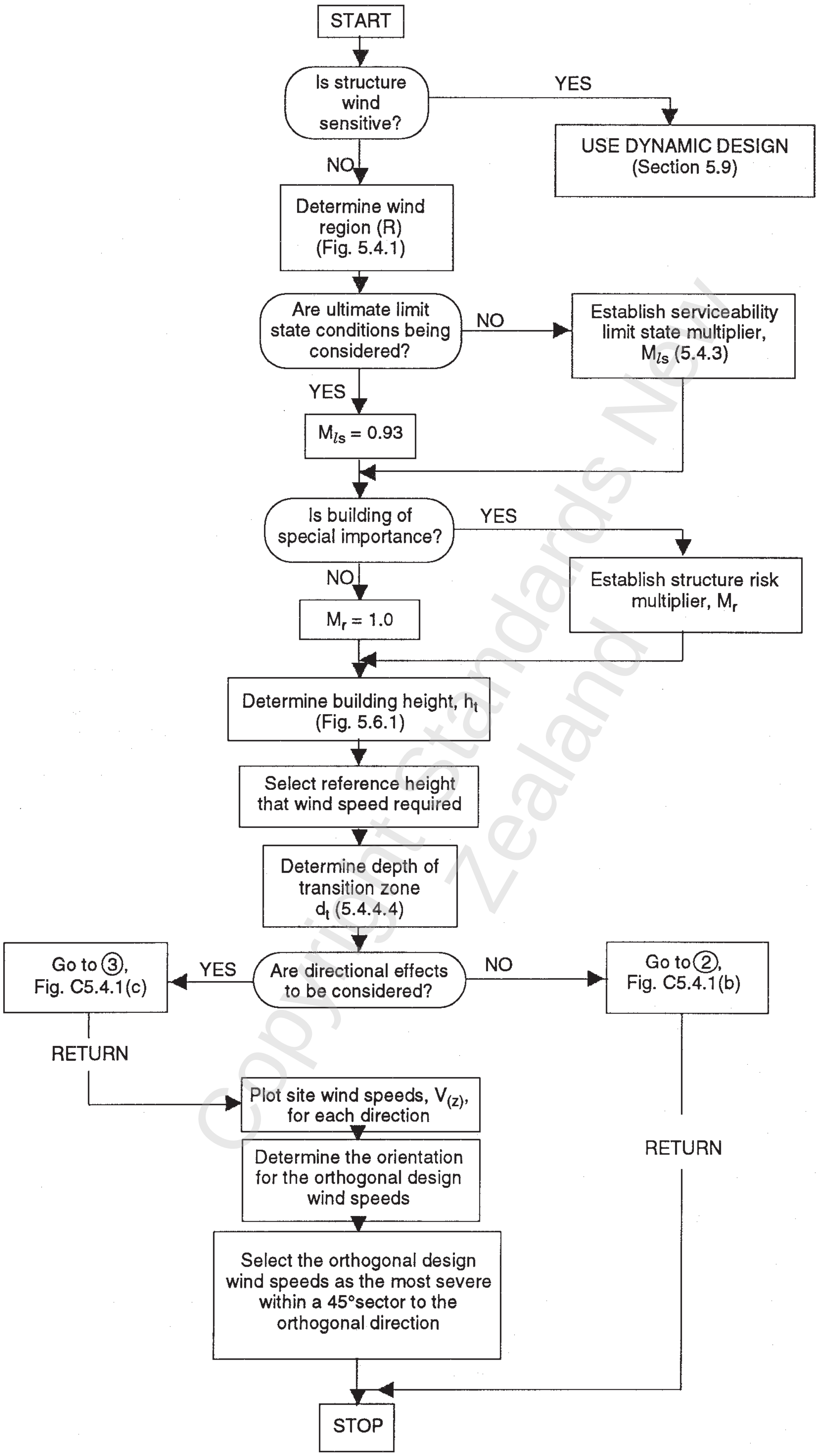


Figure C5.4.1 (a) – Flow chart for determining design wind speed

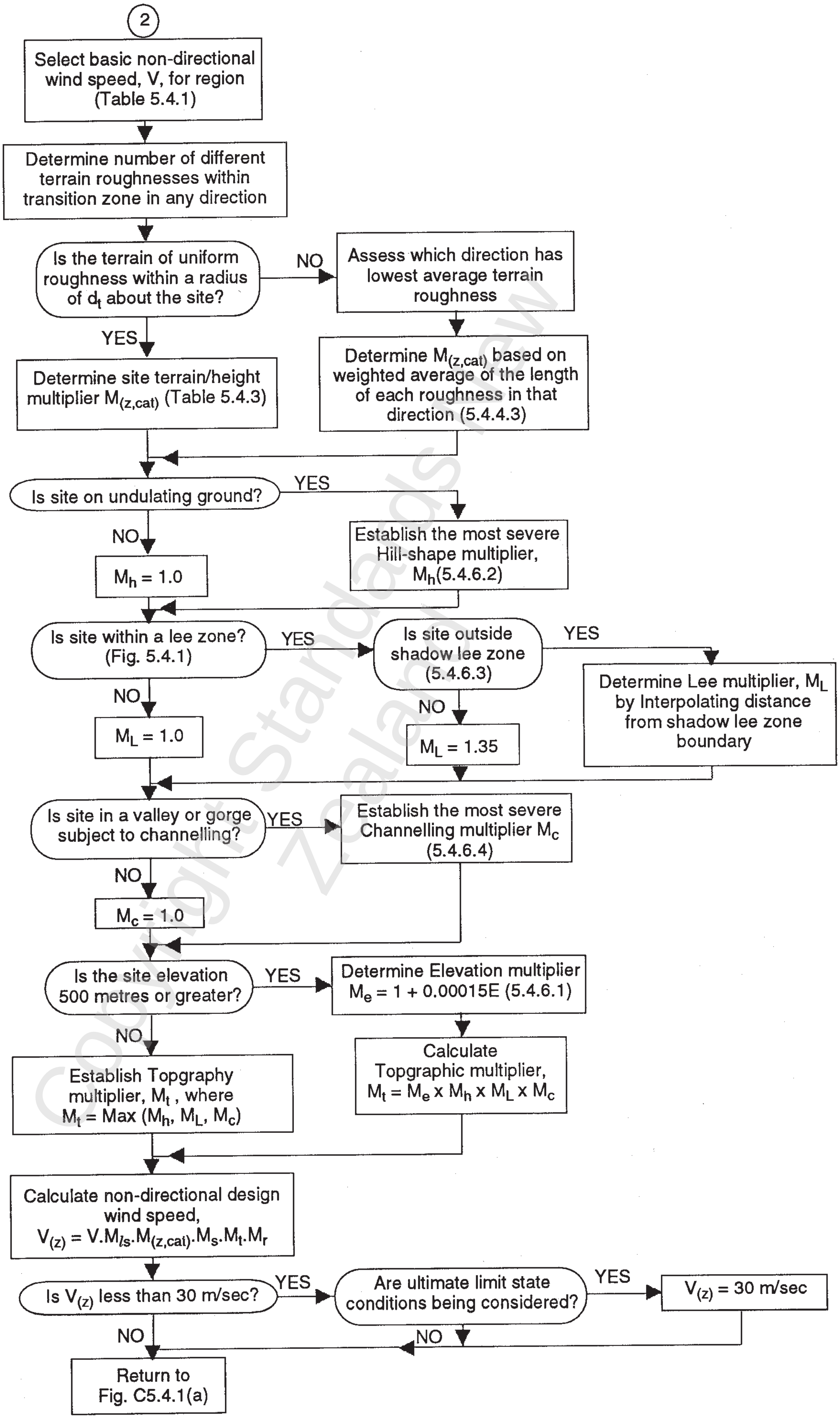


Figure C5.4.1 (b) – Flow chart for determining non-directional site wind speed, $V_{(z)}$

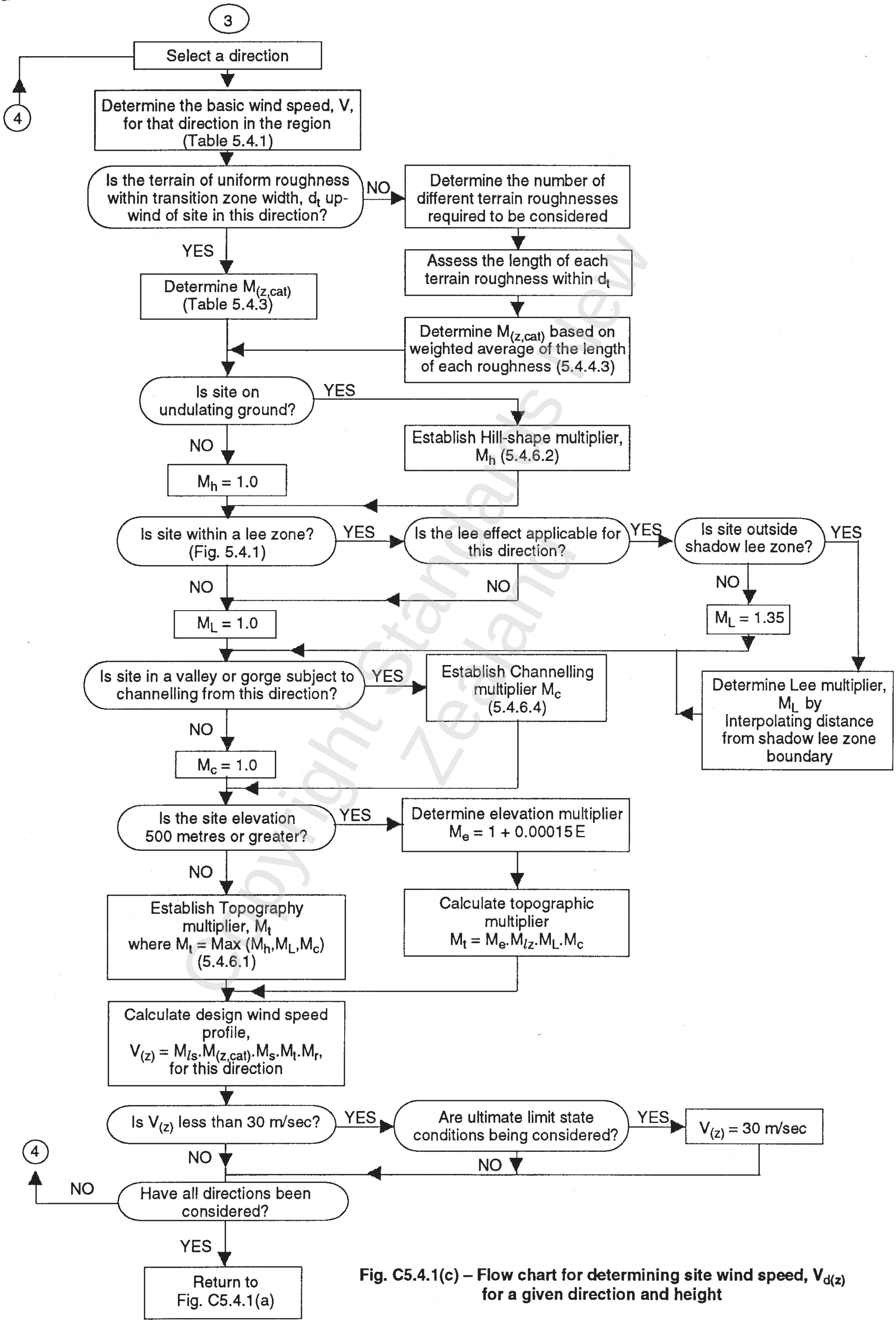


Fig. C5.4.1(c) – Flow chart for determining site wind speed, $V_{d(z)}$ for a given direction and height

C5.4.2 *Basic directional wind speed*, V

C5.4.2.1
The basic directional wind speeds are based upon corrected anemometer readings collected by the NZ Meteorological Service and analysed by Reid. The regional boundaries for each wind zone are shown on figure 5.4.1. These boundaries enclose stations which exhibit similar characteristics both with regard to maximum speeds and directional variations. The directional variations are based upon observed readings from various stations within each region. It should be noted that in region V, the south westerly, southerly and south easterly winds have been magnified by 10 % to allow for channelling effects known to occur in this region.

The "non-directional" basic wind speeds have been derived by assessing the probability that the peak wind experienced at a site is from a direction other than that of the maximum wind speed.

C5.4.3 *Limit state multiplier*, M_{ls}
The limit state multiplier is the factor applied to the basic regional wind speed to adjust it to the required probability level. In the case of the ultimate limit state multiplier, values of M_{ls}=1.0 result in wind speeds which have a 5 percentile probability of exceedence within 50 years (i.e., a return period of approximately 1000 years). However calibrating wind forces derived using this revision against earlier editions (which have generally resulted in buildings with an acceptable level of performance) it becomes apparent that a return period of approximately 1 in 350 years is appropriate. The M_{ls} values in table 5.4.2 reflect this and produce wind forces which are probabilistically consistent with the ultimate wind forces stipulated in NZS 4203:1984.

The lower value of M_{ls} for the serviceability limit state in Region I reflects the different statistical characteristics of the extreme weather patterns in this region, partially brought about by the influence of tropical cyclones in this area which are not present to the same extent elsewhere in New Zealand. There is a lesser chance of a severe storm system over Northland at low return period.

C5.4.4 *Site terrain/height multiplier*, $M_{(z,cat)}$
The engineering wind model applied for this Standard has been adapted from that prepared by Melbourne [5.3] for Australia, which has been derived from the Deaves and Harris model [5.4]. This model is based on extensive full scale data and on the classic logarithmic law in which the mean velocity profile in strong winds is applicable in non-cyclonic regions (neutral stability conditions) (Ref AS 1170.2).

C5.4.4.1
The terrain/height multiplier, $M_{(z,cat)}$, defines the variation of wind speed with height above the ground. It is a function of both height (z) and the upwind terrain (cat) which determines the drag effects that the wind experiences. The profiles identified in table 5.4.3 apply once the full effect of ground roughness (drag) has developed. The distances over which this occurs have been identified as the depth of the transition zones. Although identified in terms of building height, h, these are but two points on a continuous profile and this method of defining the depth of the transition zone is intended to allow a simplified approach for the designer. A more detailed assessment of the effect of changing the ground roughness, and hence the drag, is outlined in AS 1170.2, Section 4. The application of this more precise method is intended to be an alternative acceptable means of determining the terrain/height multiplier.

The 'probable future changes' are intended to cover reasonable future growth patterns over the next 5 years, such as subdivisional development etc. The designer may use discretion in this assessment.

C5.4.4.2 *Terrain categories*
Four terrain categories have been defined in 5.4.4.2. Figures C5.4.2 to C5.4.4 are pictorial representations of the typical types of terrain intended by this clause. Table C5.4.1 provides a pictorial guide to the intended terrains, and some indication of the transition between terrain categories.

2.0	City buildings (10.0 m to 30.0 m)		TERRAIN CATEGORY 4, z_o = 2.0 m
1.0	Forests		
0.8	High density metropolitan		
0.6			
0.4	Centres of small towns		
0.2	Level wooded country (Low density suburban)		TERRAIN CATEGORY 3, z_o = 0.2 m
0.1			
0.08			
0.06	Few trees Long grass (600 mm)		
0.04	Crops		
	Rough open water surfaces at windspeeds		
0.02	V_u and V_p, isolated trees, uncut grass and airfields		TERRAIN CATEGORY 2, z_o = 0.02 m
0.01			
0.008	Cut grass (10 mm)		
0.006	Desert (stones)		
0.004			
0.002	Natural snow surface (flat)		TERRAIN CATEGORY 1, z_o = 0.002 m

Table C5.4.1 – Roughness length, z_o, and terrain categories as a function of terrain description of an area

NOTES TO TABLE C5.4.1

(1) For sites with open water (sea or lakes) up-wind, the roughness length, z_o , varies with wave height and spray density. A general rule would be from extreme winds (ultimate limit state), $z_o = 0.020$ m (Terrain Category 2) and for less winds (serviceability limit state), $z_o = 0.002$ m (Terrain Category 1). Coastal waters are estimated to develop surface roughness equivalent to at least Terrain Category 2 during a severe storm.

(2) Terrain Category 4 conservatively covers city centres, where high-rise development of buildings and structures occurs (30.0 m to 500.0 m height). The only way of determining true design wind speeds and loadings in such locations is through specific model or full scale studies.

(3) In general, commercial and suburban areas of New Zealand may be considered as Terrain Category 3, and most rural areas as Terrain Category 2.

(4) Design wind speeds and loads may be determined through specific model or full scale studies.

(5) Interpolation for roughness length, z_o, between terrain categories given in table C5.4.1 is permitted according to the equation $z_o = 2 \times 10^{(\text{terrain category} - 4)}$.

Figure C5.4.2 – Typical example of terrain category 2

Figure C5.4.3 – Typical examples of terrain category 3

Figure C5.4.4 – Typical example of terrain category 4

C5.4.5 *Shielding multiplier*, M_s

The shielding multiplier, M_s, has been based on work by Holmes and Best [5.5], Hussain and Lee [5.6] and Lee [5.7] and is a conservative generalization to accommodate effects of total and local wind forces on structures in a range of shielding configurations.

The shielding parameter, s, for typical New Zealand suburbs (Terrain Category 3), is usually in the range from 3 to 6, giving a typical range of M_s from 0.8 to 0.9.

The sector for the determination of the density of shielding is shown in figure C5.4.5.

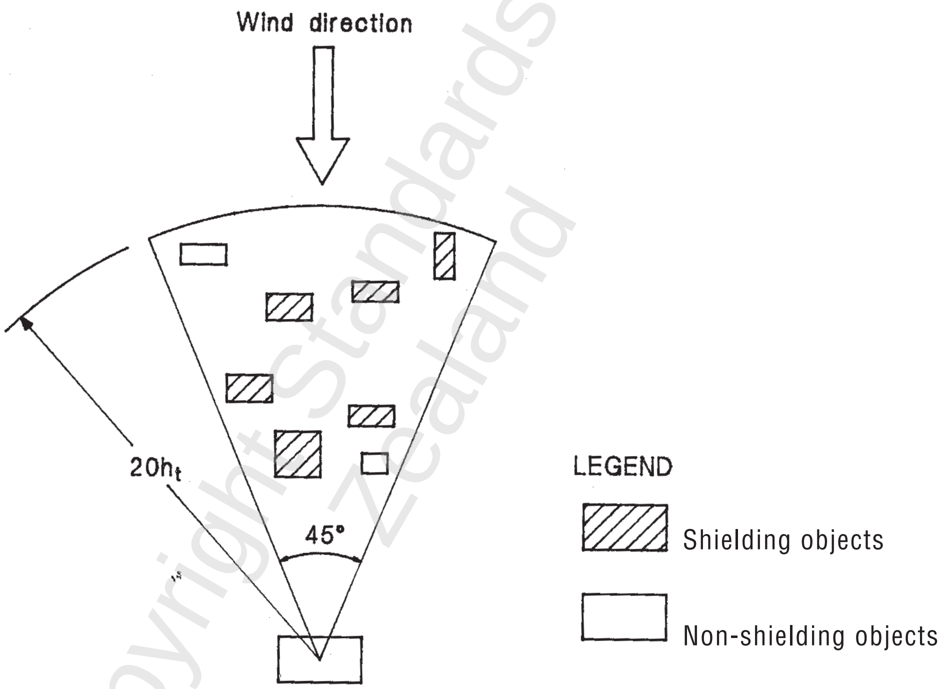


Figure C5.4.5 – Shielding in complex urban situations

For the evaluation of the effective shielding spacing, l_s, equation 5.4.3 gives reasonable values for the cases of regular rows of buildings and of randomly distributed shielding objects within the sector. However the user should beware of ‘corridors’ with no shielding objects immediately upwind of the structure. In such cases the shielding multiplier may increase or may equal 1.0. Shielding objects must be of a permanent nature and be capable of withstanding extreme wind speeds. Buildings, some fences and mature trees may be considered to be shielding objects.

C5.4.6 *Topographic multiplier*, M_t

C5.4.6.1

The topographic multiplier is applied to account for wind accelerations which result from free wind streams being displaced around various land formations such as mountain ranges, hills, escarpments or valleys. On simple terrain, one effect is dominant and it is sufficient to limit the topographic multiplier to the maximum of the most severe of the individual effects. For sites with an elevation above 500 m, wind acceleration effects are complex and will usually require the individual effect to be compounded. An additional enhancement of the basic wind speed with elevation is also required for all sites above this elevation.

C5.4.6.2 *Hill-shape multiplier*, M_h
The hill-shape profile multiplier is applied to account for the acceleration of the wind stream as it passes over a hill or an escarpment. The steepness of the hill profile over the upper half of its section dictates the wind pattern. Such influences extend well beyond the crest (as indicated in figures 5.4.2(a) and (b). In instances where the site is located away from the crest, the distance to the crest, and the gradient of the formation are to be determined by considering horizontal distances in the along-wind direction. The extent of the topographic zone is thereby determined in accordance with the parameters outlined in figures 5.4.2 (a) and (b). The hill-shape multiplier is then determined by interpolating between the crest and the boundary value, first with the horizontal distance of the site from the crest in the direction of the wind, and secondly with elevation from ground level to the upper boundary. It is conservative to ignore this latter interpolation. (Ref. 5.8, 5.9, 5.10 for background direction)

The values of M_h given in table 5.4.5 apply at ground level at the crest. M_h equals 1.0 at and beyond the boundary of the local topographic zones (as indicated in figures 5.4.2 (a) and (b)). The hill shape multiplier for a building of height h, and at a (horizontal) distance x from the crest (but still within the local topographic zone), should be determined by interpolating between the crest value and the boundary value based on x and then by interpolating between this ground elevation value to the upper boundary of the local topographic zone based on h.

C5.4.6.3 *Lee multiplier*, M_L
The lee multiplier is applied to allow for the high wind speeds that occur as a result of orographic effects established as the wind flows over broad, high and relatively long mountain ranges. The orographic effect is the phenomenon where a pattern of standing waves of air flow are established in the upper atmosphere when wind is displaced upwards by wide, high features such as mountain ranges. The standing waves, once established, remain largely stationary and include the high speed winds from the upper elevation wind streams, which are drawn into the wave pattern. Beyond the leeward extent of the range, the wave pattern continues with the various low points of the wave impinging on the ground in localized places over the extent of the lee zone. The wind speeds are most severe close to the ranges, (the shadow lee zone), and reduce in severity over the outer lee zone. Damage to the forests on the Canterbury Plains from north-westerly winds during the 1970's, and at Te Aroha during the 1978 storm are examples of this effect.

In addition to the areas indicated in figure 5.4.1, the topographic characteristics understood to be responsible for generating these effects are present along the main North Island ranges from Gisborne and Opotiki in the north, to the Cook Strait in the south. The Nelson region has similar topographic characteristics for south easterly winds. However there are neither confirmed wind speed readings nor evidence of building damage in these areas to justify their classification as lee zones at this time.

C5.4.6.4 *Channelling multiplier*, M_c
The channelling multiplier, M_c, is applied to allow for the acceleration of the wind as it is channelled by land formations which form valleys and gorges. Clear guidelines as to the magnitude of M_c are difficult to detail. However, where channelling effects are acknowledged to occur, M_c will usually be in the range of 1.1 to 1.2. Examples of areas where this effect is known to occur are Rakaia Gorge (M_c = 1.2), Lake Wakatipu (M_c = 1.15), Milford Sound (M_c = 1.2) and Whangarei Harbour (M_c = 1.1).

Channelling effects are also known to occur in most areas of the Wellington region both because of local channelling and geographic channelling relating to the Cook Strait. As this effect is wide-spread enough to cover nearly all of this relatively narrow wind region, a channelling multiplier of 1.1 is to be applied to the basic wind speed for winds from the north, south and south west, and of 1.2 for the Lyall Bay/Evans Bay isthmus.

Other features which may affect the magnitude of M_c are as follows:

(a) The height and breadth of hills bounding the valley. A valley formed by high hills, with a wide entrance which is exposed to direct wind influx (such as a coastal valley) tends to concentrate the wind streams within the valley, resulting in a higher value of M_c.

(b) The linearity of the valley and the extent that the boundary hills converge. If the hills converge, the contained wind streams become more concentrated resulting in a higher value of M_c. Where the valley is twisting and highly irregular, a significant proportion of the wind stream is spilt from the valley, interrupting the wind flow and thereby reducing the wind speeds.

(c) The channelling multiplier is usually greatest across the floor of the valley. Boundary interference, often initiated by the presence of irregular intrusions such as spurs and other secondary formations, disrupt the adjacent wind flow thereby reducing the channelling effect, resulting in a lower value of M_c.

(d) Valleys which are heavily incised, or have narrow or confined mouths which inhibit the inflow of wind, will not usually be subjected to channelling effect and M_c =1.0.

C5.4.7 *Structure risk multiplier*, M_r

Structure risk multipliers may be applicable for both methods, static analysis and dynamic analysis. Structures designed with lower risk multipliers should be positioned such that, if failure occurs at higher wind speeds, resulting debris will not endanger other structures nearby.

C5.5 Design wind pressure

Dynamic pressure of the wind

In equation 5.5.1, the conversion constant (0.6) which is half the air density at an air density of 1.20 kg/m^3, which is appropriate for air temperatures of approximately 20°C, has been selected for general use.

C5.6 Forces and pressures on enclosed buildings

For low-rise buildings, a detailed background discussion of pressure coefficients is given by Holmes [5.11, 5.12]. More detailed information on monoslope roofs is given by Stathopoulos and Mohammadian [5.13].

The pressure coefficients given in tables 5.6.2(a),(b) and (c) and tables 5.6.3(a),(b) and (c) and in Appendix 5.B are deemed to be related to extremes of wind loads on the wall and roof surfaces of structures of the form illustrated.

It should be noted, that in these tables, a positive pressure denotes pressure towards a surface and a negative pressure denotes suction away from a surface.

The pressure coefficients given in these tables are average values for use in establishing overall wind loads. The values quoted are applicable for sharp-edged rectangular buildings when the wind is blowing normal to one face. Local peak pressures are higher than these average values and the pressure coefficients (with local pressure factors K_l) given in 5.6.3.4 should be used for the determination of forces on windows and cladding elements.

An elevated building is one with a clear unwalled area below the floor, with a height from ground to underside of floor of at least one-third the height to the eaves.

The values given in tables 5.6.2(a), (b) and (c) take into account the effect of the variation of velocity with height on the pressures produced on a tall building which is relatively isolated and exposed within the particular terrain category. It should be noted that some combinations of isolated tall buildings placed together could lead to local and overall increases in the values of the average pressure coefficients given in tables 5.6.2(a),(b) and (c). Under these conditions the appropriate coefficients can be determined only from correctly scaled wind tunnel tests.

The use of q_z varying with height, for windward wall pressures, is more appropriate for buildings of slender form (high aspect ratio). However the user may find the use of q_h, giving a constant windward wall pressure with height, more convenient in most cases of buildings less than 25.0 m in height.

C5.6.2 *Internal pressures*, p_i

Table 5.6.1 allows the determination of internal pressures by a quasi-steady assumption. The validity of this assumption and possible resonant dynamic effects on internal pressure is discussed by Holmes, [5.14].

In determining the most critical loading condition, designers may use their discretion as to which opening can be relied upon to be closed, with closures capable of withstanding peak wind forces, at the critical loading conditions. Possible debris effects may also require attention.

Internal pressures developed within an enclosed structure may be positive or negative depending on the position and size of the openings.

In table 5.6.1 the permeability of a surface is defined as cracks and gaps arising from normal construction tolerances.

As a guide, the typical permeability of an office block or house with all windows nominally closed is between 0.01 % and 0.2 % of the wall area, depending on the degree of draught-proofing.

Industrial and farm buildings can have permeabilities of up to 0.5 % of wall area. Such walls may be considered ‘permeable’ when assessing the internal pressure coefficients. Concrete, concrete masonry or other walls specifically detailed to prevent air passage can be considered ‘non-permeable.’

Openings include open doors and windows, vents for air conditioning and ventilation systems, deliberate gaps in cladding, etc.

A dominant opening is an opening, whose area exceeds the open areas on any other surface. A dominant opening does not need to be large.

The value of C_{pi} can be limited or controlled to advantage by deliberate distribution of permeability in the wall and roof, or by the deliberate provision of a venting device which can serve as a dominant opening at a position having a suitable external pressure coefficient. An example of such is a ridge ventilator on a low-pitch roof, and this, under all directions of wind, can reduce the uplift force on the roof.

For buildings where internal pressurization is utilized, this additional pressure must also be considered.

C5.6.3 *External pressures*, p_e

Combinations of possible positive and negative internal and external pressure coefficients should be considered to obtain the most severe condition for design.

C5.6.3.3 *Area reduction factor*, K_a, *for roofs and side walls*

These factors provide an approximate reduction for the lack of spatial correlation of fluctuating pressures on roofs and side walls, and are for the calculation of loads on the major supporting structure, and on cladding elements, or battens or purlins, etc., to which cladding is directly fixed.

The values used in table 5.6.4 were derived from direct measurements of total roof loads in wind tunnels. (See Davenport, Surry and Stathopoulos, [5.15]; Holmes and Rains, [5.16]; Roy and Holmes, [5.17]; and full scale tests of Kim and Mehta, [5.18]).

C5.6.3.4 *Local pressure factor*, K_l

The local pressure factors, K_l, allow for the pressures on small areas compared with the average increase over the surface in question and especially the suction peaks that occur on small areas near windward corners and roof edges on buildings.

C5.6.3.5 *Porous cladding reduction factor*, K_p

Negative surface pressures on porous cladding are found to be lower than those on a similar non-porous cladding because air flow through the porous surface induces a negative pressure in the internal volume behind the surface. Cheung and Melbourne [5.19] have quantified a number of configurations resulting in the generalized data given in table 5.6.6.

C5.6.4 *Frictional drag force for rectangular enclosed buildings*, F_f

The size of the ribs and corrugations considered are those typically encountered in common corrugated metal roofing profiles.

C5.7 Forces and pressures on canopies, free-standing walls and roof structures

C5.7.2

Net pressures for canopies, awnings and carports, attached to enclosed buildings

The data in table 5.7.1(a) is derived from wind tunnel tests described by Jancauskas and Holmes [5.20] and that in table 5.7.1(b) from Jancauskas and Eddleston [5.21]. The values given in table 5.7.2 is from unpublished data.

The net wind pressure acting on a canopy for a wind direction normal to the wall to which the canopy is attached, depends on the height of the canopy above ground in relation to the height of the adjacent wall, and on the height/width ratio for the canopy. Short canopies at the top of a building experience similar loads to those for overhanging eaves.

For wind directions parallel to the walls of the adjacent buildings, wind loads on the canopy or awning can be obtained from table 5.7.4(a) for monoslope free roofs. Note that in table 5.7.4(a) the net wind pressures are modified where the 'blocked under' condition exists under the canopy.

For the purposes of this clause, 'attached' means canopies and awnings where wind flow is prevented between the roof of the canopy and the face of the building. Such 'attached' links may be non-structural.

C5.7.3 *Net pressures*, P_n, *on free roofs*

The net pressure coefficients for monoslope, pitch and trough free roofs are based mainly on wind tunnel tests described by Gumley [5.22, 5.23]. The roof pitches specified are those for which the tests were carried out. Some adjustment to table 5.7.4(a) has been made based on the full scale measurements by Robertson, Hoxey and Moran [5.24].

C5.7.5

Net pressures, p_n , *on hoardings and free-standing walls*

The main sources of data for the pressure coefficients are wind tunnel studies carried out by Holmes [5.25] and Letchford [5.26].

The loads specified for wind blowing parallel to the hoarding or wall in table 5.7.7(c) are caused by turbulence and unsteady flow effects. In this case, wind loads in both directions must be considered.

When an adjacent wall, with a length of greater than 2 c for hoardings, or greater than 2 h for free-standing walls, runs at right angles to a free end, forming a corner, reduced wind loads may be used for the 45˚ and 90˚ directions. It is suggested, that the values given for 2 c to 4 c, or 2 h to 4 h, be extended up to the windward corner, i.e. they should apply to a distance of 0 to 4 c, or 0 to 4 h from the windward corner.

The pressure coefficients for circular hoardings, closely approximate those of rectangular hoardings with h/b = 1.

C5.8 Forces on exposed structural members

C5.8.2 *Wind forces on individual structural members*

C5.8.2.1 *General*
This method of calculation will give reasonable results for low solidity ratios. For higher solidity ratios, equation C5.8.1 has been found to match experimental data well for sharp-edged rectangular or structural sections.

$$C_d = 1.20 + 0.26(1 - \delta) \qquad \text{(Eq.C5.8.1)}$$

Equation C5.8.1 is due to Georgiou and Vickery [5.27] and is applicable for $0.1 < \delta < 1.0$.

C5.8.2.3 *Shielding factor for multiple open frames*, K_{sh}
The shielding factors given in tables 5.8.1(a) and (b) are derived from the study of Georgiou and Vickery [5.27].

C5.8.4 *Wind forces on multiple open frames*

C5.8.4.3
The effective solidity, δ_e, for circular cross-section members in equation 5.8.4 was derived by Whitbread [5.28] for critical and super-critical flow.

C5.8.5 *Drag forces on lattice towers*
This clause gives overall drag force coefficients for lattice towers. The projected area, A_z, is calculated for the windward face or faces only. The values in tables 5.8.3 to 5.8.5 incorporate the drag forces on the down-wind members shielded by the windward face or faces.

The values in table 5.8.3 for square towers with flat-sided members are based on equations C5.8.2(a) and C5.8.2(b) given by Bayar [5.29].

$$C_d = 4.2 - 7\delta \quad \text{for } 0 < \delta < 0.2 \qquad \text{(Eq.C5.8.2(a))}$$

$$C_d = 3.5 - 3.5\delta \quad \text{for } 0.2 < \delta < 0.5. \qquad \text{(Eq.C5.8.2(b))}$$

More detailed methods for calculating wind forces on lattice towers are given in Refs. [5.30] and [5.31].

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PART 6 PROVISIONS FOR SNOW LOADS, RAIN WATER PONDING LOADS, ICE LOADS, SOIL LOADS, AND GROUND WATER LOADS

C6.1 Notation

C6.1.1

The following symbols appearing in the Commentary to this Part are additional to those used in Part 5 of the Code of practice.

c	Cohesion of a soil (C6.6.1)
c_d	Dependable cohesion (C6.6.1)
f	Internal friction angle of a soil (C6.6.1)
f_d	Dependable internal friction angle (C6.6.1)
M_s	Wind shielding multiplier (C6.3.1, and refer Part 5)
p	Normal stress in a soil (C6.6.1)
s_d	Dependable shear strength of soil (C6.6.1)
w_u	Factored soil weight (Example C6.6)
ϕ	Strength reduction factor (C6.6.1)

C6.2 General requirements

Part 2 specifies requirements which are general to all loads and combinations of loads. It is to be noted that, with the exception of snow load (or rain or ice load as the case may be), loads specified in this Part do not appear in the load combinations. This is because soil and ground water loads are treated as derivatives of other loads (such as dead and live loads, with dead load including the weight of the soil or water).

It is recognized that some designers may wish to treat soil load, for example, as a separate load. Care must be taken, however, in combining the loads to ensure that consistency in loading is achieved.

C6.3 Snow loads

C6.3.1 *Basic roof snow load*

The Snow Zones coincide with 1988 County boundaries in the South Island. the specification of requirements generally follows that used in ANSI/ASCE 7. However, that reference has been drawn up for a continental climate, and many of the requirements are not directly applicable to New Zealand. This Code therefore specifies less severe requirements and simpler load cases.

It is expected that the value of $C_e = 1.0$ will be selected as appropriate for most sites. The minimum value for C_e, corresponding to more exposed sites where the wind can be expected to reduce snow accumulating, occurs at $V_{d(h)}$ = 30 m/s. C_e reaches its maximum value at $V_{d(h)}$ = 10 m/s, corresponding to very sheltered sites. All wind directions should be considered. The value of the shielding multiplier, M_s (see Part 5) should be assessed for conditions likely to exist during the life of the building. Trees should generally be considered as providing shelter only on rural sites, and then only when the stand of trees is likely to remain for the design life of the building. Where the trees are deciduous, M_s should be increased by 10 % to a value not exceeding unity.

Roofs where shedding of snow is prevented, such as for roofs with valleys or high parapets, multiple plate, saw tooth or barrel vault roofs may collect snow within them through drifting due to wind and sliding and creeping of snow. They therefore do not qualify for reduction in snow load applying to other roofs.

The basic snow loads are subject to variation due to wind drifting, and removal by wind and sun. However, extreme variation is not specified except for roofs expected to trap significant quantities of snow.

C6.3.2 *Distribution of snow loads on roofs*
The objective of the load distributions specified is to simplify otherwise rather complex loading patterns while remaining adequately cautious. The loads specified may be assumed to take proper account of drifting of snow due to wind. Loads due to the wind itself need not be combined with the loads due to snow.

Loading on each defined region can largely be treated independently. The most adverse loading should be used for the design of members supporting the region, but, for the design of the overall structure, loads specified in 6.3.2.2 will normally control.

The load specified in 6.3.2.2(a) will normally control the strength design of clear-span structures, and that specified in 6.3.2.2(b) the design for sidesway or the strength design of structures spanning only part of the building width. The use of C_e in this latter loading case recognizes that the exposure will affect the amount of leeward drifting.

Clause 6.3.2.3 specifies loads for cases where snow can be expected to accumulate, slide, and drift. Simple valley forms will produce maximum intensities of load equal to twice the average. For extreme cases of roof planes forming an inverted pyramid, the maximum intensity of load will equal three times the average load intensity for a flat roof.

Where basic snow loads for roofs exceed 1 kPa, problems of sliding of snow onto lower roofs must be examined. It is preferable to avoid these problems through revision of the roof form and layout. Where this is not possible, because of architectural or other constraints, designers should consult reputable authorities, such as BS 6399:Part 3.

C6.4 Rain water ponding loads

C6.4.1
Where secondary drainage such as overflow outlets from gutters is provided, the depth of water which may collect may be assumed to be limited to the resulting maximum feasible depth.

C6.5 Ice loads

C6.5.1
Ice loads are generally critical for slender members such as are used in lattice towers.

C6.5.2
The specific gravity of ice deposits is typically in the range of 0.1 to 0.6 [6.1]. For the purposes of this clause it is recommended that a specific gravity of 0.4 be used.

The reduced wind return period is used because it is considered unlikely that high wind loads will coincide with high ice loads.

C6.5.3
Reference 6.2 records coating thicknesses observed in the United States. The condition described is that of a “rime” coating deposited by small cloud droplets passing a surface which is at a temperature near freezing. Some observations of this phenomenon in New Zealand have been recorded [6.3].

C6.5.4

At altitudes below 1500 m it is considered that the coexistence of ice and earthquake loads is unlikely and their simultaneous application is not required. For structures above 1500 m the ice load specified as coexisting with earthquake load results from the transformation of a snow deposit into an ice layer after a short period of weathering. (This is not the "rime" coating referred to in C6.5.2).

C6.6 Soil loads

C6.6.1

The earth retaining structure and the retained material should be treated as parts of the one structural system in which the resistance to the loads is provided by both parts. In the absence of abnormal forces such as those due to water seepage, the only loads on the system are body forces due to the acceleration of gravity and earthquake, and live loads, usually applied to the surface of the retained material. Consistent with the approach used for other structures, it is these forces which are to be factored, and not the internal stress resultants. Because the soil provides resistance, its ideal strength should be reduced by applying a strength reduction factor.

The simplest way to allow for the strength reduction factor is to modify the soil strength parameters. These are commonly cohesion, c, and internal friction angle, f. The Mohr-Coulomb yield criterion for dependable soil strength, s_d, may then be written as

$s_d = \phi\,(c + p\tan(f))$ .... (Eq. C6.6.1)

where p is the normal stress and ϕ is the strength reduction factor. By employing the suggested modification this may be written as

$s_d = c_d + p\tan(f_d)$ .... (Eq. C6.6.2)

where $c_d = \phi c$, and $f_d = \arctan(\phi.\tan(f))$.

This method allows standard formulae (and graphs and tables derived from them) to be used directly.

EXAMPLE C6.6.1

Figure C6.6.1 shows a cantilever wall retaining cohesionless backfill of unit weight 16 kN/m^3, and with f = 30°. The wall bears on clay for which c = 100 kPa and f = 0°. It is required to find the minimum footing breadth.

For the backfill	f_d = arctan (0.6 tan (30)) = 19.1°.
For the clay	c_d = 0.6 x 100 = 60 kPa.
The factored soil weight is	w_u = 1.4x 16 = 22.4 kN/m^3.

For this example the unit weight of the soil will also be used for the wall and footing.

Considering the soil wedge behind the heel, as shown in figure C6.6.1(b), and using standard procedures, the design forces on the vertical plane through the heel are found.

A breadth of 0.6 h = 2.4 m is tried first. All design forces are shown in figure C6.6.1(c), where it is to be noted that the soil in front of the wall is ignored.

For the loading shown in figure C6.6.1(c), the distance from the toe of the wall to the intersection of the resultant reaction with the base is given by:

$$e = \frac{(215 \times 1.2) + (26 \times 2.4) - (75 \times 4/3)}{215 + 26}$$

= 0.915 m

The resultant is at an angle A from the vertical, where

A = arctan (75/241) = 17.3°.

Meyerhoff's solution for oblique bearing may be written in the form

$\phi R_i = K.(\phi c).(2e)$.

For A = 17.3°, K = 2.89. So the dependable force is 317 kN, which is greater than the actual reaction of 252 kN.

For stability calculations, mechanisms are assumed. Sliding stability is obviously assured because the horizontal force (75 kN) is resisted by the dependable soil strength only. For overturning, the wall will pivot about the centre of bearing. All loads are assumed fixed, so the reaction remains at 252 kN. This means that e = 252/(2.89 x 60 x 2) = 0.73 m.

The geometry of the collapse mechanism is now fixed, and zones of stabilizing and destabilizing forces are as marked in figure C6.6.1(d). Taking moments about the pivot and applying the appropriate factors from 2.5.3.4,

$$\begin{aligned} M^I &= 65 \times 0.73/2 + 54.8 \times 1.72 - 19 \times 1.67 \\ &= 86 \text{ kNm} \\ M^S &= (0.9/1.4) \times (150 \times 1.67/2 + 7 \times 1.67 - 20.2 \times 0.28) \\ &= 84 \text{ kNm} \end{aligned}$$

Thus the trial breadth is marginally unacceptable. A small increase is required to a practical value of 2.45 m.

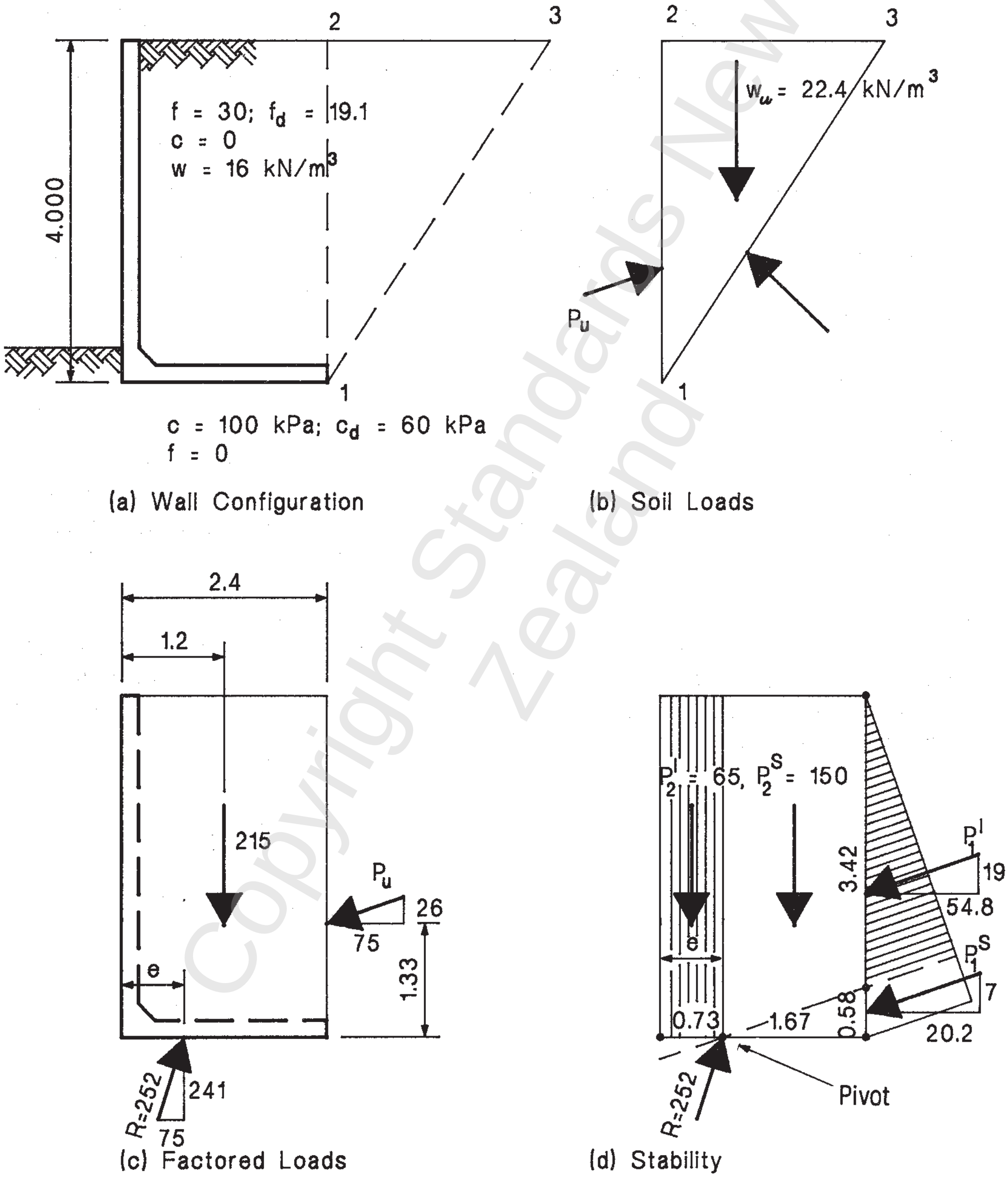

Figure C6.6.1 – Example C6.6.1

C6.6.2
A common load factor for soil and water is appropriate. This also leads to a simple treatment of saturated and submerged soil weight and to a uniform treatment of pore pressure and effective stress, as affecting the soil strength.

C6.6.3
Pending the specification of the strength reduction factors for soil in a geomechanics standard, its value is specified here. Examples of sloping backfill include backfill placed at its natural angle of repose, such as grain or fertilizer stored from conveyor and road fill placed by tipping. In these cases a strength reduction factor of unity is implied since the material must possess a given strength (internal angle of friction, very approximately equal to the angle of repose) to allow placement to the assumed slope. The clause allows for this case and for cases with shallower slopes. It is to be noted, however, that the provisions of 6.6.4 may require that slopes be less steep than the angle of repose.

C6.6.4
An example of where an embankment may pose an unacceptable risk, is above an accessway to an emergency vehicle garage. In circumstances such as this, due care must be exercised to ensure adequate stability. Stability needs to be defined in terms of 2.5.3.4, recognizing that rather ideal conditions given in classical solutions may not always apply. The possibility of saturation and the presence of free water, for instance, need to be considered.

C6.7 Ground water loads

C6.7.1
It may be necessary to make other provisions to safeguard against the occurrence of excessive pressure or flooding. Spring tides have a known period and therefore design for their certain occurrence would be prudent. For soil retaining structures and foundations, effects of ground water need to be considered in conjunction with the effects of soil loads and with soil strength.

C6.7.2
Absence of hydrostatic water pressure may produce more critical conditions than its presence. Design should therefore allow for absence, by treating ground water loads as variant dead loads as specified in this clause, in the same manner as superimposed dead loads are treated elsewhere in the Standard.

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APPENDIX CA

NOTATION

The following summarises, in alphabetical order, the notation employed in all parts of the Commentary to the Code of practice. It is additional to the notation used in the Code of practice itself.

A Angle of the effective earthquake force measured from the X-axis – Part 4

C cos(A) – Part 4

c Cohesion of a soil – Part 6

c_d Dependable cohesion – Part 6

e Eccentricity of the effective earthquake force, measured in a perpendicular direction to the mass centroid – Part 4

F_t Additional equipment static lateral force applied at the top level of a structure – Part 4

f Internal friction angle of a soil – Part 6

f_d Dependable internal friction angle – Part 6

I_o Polar moment of inertia of the level under consideration about the co-ordinate origin – Part 4

K_j Load factor applicable to loads producing component j of the stabilizing moments – Part 2

M Translational mass – Part 4

M^I Moment about the centre of rotation of loads tending to cause instability – Part 2

M^S Moment about the centre of rotation, of the loads tending to resist instability, and scaled to a load factor of 0.9 – Part 2

M^S_j Component j of the stabilizing moments, M^S – Part 2

M_s Wind shielding multiplier – Part 6

p Normal stress in a soil – Part 6

S sin(A) – Part 4

s_d Dependable shear strength of soil – Part 6

T_1 Fundamental period of the building vibrating predominantly in translation – Part 2

w_u Factored soil weight – Part 6

X,Y Coordinates of the mass centroid from the origin – Part 4

ϕ Strength reduction factor – Part 6

NOTES

NZS 4203:1992

Code of practice for
GENERAL STRUCTURAL DESIGN
AND DESIGN LOADINGS FOR BUILDINGS

Volume 1 Code of Practice

CORRIGENDA

January 1994

Figure 4.6.1(a) (page 41)
Delete % sign on vertical axis of graph.

Figure 4.6.1(b) (page 42)
Delete % sign on vertical axis of graph.

Figure 4.6.1(c) (page 43)
Delete % sign on vertical axis of graph.

Figure 5.6.1 (page 75)
Top right hand diagram
Delete "$\theta = 0°$" and **replace** with "$\theta = 90°$".

Figure 5.7.6 (b) (page 88)
Delete dimension "c" and **replace** with "h".

Figure 5.B4.2 (page 109)
Under heading "Force coefficients" **delete** "$C_{F,x}$" and **replace** with "$C_{F,y}$".

Volume 2 Commentary
Clause **C5.3.1** (page 67)
In example C5.3.1 **delete** line "Importance multiplier $M_i = 1.0$".

Clause **C5.4.2.1** (page 73)
Delete last sentence of first paragraph:
"It should be noted that in region V, the south westerly, southerly and south easterly winds have been magnified by 10 % to allow for channelling effects known to occur in this region".

STANDARDS NEW ZEALAND
PRIVATE BAG 2439
WELLINGTON 6020

NOTES

Approved by the Standards Council on 16 October 1992 to be a New Zealand standard pursuant to the provisions of section 10 of the Standards Act 1988.

First published: 14 December 1992

Reprinted with Corrigenda appended: 31 January 1994

Reprinted: 13 May 2015

The following references relate to this standard:

Project No. P 4203

Draft for comment No. DZ 4203

Typeset by: Standards New Zealand

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