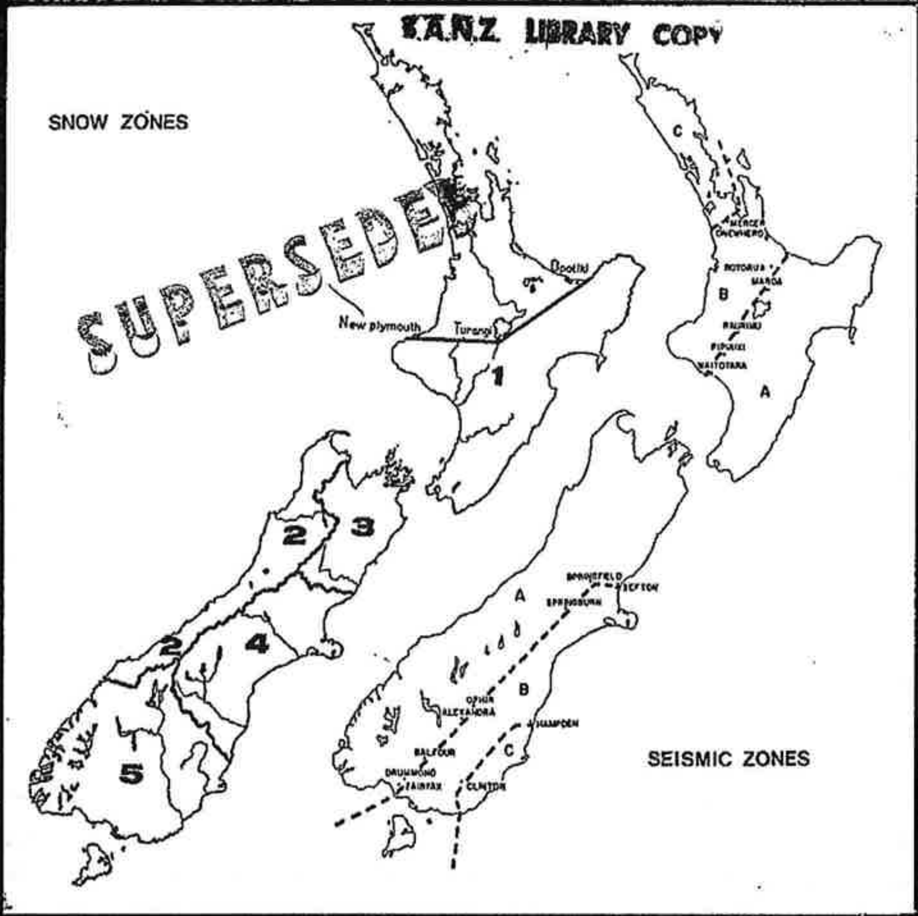
NZS 4203:1984 LOADINGS CODE

SANZ



NEW ZEALAND STANDARD

Standards Association of New Zealand

4203: 1984

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Declared on 26 October 1984 by the Standards Council to be a standard specification pursuant to the provisions of section 23 of the Standards Act 1965.

First published	February 1976
Reprinted incorporating Amendments No. 1 and 2	March 1980
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Note to 1984 edition
Because of the extent of Amendment No. 3 it has been incorporated into a 1984 edition rather than being made available as a separate amendment.
Text, tables and diagrams that have been amended from the 1980 edition are identified by a vertical line in the margin.

The following SANZ references relate to this standard:

Project No. P.973
Draft for comment No. DZ 4203
Printing code: 5M-1984/7032/20191

AMENDMENTS

No.	*Date of issue*	*Description*	*Entered by, and date*
1	Feb. 1977	Corrects typographical errors and omissions.	
2	Dec. 1979	Makes various technical clarifications and amendments and corrects typographical errors and omissions.	
3	Dec. 1984	Corrects typographical errors and omissions and inserts revised material for technical clarification.	
Erratum	Feb 1985		

Tolan Printing Co. Ltd., Wellington

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NEW ZEALAND STANDARD

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

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DECEMBER 1984

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PART 4 WIND LOADS

APPENDIX

COMMITTEE REPRESENTATION

This standard was commenced under the supervision of the Building Bylaw Sectional Committee and completed under the supervision of the Building and Civil Engineering Sectional Committee (38/–) for the Standards Council, established under the Standards Act 1965. The Building and Civil Engineering Sectional Committee consisted of representatives of the following:

*Building Research Association of New Zealand
*Department of Scientific and Industrial Research
Housing Corporation of New Zealand
*Ministry of Works and Development
*Municipal Association of New Zealand
New Zealand Contractors Federation
New Zealand Counties Association
*New Zealand Institute of Architects
*New Zealand Institution of Engineers
New Zealand Manufacturers Federation
New Zealand Master Builders Federation

The Loadings Committee (38/11) was responsible for the preparation of this standard and consisted of representatives of the following organizations in addition to those marked with an asterisk (*) above:

Association of Consulting Engineers
New Zealand Meteorological Service
University of Auckland
University of Canterbury.

Amendment No. 3 was prepared under the direction of the Building and Civil Engineering Divisional Committee (30/-) by the Loadings Code Amendments Committee (38/11). In addition to the representation shown above, the New Zealand Society for Earthquake Engineering and the New Zealand Master Builders Federation are now represented on this committee.

RELATED DOCUMENTS (see also Appendix D)

Reference is made in this document to the following:

NEW ZEALAND STANDARDS

NZS 1900 :	*Model building bylaw*
NZS 3101 : 1982	*Design of concrete structures*
NZS 4205P:1973	*Code of practice for design of foundations for buildings*
NZS 6502 : 1972	*Metrication factors and tables for conversion to SI units*
MP 3801 : 1972	*A guide to the adoption of the model building bylaw (NZS 1900) by local authorities using the standard adoption and annual updating procedure*

BRITISH STANDARDS

BSCP 3	*Basic data for the design of buildings: Chapter V, Loadings: Part 2, Wind loads*
BS 648 : 1964	*Schedule of weights of building materials*
BS 4076 : 1978	*Steel chimneys*

AUSTRALIAN STANDARD

AS 1170	*Rules for minimum design loads on structures* (SAA Loading Code): *Part 1 – 1971 Dead and live loads;* *Part 2 – 1975 Wind forces.*

OTHER DOCUMENTS

ACI–318–83	*ACI Standard Building Code: Requirements for reinforced concrete* – 1983. American Concrete Institute
AISC–1969	*Specification for the design, fabrication and erection of structural steel for buildings*, American Institute of Steel Construction
UBC–1982	*Uniform building code*, International Conference of Building Officials
ISO 4356–1977	*Bases for the design of structures – Deformations of buildings at the serviceability limit states*
NBC–1980	National Building Code of Canada, the Supplement, Chapter 4, National Research Council of Canada

FOREWORD

General

This standard is a revision, in the means-of-compliance format and using SI units, of NZS 1900* : Chapter 8 : 1965. It aims at setting down minimum requirements for the general run of buildings rather than for special structures (such as bridges, towers, dams, major storage tanks, or special industrial equipment) for which the provisions of this standard may be taken only as a general guide to be supplemented by special studies and judgment.

The Loadings Committee's task in drafting this standard was seen mainly to be one of providing a set of minimum design criteria of an effective and economic nature which would not be too difficult for the designer to apply, but at the same time would leave him scope for innovation and imagination.

The committee believes that the requirements of this standard provide a reasonable level of protection to life and property at an economic level of cost, taking into account the relative seismicity of New Zealand as compared with the rest of the world and the particular building practice and design methods adopted in this country.

General structural design

NZS 1900* : Chapter 8 : 1965 was based on the "working stress" method of design, which is called "the alternative method" in this standard to emphasise that the strength method is preferred.

For the strength method, the load factors and load equations have been derived from ACI-318-71: ACI *Standard Building Code: Requirements for reinforced concrete — 1971.* A load combination probability factor of 0.75 has been applied to load combinations involving dead and live loads and wind or earthquake while for dead and snow loads and wind a factor of 0.85 has been used. For both snow and wind the design loads are based upon return periods of 50 years.

For the alternative method, a significant change is the inclusion of equations to cover reversal of load under wind and earthquake where only dead load is available to stabilize the members. As E is now calculated as a design load for strength design a load factor of 0.8 is necessary.

Dead, live, and snow loads

Live loads have been set out for various types of building use so that the decision as to which loads are applicable may be made more easily.

The levels of live load have been based on BSCP 3 : Chapter V*, AS 1170 : Part 1*, and NZS 1900* : Chapter 8 : 1965.

The procedures for obtaining reduced live load (L_R) are based on those of the National Building Code of Canada*.

The section on snow loads is based on recent work at the New Zealand Agricultural Engineering Institute and information supplied by the New Zealand Meteorological Service. A distinct departure from overseas practice is the use of the "open field snow load" as the basic design load.

Earthquake provisions

Although New Zealand has suffered several major earthquakes since the last revision of NZS 1900* : Chapter 8 in 1965, these did not produce direct local evidence as to the degree of effectiveness of the applicable requirements. However, evidence from recent earthquakes in other countries, in particular Caracas (1967), Tokachi-oki (1968), and San Fernando (1971), has assisted the committee with the present document.

No evidence was available to cause any change in the basic level of seismic coefficients for ductile structures, and it is believed that those previously chosen should in general be left unaltered until shown to be inadequate or excessive by service experience under earthquake attack on buildings detailed for ductility. However, analysis indicates that

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* See list of related documents.

short-period buildings may be subjected to very high ductility demands, which can be reduced by adopting higher seismic coefficients. Such high ductility demands may have contributed to the damage during the Tokachi-oki earthquake.

Significantly increased loadings are specified by this standard for structures required to dissipate seismic energy in a manner other than by ductile flexural yielding. In addition, the general recommendations made in the previous code for soil types have been formulated into specific requirements.

This standard introduces the concept of a multi-term evaluation of the horizontal seismic design coefficient to be applied to various structures. One of these terms relates to the importance of structures and to the degree to which structures are required to be functional following earthquakes. The need for such a requirement has been clearly demonstrated in recent overseas earthquakes, and similar provisions exist or are proposed in the codes of several countries. Comment on the other terms is given in the standard. The committee believes that at this stage of development in earthquake engineering there is a lack of adequate data that would allow complete separation of all the factors influencing the appropriate level of the design coefficient, and therefore such effects as damping, dynamic amplification, and dutcility are incorporated in the given terms.

An important new provision is the requirement that buildings designed for flexural ductile yielding or for yielding in diagonal braces are to be the subject of capacity design. This requirement includes consideration of the concurrence of earthquake components at right angles to one another, and hence the effect of beam hinges forming simultaneously in all beams framing into a column for a number of adjacent storeys. Although the levels of seismic coefficient for regular buildings have not in general been altered, these levels are considered satisfactory only where the relevant design standards provide an acceptable degree of ductility.

Designers should recognize that the precise properties of construction materials and of structural elements made from them are not clearly known. Furthermore, the interaction of these elements in a building frame under load is extremely uncertain, so that the total design technique is one of some degree of imprecision. In fact, the design result depends so much on the nature of the mathematical model of the building as envisaged by the designer that the use of more advanced techniques of earthquake analysis can easily lose validity.

Dynamic analysis may provide useful information on performance of buildings both in the elastic and inelastic range. To apply these methods, design earthquakes must be chosen and assumptions must be made about damping and inelastic behaviour. However, to avoid large differences in strengths of structures which may result from varying assumptions, it has been considered necessary to relate the requirements found from dynamic analysis to those of the equivalent static force analysis. An important change for spectral modal analysis is that a design spectrum is prescribed which is scaled from the spectrum for the equivalent static force method so that no direct assumptions need to be made for damping and inelastic behaviour.

The principle of multi-term evaluation has also been applied to parts and portions of buildings. This has resulted in design coefficients which may vary from the 1965 provisions. It is believed that these reflect more closely the actual response and result in better performance.

The committee considers that the 1965 provisions concerning "non-structural" damage are inadequate and could result in structural damage and loss of life. Non-structural damage typically represents the greatest monetary loss in an earthquake. There is considerable evidence that modern buildings have low damping and sustain large amplification even in small-to-moderate earthquakes. The separation and interstorey deflection provisions of this standard recognize this.

Further information on the approach adopted in the drafting of the earthquake provisions is given in the March 1976 issue of the Bulletin of the New Zealand National Society for Earthquake Engineering (Vol. 9, No. 1).

Wind loads

Wind loading is now treated in more detail because changes in building design and construction have, for some types of buildings, increased the influence of wind loading in relation to other imposed loads.

An important change is the adoption of gust loadings as the basis for design, in place of the mean load averaged over one minute that was the basis hitherto. There is evidence that this change will lead to a more realistic assessment of the wind loads. At the same time the opportunity has been taken to assess the probable wind speeds on a statistical basis in accordance with the current advice of the New Zealand Meteorological Service.

Note to 1984 edition

This edition incorporates Amendment No. 3. Among the Amendment's more significant contributions is an upgrading of the section dealing with earthquake provisions. It also irons out any parts of the Loadings Code that happened to conflict with the various materials codes, in particular the newly-issued concrete code, NZS 3101 *The design of concrete structures.*

Rather than merely issuing an amendment slip, it was decided the extent of Amendment No. 3 warranted a reprint of NZS 4203.

NEW ZEALAND STANDARD

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

PART 1 GENERAL

C1.1 This standard aims at setting down minimum requirements for the general run of buildings rather than for special structures (such as bridges, towers, dams, major storage tanks, or special industrial equipment) for which the provisions of this standard may be taken only as a general guide to be supplemented by special studies and judgment.

This standard is divided into four parts. The major subdivisions within parts are referred to as "sections", and all subsequent subdivisions are referred to as clauses. Reference to a particular part includes all sections within that part, reference to a particular section includes all clauses within that section, and reference to a particular clause includes all other clauses subordinate to it.

Pending the revision of various other New Zealand standards, this standard should be regarded as the "master document" with other standards, where appropriate, subject to it. See clauses 3.3.1.1 and C3.3 for a particular application of this general principle. An exception, however, is clause 5.14 of Chapter 5 of NZS 1900, which relates to the lateral stability of fire rated external walls. The possibility that severe earthquake or wind loads might occur in the interval between a damaging fire and remedial work is small compared to the possibility that horizontal loads might be imposed during a fire by, say, the collapse of the roof structure. Accordingly, pending the revision of Chapter 5, the design loads previously used for clause 5.14 need not be increased.*

C1.1.2.2 Figures and tables appear in the most convenient position regardless of the general distinction between right-hand (mandatory) and left-hand (commentary) columns.

C1.1.2.3 The date at which an amendment or superseding standard is regarded as "current" is a matter of law depending upon the particular method by which this standard becomes legally enforceable in the case concerned. In general, if this is by contract the relevant date is the date on which the contract is created, but if it is by Act, regulation, or bylaw then the relevant date is that on which the Act, regulation, or bylaw is promulgated; for bylaws, promulgation includes updating by the procedure set out in MP 3801.*

1.1 SCOPE AND INTERPRETATION

1.1.1 Scope

1.1.1.1 This standard sets out requirements for general structural design (as distinct from detailed design appropriate to particular construction materials) and design loadings for buildings, and is approved as a means of compliance with the relevant requirements of NZS 1900*.

1.1.1.2 A special study shall be made for any building which, in the opinion of the Engineer, is sufficiently unusual for the provisions of this standard to be appropriate only as a general guide.

1.1.2 Interpretation

1.1.2.1 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.

1.1.2.2 Subject to clause 1.1.2.1, clauses prefixed by "C" are intended as comments on the corresponding mandatory clauses.

1.1.2.3 Where any other standard named in this standard has been declared or endorsed in terms of the Standards Act 1965, then:

(a) Reference to the named standard shall be taken to include any current amendments declared or endorsed in terms of the Standards Act 1965; *or*

(b) Reference to the named standard shall be read as reference to any standard currently declared or endorsed in terms of the Standards Act 1965 as superseding the named standard, including any current amendments to the superseding standard declared or endorsed in terms of the Standards Act 1965.

*See list of related documents.

1.1.3 Definitions

1.1.3.1 In this standard, unless inconsistent with the context, and subject to clause 4.1.2.1:

ADEQUATE DUCTILITY see section 3.2.

ADEQUATE REDUNDANCY see clause 3.3.3.5.1.

ANALYSIS:

EQUIVALENT STATIC FORCE ANALYSIS means a method of analysis using static forces to simulate the effects of earthquake ground motion.

SPECTRAL MODAL ANALYSIS means a method of dynamic analysis in which a given earthquake design spectrum is applied to a building and the response of several modes is determined.

NUMERICAL INTEGRATION RESPONSE ANALYSIS means a method of dynamic analysis in which a mathematical model of the structure is subjected to time histories of actual or synthetic earthquakes.

ATTIC means a roof space that is not used for business, storage or habitation.

DEAD LOAD means the weight of all permanent components of a building including walls, partitions, columns, floors, roofs, finishes and fixed plant and fittings that are an integral part of the structure.

DESIGN means the use of rational computational or experimental methods in accordance with the established principles of structural mechanics.

DIAPHRAGM means a member composed of a web (such as a floor or roof slab), or a truss which distributes forces to the horizontal force resisting system.

DUCTILE CANTILEVER SHEAR WALL means a shear wall complying with clause 3.3.4.2.

DUCTILE COUPLED SHEAR WALL means a wall complying with clause 3.3.4.1.

DUCTILITY means the ability of the building or member to undergo repeated and reversing inelastic deflections beyond the point of first yield while maintaining a substantial proportion of its initial maximum load carrying capacity.

DUCTILITY FACTOR:

MEMBER DISPLACEMENT DUCTILITY FACTOR means the ratio of acceptable transverse deflection of a member to its transverse deflection at first yield.

SECTION CURVATURE DUCTILITY FACTOR means the ratio of acceptable curvature at any cross-section of a member to that at first yield.

ELEMENTS includes primary and secondary elements.

PRIMARY ELEMENTS means elements forming part of the basic load resisting structure, such as beams, columns, diaphragms, or shear walls necessary for the building's survival when subjected to the specified loadings.

SECONDARY ELEMENTS means elements such as partition walls, panels, or veneers not necessary for the survival of the building as a whole but subject to stresses due to loadings applied directly to them or to stresses induced by the deformations of the primary elements.

FRAME means a system composed of interconnected members functioning as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

DUCTILE FRAME means a frame complying with clause 3.3.3.

MOMENT RESISTING FRAME means a load carrying frame in which the members and joints are capable of resisting horizontal forces by bending moments.

HORIZONTAL FORCE RESISTING SYSTEM means that part of the structural system to which the horizontal forces prescribed by this code of practice are assigned.

LIVE LOAD means the load assumed or known to result from the occupancy or use of a building and includes the loads on floors, loads on roofs other than wind or snow, loads on balustrades and loads from movable goods, machinery, and plant that are not an integral part of the structure and may be changed during the life of the building with a resultant change in floor or roof loading.

MATERIAL CODE means the relevant New Zealand standard or other approved document setting down requirements for detailed design appropriate to a particular construction material (as distinct from requirements for general structural design as given in this standard).

OVERTURNING MOMENT at any given level means the moment of the horizontal forces acting on the structure as a whole above that level.

SET-BACK means any offset horizontally in from the plane of an exterior wall of a structure.

SHEAR WALL means a wall designed to resist horizontal forces in the plane of the wall.

C1.1.3.2 See in particular the definitions of "approved" and "building" in Chapter 1 of NZS 1900.*

1.1.3.2 Unless inconsistent with the context, and subject to clauses 1.1.3.1 and 4.1.2.1, terms defined in NZS 1900* shall have the same meaning in this standard.

1.1.4 Symbols

C1.1.4.1 For the symbols used in Part 4 see clause 4.1.3.

1.1.4.1 In Parts 1, 2 and 3 of this standard, symbols shall have the following meanings, provided that other symbols, or other meanings for symbols listed below, that are defined immediately adjacent to formulae or diagrams shall apply in relation to those formulae or diagrams only:

A design load for the alternative (working stress) design method (in Part 1): beam wall moment parameter used in the specification of S for coupled shear walls (in Part 3).

b length of the building perpendicular to the horizontal loading direction under consideration used when computing building eccentricity.

C basic seismic coefficient.

C_d seismic design coefficient.

C_p seismic design coefficient for a building part.

$C_{p\ max.}$ $C_{p\ min.}$ coefficients used in expressions which limit the range of C_p (note that $C_{p\ max.}$ and $C_{p\ min.}$ do not themselves directly limit C_p).

D dead loads or their related internal moments and forces.

E earthquake loads or their related internal moments and forces.

F liquid pressures or their related internal moments and forces.

F_p the horizontal force on a part or portion of a building.

F_x horizontal force in the direction under consideration that is applied to the level designated as x.

h_{cg} height of the centre of gravity of W_t.

h_n height to the top of the main portion of the building.

h_x height to the level designated as x.

K factor by which the values of C are scaled to give the spectrum to be used for the spectral modal analysis of a particular building.

K_{cp} factor used in computing deflections allowing for long term effects.

K_x local seismic force factor.

L live loads or their related internal moments and forces.

L_c basic minimum concentrated live load.

L_u basic minimum uniformly distributed live load.

L_R reduced live loads.

M structural material factor.

M_p structural material factor for a part or portion of a building.

P prestressing force after losses or the related internal moments and forces.

* See list of related documents.

Q earth pressures or their related internal moments and forces.

R factor related to the relative direct risk to life of a failure and to the relative importance to the community of the survival of particular categories of building.

R_p risk factor for a part or portion of a building.

S snow loads or their related internal moments and forces (in Part 1) structural type factor (in Part 3).

S_p the structural type factor for a part or portion of a building.

T fundamental period of vibration of the building in the direction under consideration, in seconds.

U design load for the strength method.

V total horizontal seismic force or shear at the base in the direction under consideration.

W wind loads or their related internal moments and forces.

W_p seismic load for a part or portion of a building (determined in the same manner at W_t).

W_t total reduced gravity load above the level of imposed lateral ground restraint.

W_x that portion of W_t that is located at or assigned to the level designated as x.

Z seismic zone factor — function of wall aspect ratio.

α coefficient used in expressions which limit the range of C_p.

Δ the deflection used for calculating T.

ν modification factor for computed earthquake deformation.

ϕ strength reduction factor.

1.2 GENERAL DESIGN REQUIREMENTS

1.2.1 Methods of analysis

C1.2.1.1 For definition of "design" see clause 1.1.3.1.

1.2.1.1 Except as provided by clause 1.2.2, buildings and parts of buildings shall be designed.

C1.2.1.2 Arrangements of structural elements requiring double design should be minimized and approached with caution, for example, the effect of a structural wall extending part height between columns acting in conjunction with a series of more flexible frames cannot be assessed by separate double design of frame and walls. The presence of the wall will attract high shears to the shortened columns in the bay. Similarly, the presence of stiff mezzanine beams may radically alter the ductility of a building.

1.2.1.2 When the effect of any element on the structural behaviour of the building cannot be assessed with confidence, then that element shall not be considered as contributing to the basic load-resisting structure of the building, but the effect on the element itself of the design loadings shall be assessed, and allowance shall be made for the effect of the element on the distribution of loads and on building ductility.

1.2.2 Test loads

1.2.2.1 Buildings or parts of buildings not fully amenable to analysis and design may be test loaded to demonstrate that the construction is adequate for its intended purpose.

1.2.3 Design methods

1.2.3.1 In the design of structures, members shall be proportioned for adequate strength in accordance with the appropriate New Zealand standard.

1.2.3.2 For the strength method the load combinations given by clause 1.3.2 shall be used.

1.2.3.3 Where the alternative method is permitted the load combinations given by clause 1.3.3 shall be used.

C1.2.3.3 The term "alternative method" is preferred to "working stress design" since for materials, such as reinforced concrete, exhibiting creep properties a true working stress design method has not been in use for many years (for example in column design).

1.2.4 Strength reduction factors

1.2.4.1 The strength reduction factor (ϕ) shall provide for the possibility that small adverse variations in material strengths, workmanship, dimensions, control, and degree of supervision, although individually within required tolerances and the limits of good practice, may occasionally combine to result in reduced strength.

C1.2.4.1 In some materials codes the strength reduction factor includes also an allowance for member importance.

1.2.4.2 Values of ϕ as given in the relevant material code for the structural element concerned shall not be exceeded. When values of ϕ are not given, then ϕ shall be taken as 1.0.

1.2.5 Distribution of horizontal forces

1.2.5.1 The total horizontal force at any level imposed by wind or earthquake loads shall be considered to be resisted by the various resisting elements in proportion to their stiffnesses, considering the stiffness of the horizontal bracing systems or diaphragms as well as the stiffnesses of the vertical resisting elements and their foundations.

C1.2.5.1 In practice the determination of relative stiffnesses is fraught with difficulties. The effect of foundation deformations can be considerable and should be minimised by aiming for geometrical similarity of resisting elements. There is uncertainty with regard to strain distribution in deep-membered reinforced concrete elements. Yielding and cracking affects the stiffness of geometrically different resisting elements to a different degree. Some redistribution of seismic horizontal forces between elements is therefore acceptable.

1.2.6 Stability

1.2.6.1 All buildings and parts of buildings shall be designed against the adverse effects arising from uplift or overturning moments.

1.3 STRENGTH AND DEFORMATION REQUIREMENTS

1.3.1 Design loads

1.3.1.1 Structures shall be designed to resist all applicable loads as specified in this standard.

1.3.2 Design load combinations: Strength method

1.3.2.1 Structures and members designed by the strength method shall be designed to resist the loading combinations specified in clause 1.3.2.3 as applicable, except that none of: wind load, roof live load, earthquake load, and other transient dynamic effects, need be combined with each other.

C1.3.2.2 In determining the maximum load effects due to live load and earthquake in continuous beams and the like, it is acceptable to assume that all spans carry reduced live loads, or that there is no live load, whichever combination produces the greater strength demand.

C1.3.2.3 The word 'loads' is used loosely and is here employed as a general term standing for either loads or member internal actions such as bending moments and shear forces. The fact that different load factors are used for dead and live loads reflects the greater certainty with which the dead load is known. The values of live load given in table 2 could have been increased by a factor of (1.7/1.4) so as to allow a factor of 1.4 to be applied to both dead and live load in equation 1, but it was decided to keep the live loads at generally familiar levels and use separate load factors instead.

Equation 2 is derived from

$$U = 3/4\,(1.4\,D + 1.7\,L_R + 1.7\,W)$$

and equation 4 is derived from

$$U = 3/4\,(1.4\,D + 1.7\,L_R) + E$$

but terms 1.05 D and 1.27 L_R have been rounded off to 1.0 D and 1.3 L_R respectively.

Although significant vertical acceleration components of ground motions have been recorded during earthquakes (for example 0.2 to 0.3 g in the 1971 San Fernando earthquake) no vertical acceleration load terms have been included in the design loads of this standard except for parts such as horizontal cantilevers and anchorage of machinery because there is at present no certainty about the damage potential of combined dynamic effects.

In New Zealand, snow loads are of short duration at low altitudes, so that the probability of a severe earthquake coinciding with a full snow load is remote. Accordingly, equation 7A applies only to buildings at comparatively high altitudes.

1.3.2.2 Except for load cases involving earthquake, the most adverse distribution of live loads shall be considered. For seismic loading cases, the reduced live load may be considered to be applied uniformly over all relevant floor areas.

1.3.2.3 The design loads U for the strength method shall be not less than whichever of the following load combinations is applicable and gives the greatest effect:

	$U = 1.4D + 1.7L_R$	(1)
with wind	$U = 1.0D + 1.3L_R + 1.3W$	(2)
	$U = 0.9D + 1.3W$	(3)
with earthquake	$U = 1.0D + 1.3L_R + E$	(4)
	$U = 0.9D + E$	(5)
with snow	$U = 1.4D + 1.4S$	(6)
	$U = 1.2D + 1.2S + 1.1W$	(7)
with snow and at altitudes exceeding 1500 m in snow zone 1 and 1000 m in snow zones 2, 3, 4, and 5.	$U = D + S + E$	(7A)
with earth pressure	$U = 1.4D + 1.7L_R + 1.7Q$	(8)
	$U = 0.9D + 1.7Q$	(9)
with liquid pressure	$U = 1.4D + 1.7L_R + 1.4F$	(10)
	$U = 0.9D + 1.4F$	(11)

1.3.2.4 When forces are included that result from contained liquids or solids, filled to maximum capacity, the load factors for contents shall be those applicable to dead loads and not those applicable to live loads or reduced live loads.

1.3.2.5 Impact effect, if any, shall be included with the live load L.

1.3.2.6 Where the structural effects of differential settlement, creep, shrinkage, or temperature change might be significant, realistic service loads only may be used in assessing deformations, crack widths, or other forms of local damage.

C1.3.3 Previously, the alternative method was generally referred to as the working stress method. Use of the alternative method for any loading case involving earthquake loads is provided for in some materials codes.

1.3.3 Design load combinations: Alternative method

1.3.3.1 Where the alternative method is permitted, structures and members designed by the alternative method shall be designed in accordance with the allowable stresses given in the relevant material code to resist the loading combinations specified in clause 1.3.3.3 as applicable, except that none of: wind load, roof live load, earthquake load, and other transient dynamic effects, need be combined with each other.

C1.3.3.2 In determining the maximum load effects due to live load and earthquake in continuous beams and the like, it is acceptable to assume that all spans carry reduced live loads, or that there is no live load, whichever combinations produces the greater strength demand.

C1.3.3.3 In New Zealand, snow loads are of short duration at low altitudes, so that the probability of a severe earthquake coinciding with full snow load is remote. Accordingly, equation 18A applies only to buildings at comparatively high altitudes.

C1.4 See also clauses 2.2.2.5 (ponding) and 3.8 (deformation due to earthquake loads).

C1.4.1 Clause 1.4.1 applies whether or not the parts are tied or interconnected.

C1.4.2 Deformations should be appropriate to the location, loading, and function of the building or component concerned, and should be estimated by analysis based

1.3.3.2 Except for load cases involving earthquake, the most adverse distribution of live loads shall be considered. For seismic loading cases, the reduced live load may be considered to be applied uniformly over all relevant floor areas.

1.3.3.3 The design loads A for the alternative method shall be not less than whichever of the following load combinations is applicable and gives the greatest effect:

$$A = D + L_R \quad \ldots\ldots (12)$$

with wind

$$A = D + L_R + W \quad \ldots\ldots (13)$$
$$A = 0.7D + W \quad \ldots\ldots (14)$$

with earthquake

$$A = D + L_R + 0.8E \quad \ldots\ldots (15)$$
$$A = 0.7D + 0.8E \quad \ldots\ldots (16)$$

with snow

$$A = D + S \quad \ldots\ldots (17)$$
$$A = D + S + W \quad \ldots\ldots (18)$$

with snow and at altitudes exceeding 1500 m in snow zone 1 and 1000 m in snow zones 2, 3, 4, and 5.

$$A = D + 0.8S + 0.8E \quad \ldots\ldots (18A)$$

with earth pressure

$$A = D + L_R + Q \quad \ldots\ldots (19)$$
$$A = D + Q \quad \ldots\ldots (20)$$

with liquid pressure

$$A = D + L_R + F \quad \ldots\ldots (21)$$
$$A = D + F \quad \ldots\ldots (22)$$

1.3.3.4 When forces are included that result from contained liquids or solids, filled to maximum capacity, the load factors for contents shall be those applicable to dead loads and not those applicable to live loads or reduced live loads.

1.3.3.5 Impact effect, if any, shall be included in the live load L.

1.3.3.6 Where the structural effects of differential settlement, creep, shrinkage, or temperature change may be significant this shall be included with the dead load D in equations 12 to 22 inclusive. Estimation of these effects shall be based on a realistic in-service assessment.

1.4 BUILDING DEFORMATION

1.4.1 Provision shall be made for the effects of relative movement due to forces, and for the structural effects of differential settlement, creep, vibrations, shrinkage or temperature change on buildings or parts of buildings. Structural members shall be designed for acceptable deflections and vibrations under service loads.

1.4.2 Under the most adverse loading conditions, other than earthquakes, the deflections of the structure as a whole, and of any of its parts, shall not be such as to impair *on reasonably accurate assumptions as to the actual response to load or by full-scale or model tests.*

Recommended maximum deformations are listed in table 1 for the guidance of designers; these deformations should be acceptable for most conditions but might not be adequate for such things as special structures supporting machinery or floors supporting sensitive instruments. Furthermore, deflections should be checked to ensure that they do not result in damage to non-structural elements, for example with masonry or precast panels between concrete floors, and do not affect the weatherproof qualities of the building. (See also Part 4.)

The deflections given are not intended to guard against ponding or dead load deflections. For instability of steel roof systems due to ponding refer to AISC 1969†, clause 1.13.3.

Pending the specification of values for K_{cp} in the appropriate New Zealand standards for timber construction and steel construction, the following values (from UBC 1982†) may be used:

Seasoned timber (that is, timber having a moisture content of less than 16 percent at the time of installation and used under dry conditions of use as in most covered structures .... $K_{cp} = 0.5$

Unseasoned timber .... $K_{cp} = 1.0$

Steel .... $K_{cp} = 0$

Relatively large lateral deflections may be acceptable in single storey industrial structures. However, such deflections may damage roof cladding, particularly in the vicinity of stiff end walls. Similar damage can occur to wall cladding. In the case of prestressed concrete members, the initial deflection due to prestress (camber) should also be checked to ensure that it is within tolerable limits.

Document ISO 4356† contains detailed recommendations of interest to designers, including the following on resonance:

"Near-coincidence of forcing and natural vibrations may produce resonance of any building element. The degree of resonance may be reduced by appropriate adjustment of either of the two frequencies, or by the provision of vibration insulation or adequate damping. The problem arises mainly where the disturbing force is of large magnitude; that is, with auditoria, dance halls, sports stands, and in buildings having long-span suspended floors with a natural frequency of about 1 to 5 Hz, or containing machines with large unbalanced forces."

the strength or serviceability of the structure or part, or lead to damage of other building components, or be unsightly.

1.4.3 Buildings that may be subject to oscillation due to wind shall be investigated with regard to the possibility of vibrations at critical frequencies.

Table 1

RECOMMENDED MAXIMUM DEFORMATIONS

l = the length (clear span) of a member fixed at each end, or double the length of a fully-fixed cantilever.

h = height of a building or height of a storey.

Item	*Recommended maximum deformation due to:*	
	L_R	$L_R+K_{cp}(D+P)$
Roof member not supporting plaster	0.006 l	–
Roof member supporting plaster	0.004 l	0.004 l
Floor member	0.004 l	0.004 l
Structural roofing and siding made of formed sheet	0.016 l	–

Item	*W*	*S*
Roof member not supporting plaster	0.006 l	0.006 l
Roof member supporting plaster	0.003 l	0.003 l
Wall member not supporting plaster	0.006 l	–
Wall member supporting plaster	0.003 l	–
Building or storey, in each case excluding the storey supporting the roof	0.002 h	–

NOTE – This table relates to clause C1.4.2.

† See list of related documents.

1.5 CONSTRUCTION REQUIREMENTS

1.5.1 Loading notice

1.5.1.1 In all workrooms, workshops, factories, warehouses, stores, and garages the design live load appropriate to each section of the floor shall be displayed on a tablet placed and maintained in a conspicuous position.

1.5.1.2 Such tablet shall be at not less than 900 mm above the floor, 230 mm long and 100 mm wide and boldly lettered as follows, or as approved by the Engineer:

MAXIMUM
DISTRIBUTED LOAD ON
ANY PART OF THIS FLOOR
____
KILOGRAMS PER SQUARE METRE
(COUNCIL) BYLAW

C1.5.1.2 Loadings in this standard are given in kilopascals (kPa), but many building users will wish to think in terms of mass rather than force. It is therefore recommended that the figure in kilograms per square metre given on a loading tablet should be the design live load figure in kilopascals multiplied by 100.

PART 2 DEAD, LIVE, AND SNOW LOADS

2.1. DEAD LOADS

2.1.1 General

2.1.1.1 The mass of a material may be calculated from data given in the appropriate New Zealand Standard.

C2.1.1.1 There is at present no New Zealand standard listing weights of building materials, but such information is given in BS 648† and AS 1170 Part 1†.

2.1.1.2 The dead load in newtons shall be not less than the mass in kilograms multiplied by 9.80665 or as given by NZS 6502†.

C2.1.1.2 The value of 9.80665 m/s^2 (standard gravity) is too precise for the general case of structural design, where mass is unlikely to be known with any great accuracy. For simplicity of calculation, therefore, the dead load in newtons should be taken as the mass in kilograms multiplied by 10.

2.1.2 Partitions

C2.1.2 Clause 2.1.2 avoids unnecessary calculation by treating all partitions as dead load, although strictly speaking only fixed partitions should be regarded as dead load, with movable partitions treated as live load for gravity conditions and as dead load for the computation of the total seismic load.

2.1.2.1 The mass of partitions shall be included in the dead load.

2.1.2.2 Movable partitions and future partitions shall be allowed for by an equivalent uniformly distributed mass per square metre of not less than 33 percent of the mass per metre run of the finished partition.

2.1.3 Earth pressure

2.1.3.1 The loads caused by retained materials and the effects of ground water pressure and uplift shall be calculated according to accepted methods and to the approval of the Engineer.

C2.1.3.1 The load caused by retained material will depend on the mode of yield of the material. Cantilever retaining walls will receive active earth pressures while basement walls will be subjected to higher pressures approximating earth pressure at rest. For the lateral loads on earth retaining structures during earthquakes refer to the paper "Design of earth retaining structures for dynamic loads" by H. Bolton Seed and Robert V. Whitman, ASCE 1970, Speciality Conference at Cornell University, "Lateral stresses in the ground and earth retaining structures".

2.1.3.2 Retaining walls shall be designed to resist overturning or sliding either by the strength method using the ultimate loadings of equations 8 and 9 or by the alternative method using the design loadings of equations 19 and 20.

C2.1.3.2 In designing footings for overturning moments, either by the strength method or the alternative method, the soil stress may be assumed to be uniform throughout a rectangular stress block extending over a portion of the area instead of the commonly assumed triangular stress block. For determining allowable soil stress for the strength method, a factor of safety of 1.8 should be applied to the average measured soil strength; if the alternative method is used then the usual factors of safety apply.

2.1.3.3 Retaining walls and basement walls shall be designed for additional loads due to adjacent buildings or traffic. For public roadways the surcharge load shall be not less than 10 kPa and for public footpaths not less than 5 kPa. Surcharge loadings for private roadways and footpaths shall be assessed on the basis of the anticipated traffic loadings. Consideration shall be given to the effects of exceptional loadings and surcharge effects caused by the method of compaction of the backfill.

† See list of related documents.

2.1.4 Temperature effects

2.1.4.1 Consideration shall be given to the effects of temperature changes. The normal atmospheric temperature range to be considered shall be from 0 °C to 40 °C and consideration shall be given to shading, thermal capacity, contact with the ground, and direct heating by the sun. Expansion joints to minimize temperature loads shall be constructed with due consideration for the temperature at which they are made.

2.1.5 Loads during construction

2.1.5.1 All permanent and temporary structural members of a building shall be protected against loads exceeding the design loads during the construction period except when, as verified by analysis or test, temporary overloading of a structural member would result in no impairment of that member or any other member. In addition, precautions shall be taken during all stages of construction to ensure that the building is not damaged or distorted due to loads applied during construction.

2.2 LIVE LOADS

2.2.1 General

C2.2.1.1 The live loads given in tables 2 and 3 are considered to be adequate to allow for normal impact effects, but they do not (except where specifically noted) allow for possible changes of occupancy nor for high density mobile storage. Where no value is given for L_c it may be assumed that L_u is adequate to cover the likely concentrated load.

The items listed in table 3 are not necessarily related to the occupancy classification used by NZS 1900† Chapter 5.

Roof type 1 of table 3 recognizes that, irrespective of access, people can be expected on occasions to congregate on such places as verandas over streets, verandas giving a view of sports grounds, and the like.

The live loads on roofs given by item 2 of table 3 are required in order to cover construction and maintenance loads only, and therefore need not be applied in equations 2, 4, 13 and 15.

2.2.1.1 Buildings shall be designed for *either*:

(a) The basic minimum uniformly distributed live load L_u as given by clause 2.2.2 reduced in accordance with clause 2.2.4 as appropriate applied over the plan area;

or

(b) The basic minimum concentrated live load L_c as given by clause 2.2.2 distributed uniformly over a square of 0.3 m sides for floors and stairs and of 0.1 m sides for roofs and applied in any possible position; whichever gives the most adverse effect.

2.2.2 Basic minimum live loads

2.2.2.1 *General*

2.2.2.1.1 The basic minimum uniformly distributed live load L_u and the basic minimum concentrated live load L_c shall be as given by table 2 or table 3 as appropriate.

2.2.2.2 Ceiling joists and supporting members

2.2.2.2.1 Ceiling joists and supporting members to ceiling spaces with access for maintenance only shall be designed for a point load of 1 kN in any possible position. This load need not be applied at the same time as the roof live load.

2.2.2.3 *Grandstands and the like*

2.2.2.3.1 In addition to other design requirements of this standard, grandstands, stadiums, assembly platforms, reviewing stands, and the like shall be designed to resist a

† See list of related documents.

horizontal force applied to seats of 350 N per linear metre along the line of the seats and 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 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 loads.

2.2.2.4 *Storage loads*

2.2.2.4.1 Extra heavy loads and goods causing loadings in excess of 10 kPa shall be accurately assessed and the actual masses used as the basis of live load, with an appropriate reduction with wind or earthquake where the full load is not permanently distributed over the area.

2.2.2.5 *Ponding*

2.2.2.5.1 Roofs and verandas shall be designed with sufficient slope or camber to assure adequate drainage when the long-term deflections from dead load have occurred, or shall be designed to support maximum loads including possible ponding of water due to deflection.

2.2.2.6 *Balustrades and parapets*

2.2.2.6.1 Balustrades and parapets shall be designed for the following minimum horizontal live loads:

(a)	Light access stairs, gangways and similar	225 N per metre run
(b)	Stairs, landings, and balconies in private houses and flats	375 N per metre run
(c)	All other stairs, landings, and balconies, and all parapets and handrails to roofs	750 N per metre run

2.2.2.6.2 The loads specified in clause 2.2.2.6.1 and those due to wind and earthquake need not be assumed to act concurrently.

2.2.3 **Moving live loads**

2.2.3.1 *Dynamic effects*

2.2.3.1.1 The impact effect on buildings or parts of buildings from moving live loads such as cranes, lifts, or machinery shall be provided for by an assumed increase in the live load. The minimum increase shall be –

(a)	For supports of lifts	100 percent
(b)	For travelling crane gantry girders and their connections, the increase in maximum static wheel loads shall be –	
	Electric overhead cranes	25 percent
	Hand-operated cranes	10 percent
(c)	For supports of non-reciprocating machinery	20 percent
(d)	For supports of reciprocating machinery	50 percent

Table 2

MINIMUM BASIC LIVE LOADS FOR FLOORS AND STAIRS

Item	L_u *(kPa)*	L_c *(kN)*
1 DOMESTIC (Dwellings, flats etc.)		
1.1 Attics	0.5	1.8
1.2 Balconies	2.0	1.8
1.3 Corridors, hallways, passageways, foyers, lobbies, stairways and landings	as for floor serviced	1.8
1.4 Other floors	1.5	1.8
2 RESIDENTIAL BUILDINGS		
(Boardinghouses, lodging houses, guest houses, hostels, hotels, motels, residential clubs etc.)		
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 Billiard rooms	3.0	2.7
2.5 Boiler rooms and plant rooms: *To be calculated but not less than*	5.0	6.7
2.6 Corridors, hallways, passageways, foyers, lobbies, stairways and landings; *As for floor serviced but need not be greater than*	5.0	4.5
2.7 Dining rooms, cafeterias, restaurants, etc.	3.0	2.7
2.8 Kitchens with storage	4.0	2.7
2.9 Kitchens, domestic	3.0	1.8
2.10 Laundries	3.0	1.8
2.11 Other floors	3.0	1.8
2.12 Stores, strongrooms, etc. *To be calculated but not less than*	5.0	4.5
2.13 Toilet rooms	2.0	1.8
3 EDUCATIONAL BUILDINGS		
(Schools, universities, kindergartens, technical institutes, etc.)		
3.1 Assembly halls	4.0	2.7
3.2 Boiler rooms and plant rooms: *To be calculated but not less than*	5.0	6.7
3.3 Catwalks and service walkways: *To be calculated to provide for men and items of plant likely to be placed on the walkway during repairs and overhaul of plant but not less than 1 kN at 1 m centres*		
3.4 Class and lecture rooms (not for assembly)	3.0	2.7
3.5 Corridors, hallways, passageways, foyers, lobbies, stairways and landings: *As for floor serviced but need not be greater than*	5.0	4.5

Item	L_u *(kPa)*	L_c *(kN)*
3.6 Dining rooms, restaurants etc. (not for assembly)	3.0	2.7
3.7 Dormitories	1.5	1.8
3.8 Gymnasia	4.0	3.6
3.9 Kitchens	4.0	2.7
3.10 Laboratories: *To be calculated but not less than*	3.0	4.5
3.11 Library reading areas	3.0	2.7
3.12 Library stacks		
Not exceeding 1.8 m high	4.0	
For each 0.3 m over 1.8 m high add	0.5	
Concentrated load to be calculated but not less than		4.5
3.13 Lounges, staff rooms, etc.	3.0	2.7
3.14 Offices	2.5	2.7
3.15 Store rooms *To be calculated but not less than*	5.0	4.5
3.16 Toilet and bathrooms	2.0	1.8
3.17 Workshops with light weight plant *Where each plant item weighs not more than 5 kN*	3.0	3.6
all other *To be calculated but not less than*	5.0	4.5
4 INSTITUTIONAL BUILDINGS		
(Hospitals, prisons, etc)		
4.1 Balconies	4.0	1.8
4.2 Bedrooms and wards (allows for change of occupancy)	3.0	1.8
4.3 Boiler rooms and plant rooms *To be calculated but not less than*	5.0	6.7
4.4 Catwalks and service walkways *To be calculated to provide for men and items of plant likely to be placed on to the walkway during repairs and overhaul of plant but not less than 1 kN at 1 m centres*		
4.5 Corridors, hallways, passageways, foyers, lobbies, stairways and landings *As for floor serviced but need not be greater than*	5.0	4.5
4.6 Dining rooms, cafeterias, restaurants, etc.	3.0	2.7
4.7 File rooms *To be calculated but not less than*	5.0	4.5
4.8 Kitchens	4.0	2.7
4.9 Laundries	3.0	2.7
4.10 Libraries	4.0	4.5
4.11 Lounges	3.0	2.7

Table 2 (continued)

MINIMUM BASIC LIVE LOADS FOR FLOORS AND STAIRS

Item	L_u *(kPa)*	L_c *(kN)*
4.12 Offices for general use	2.5	2.7
4.13 Operating theatres		
To be calculated but not less than	3.0	4.5
4.14 Toilet and bath rooms	2.0	1.8
4.15 Utility rooms	3.0	2.7
4.16 X-ray rooms		
To be calculated but not less than	3.0	4.5
5 PUBLIC ASSEMBLY		
(Public halls, theatres, courts of law, drill halls, grandstands, places of worship, libraries, cinemas, planetaria, concert halls, meeting houses, etc.)		
5.1 Assembly areas, fixed seating (except grandstands)	3.0	2.7
5.2 Assembly areas, moveable seating (except grandstands)	4.0	3.6
5.3 Boiler rooms and plant rooms		
To be calculated but not less than	5.0	6.7
5.4 Catwalks and service walkways		
To be calculated to provide for men and items of plant likely to be placed on to the walkway during repairs and overhaul of plant but not less than 1 kN at 1 m centres		
5.5 Corridors, hallways, passageways, foyers, lobbies, stairways and landings		
As for floor serviced but need not be greater than	5.0	4.5
5.6 Dining rooms, cafeterias, restaurants etc.	3.0	2.7
5.7 Dressing rooms	2.0	1.8
5.8 File rooms		
To be calculated but not less than	5.0	4.5
5.9 Fly galleries	4.5 kN per running metre	
5.10 Grids	2.5	
5.11 Gymnasia	4.0	3.6
5.12 Grandstands, fixed seating (see also clause 2.2.2.3)	4.0	3.6
5.13 Grandstands, moveable seating and cantilevers (see also clause 2.2.2.3)	5.0	4.5
5.14 Kitchens	4.0	2.7
5.15 Law Courts	3.0	3.6
5.16 Libraries, reading rooms	3.0	2.7
5.17 Library stacks		
Not exceeding 1.8 m high	4.0	
For each 0.3 m over 1.8 m high add	0.5	
Concentrated load to be calculated but not less than		4.5
5.18 Lounges and bars	3.0	2.7
5.19 Offices	2.5	2.7
5.20 Projection rooms	5.0	–
5.21 Store rooms		
To be calculated but not less than	5.0	4.5
5.22 Stages	5.0	3.6
5.23 Toilet and locker rooms	2.0	1.8
6 OFFICES (Offices, banks etc.)		
6.1 Banking chambers and public areas	4.0	4.5
6.2 Boiler rooms and plant rooms		
To be calculated but not less than	5.0	6.7
6.3 Cafeterias, dining rooms, etc.	3.0	2.7
6.4 Corridors, hallways, passageways, foyers, lobbies, stairs and landings		
As for floor serviced but need not be greater than	5.0	4.5
6.5 File and store rooms		
To be calculated but not less than	5.0	4.5
6.6 Kitchens (with storage)	4.0	2.7
6.7 Kitchens	3.0	2.7
6.8 Libraries	3.0	2.7
6.9 Machine and mobile storage rooms		
To be calculated but not less than	5.0	4.5
6.10 Offices: public areas and ground floors	4.0	4.5
6.11 Offices for general use	2.5	2.7
6.12 Offices with computing data processing and similar equipment	3.5	
Concentrated load to be calculated		
6.13 Toilet and locker rooms	2.0	1.8
6.14 Vaults and strongrooms		
To be calculated but not less than	5.0	4.5
7 RETAIL OCCUPANCY		
(Shops, department stores, supermarkets etc.)		
7.1 Boiler rooms and plant rooms		
To be calculated but not less than	5.0	6.7
7.2 Corridors, passageways, foyers, lobbies, stairs and landings	5.0	4.5
7.3 Dining rooms, restaurants, cafeterias etc.	3.0	2.7
7.4 Kitchens	4.0	
7.5 Machine rooms, file rooms and mobile storage units		
To be calculated but not less than	5.0	4.5
7.6 Offices for general use	2.5	2.7
7.7 Offices with computing data processing and similar equipment	3.5	
Concentrated load to be calculated		

Item	L_u (kPa)	L_c (kN)
7.8 Public areas, shop floors	4.0	3.6
7.9 Storage areas		
To be calculated, not less than 2.4 kPa for each metre of storage height with a minimum total of	4.0	
Concentrated load to be calculated		
7.10 Toilet rooms	2.0	
7.11 Vaults and strong rooms		
To be calculated but not less than	5.0	4.5
8 STORAGE		
8.1 Cold storage		
To be calculated at not less than 5.0 kPa for each metre of storage height with a minimum of	15.0	
8.2 Corridors, passageways: for pedestrian traffic stairs and landings – *As for floor serviced but need not be greater than*	5.0	4.5
8.3 Corridors, passageways: vehicle accessway		
To be calculated but not less than	5.0	9.0
8.4 Offices for general use	2.5	2.7
8.5 Plant rooms		
To be calculated but not less than	5.0	6.7
8.6 Storage areas		
To be calculated at not less than 2.4 kPa for each metre of storage height with a minimum total of	7.5	
Concentrated load to be calculated		
8.7 Toilet and locker rooms	2.0	
8.8 Vaults and strong rooms		
To be calculated but not less than	5.0	4.5
9 INDUSTRIAL (Workshops, factories etc.)		
9.1 Boiler rooms and plant rooms		
To be calculated but not less than	5.0	6.7
9.2 Cafeterias, dining rooms etc.	3.0	
9.3 Catwalks and service walkways		
To be calculated to provide for men and items of plant likely to be placed on to the walkway during repairs and overhaul of plant but not less than 1 kN at 1 m centres		
9.4 Corridors, passageways: for pedestrian traffic stairs and landings *As for floor serviced but need not be greater than*	5.0	4.5
9.5 Corridors, passageways: vehicle accessways		
To be calculated but not less than	5.0	9.0

Item	L_u (kPa)	L_c (kN)
9.6 Kitchens	4.0	
9.7 Working areas and storage		
Broadcasting studios	4.0	3.6
Workrooms without plant	2.5	2.7
Workrooms and workshops with lightweight plant		
Where each plant item weighs not more than 5 kN	3.0	3.6
Workrooms and workshops all others		
To be calculated but not less than	5.0	4.5
Printing plants, storage		
To be calculated but not less than 4.0 for each metre of clear height with a minimum total of	12.5	
Printing plants, other		
To be calculated but not less than	12.5	
Concentrated load to be calculated		
Television studios	4.0	3.6
9.8 Offices for general use	2.5	2.7
9.9 Toilet and locker rooms	2.0	
9.10 Vaults and strongrooms		
To be calculated but not less than	5.0	
10 CAR PARKING AND GARAGES		
(Carparks, garages, vehicle repair workshops etc.)		
10.1 Car parking only including driveways and ramps	2.5	9.0
10.2 Catwalks and service walkways		
To be calculated to provide for men and items of plant likely to be placed on to the walkway during repairs and overhaul of plant but not less than 1 kN at 1 m centres		
10.3 Offices	2.5	
10.4 Parking for vehicles exceeding 2500 kg, including repair workshops		
To be calculated but not less than	5.0	9.0
10.5 Plant rooms		
To be calculated but not less than	5.0	6.7
10.6 Public waiting areas	4.0	3.6
10.7 Toilet and locker rooms	2.0	1.8
10.8 Stairways and landings		
As for floor serviced but need not be greater than	5.0	4.5

Table 3 **MINIMUM BASIC LIVE LOADS FOR ROOFS**

Type of roof		L_u *(kPa)*	L_c *(kN)*
1	Roofs and verandas with access for fire escape, roof garden, light storage, and general pedestrian traffic, and also roofs and verandas where people can be expected to congregate on occasions irrespective of access, where no load will exceed 2.0 kPa.	2.0	1.0
2	Other roofs and verandas	0.25	1.0

2.2.3.2 *Horizontal forces*

2.2.3.2.1 The horizontal force acting transverse to the 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 percent

(b) For hand-operated cranes . . . . 5 percent

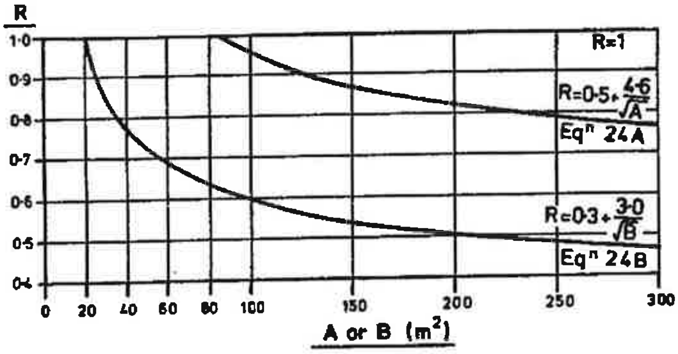
2.2.3.2.2 Horizontal forces acting along the rails shall be taken as a percentage of the static wheel loads which can occur on the rails as follows:

For overhead cranes, either
electric or hand-operated . . . . 5 percent

2.2.3.2.3 The forces specified in either clause 2.2.3.2.1 or clause 2.2.3.2.2 shall be considered as acting at the rail level, and being appropriately transmitted to the supporting systems.

2.2.3.2.4 Gantry girders and their vertical supports shall be designed on the assumption that either of the horizontal forces specified in clause 2.2.3.2.1 or clause 2.2.3.2.2 may act at the same time as the vertical load.

C2.2.4 The effect of clause 2.2.4.1 (a) and (b) is to increase the reduction with increasing tributary area of the structural member. The reduction is less marked for storage and commercial uses where the probability of fully loaded floor areas is greater than for office and residential uses. The probability that floor areas will on occasion be fully loaded in assembly uses is recognized by clause 2.2.4.2.



(See clause 2.2.4) LIVE LOAD REDUCTION FACTOR R

Fig. C2

2.2.4 Reduction of live loads

2.2.4.1 Where a structural member supports a tributary area of floor or of roof with access (item 1 of table 3) or of a combination of these then subject to clause 2.2.2.2 the member may be designed for a reduced live load L_R where

$$L_R = RL_u \quad \text{(23)}$$

and R is a factor related to the use of the floor or roof and is determined according to the tributary area of the structural member, as follows:

(a) For storage, retail stores, and garages where the tributary area A exceeds 85 m²:

$$R = 0.5 + \frac{4.6}{\sqrt{A}} \quad \text{(24A)}$$

where A is in square metres.

(b) For all other occupancies and roofs with access, where the tributary area B exceeds 20 m²:

$$R = 0.3 + \frac{3.0}{\sqrt{B}} \quad \text{(24B)}$$

where B is in square metres.

(c) For all cases not included in (a) or (b):

$R = 1.0$

2.2.4.2 No reduction shall be permitted (that is, R shall be taken as 1.0) for fixed and portable grandstands, stadiums, and the like nor for any other assembly use.

C2.3.1 The live loads for certain roofs given in this standard are significantly lower than those used in the past. The provision for minimum snow loads on flat and low-pitched roofs is intended to maintain a margin to cover uncertainties in the currently available statistical information on actual snow loads.

To simplify administration, in fig. 1 county boundaries have been taken as snow zone boundaries in the South Island.

2.3 SNOW LOADS

2.3.1 General

2.3.1.1 In snow zone 1 above an elevation of 400 m and throughout zones 2 to 5 inclusive as shown in fig. 1 buildings shall be designed for the snow loads given by this section.

2.3.1.2 The snow load shall be applied to any one portion of the roof area with no snow load on the remainder of the area if this produces a more unfavourable effect than the snow load applied over the entire area.

2.3.1.3 Projections such as gutters, vents, chimneys and the like shall be designed to resist the loads resulting from sliding snow.

C2.3.2 A value of C_R = 0.75 should be used only if:

(a) The roof is not shielded from the wind on any side and is not likely to be shielded by obstructions higher than the roof within 10 h *of the building, where* h *is the height of the obstruction above roof level;* and

(b) The roof does not have any projections, such as parapets, that would prevent the snow from being blown off.

A value of C_R = 2.0 should be used where snow loads may accumulate because of the topography, or where the building is within the "shade area" of an adjacent building, shelter-belt, windbreak, or other obstruction. The "shade area" may be taken as that distance within 2 h *of the obstruction, where* h *is the height of the obstruction above roof level.*

Local observation and experience should be used in determining the value of C_R.

The coefficient C_R relates to the wind effects of obstructions. The sliding of snow from one building to another is dealt with by clause 2.3.3.

2.3.2 Minimum snow loads

2.3.2.1 Subject to clause 2.3.3, the snow load shall be taken as:

$$S = C_R f \qquad (25)$$

where

C_R is the exposure coefficient as given by clause 2.3.2.2.

f is the basic snow load as given by fig. 2 in accordance with the snow zone and the slope of the roof.

2.3.2.2 The exposure coefficient C_R shall be:

0.75 for areas exposed to the wind;

2.0 for areas where obstructions or shielding are likely to cause snow drifts;

an appropriate value between 0.75 and 2.0 for other areas.

2.3.3 Snow drifting on to roof valleys, multi-level roofs, balconies and the like

2.3.3.1 The snow load on roof valleys, multi-level roofs, balconies and the like shall be as given by clauses 2.3.3.2, and 2.3.3.3 as appropriate provided that in special cases the effects of snow drifts may be determined in accordance with NBC 80 The Supplement, Chapter 4.†

2.3.3.2 For roof valleys:

(a) The basic snow load to be used for all roof slopes shall be that defined by clause 2.3.2 and fig. 2 using the basic snow load appropriate for 0° to 30° roof slopes, and C_R = 1.0.

(b) Roof valleys where the average roof slope is less than 10°: no separate load case other than clause 2.3.2 need be considered.

(c) Roof valleys where the average roof slope is 10° or greater: the load on each element of the valley shall be determined from the worst effects of *either*:

(1) A uniformly distributed snow load as specified in clause 2.3.3.2 (a) *or*

(2) A load varying from 0.5 of the load specified in clause 2.3.3.2 (a) at the ridges to 1.5 times the load specified in clause 2.3.3.2 (a) at the valley.

2.3.3.3 For multi-level roofs, balconies, parapets etc.: In addition to the snow load specified in clause 2.3.2, the effect of snow drifting on to the lower roofs, balconies and parapets and the like, where the upper roof is part of the

†See list of related documents.

same building or an adjacent building not more than 3 m away, shall be determined by assuming an additional depth of snow adjacent to the wall between the roofs equal to *either*:

(a) The height of the wall and taking the density of snow equal to 2 kPa per metre depth of snow: *or*

(b) Three times the load specified in clause 2.3.2

whichever is the lesser, and in either case this snow shall be assumed to be battered away from the wall so that the additional load on the roof is reduced to zero at a distance from the wall equal to twice the depth of the snow adjacent to the wall.

2.3.4 Sliding, creeping, and melting snow

2.3.4.1 For multipitch roofs where the higher roof is steeper than the lower roof an additional load case shall be considered by assuming 50 percent of the snow from the upper roof to have slid on to the lower roof. After sliding the snow shall be assumed to be battered at 2:1 on the roof sloping away from the higher roof and an equivalent height of snow at the roof junction shall be accordingly computed. Where such a distribution is longer than the lower roof, the sliding snow shall be distributed over the full length available as above, but only that snow on the roof shall contribute to the loading. This load shall be in addition to that specified in clause 2.3.2 or 2.3.3.

2.3.4.2 Lower roofs with pitches greater than 20° need have no additional load due to sliding.

2.4 TOTAL REDUCED GRAVITY LOAD

C2.4.1 For ordinary buildings with normal weight distribution and with normal-weight concrete floors (D + L/3) may be taken as 1.1 D to simplify calculations.

2.4.1 For the purpose of calculating horizontal seismic force in accordance with clause 3.3 the total reduced gravity load W_t shall be computed as follows:

(a) When $0 < L \leqslant 1.5$ kPa

$$W_t = D. \quad \text{(26A)}$$

(b) When $1.5 \text{ kPa} < L \leqslant 5$ kPa

$$W_t = D + L/3 \quad \text{(26B)}$$

(c) When $L > 5$ kPa

$$W_t = D + 2L/3 \quad \text{(26C)}$$

provided that for library stack areas W_t shall in all cases be computed in accordance with equation 26C.

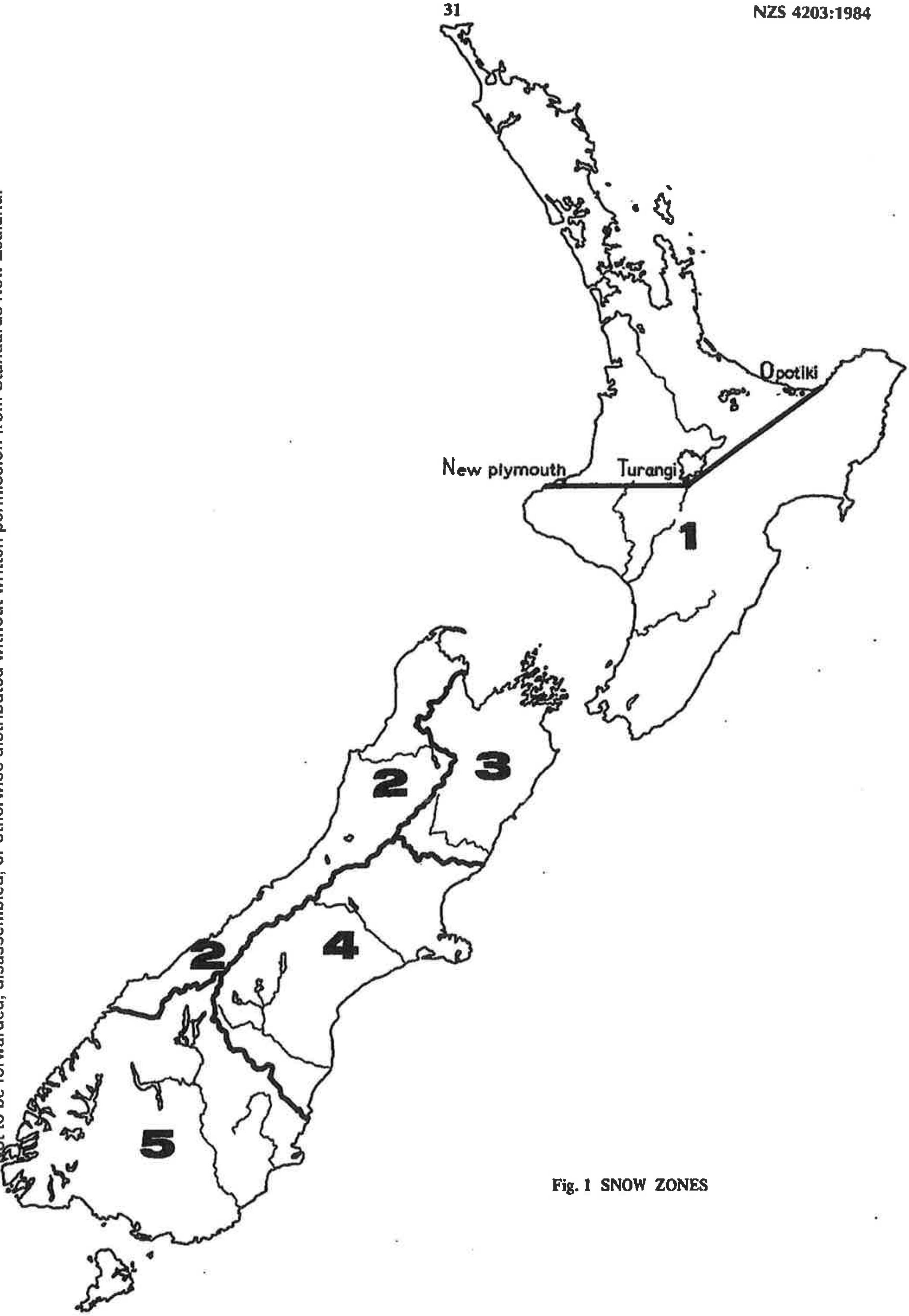


Fig. 1 SNOW ZONES

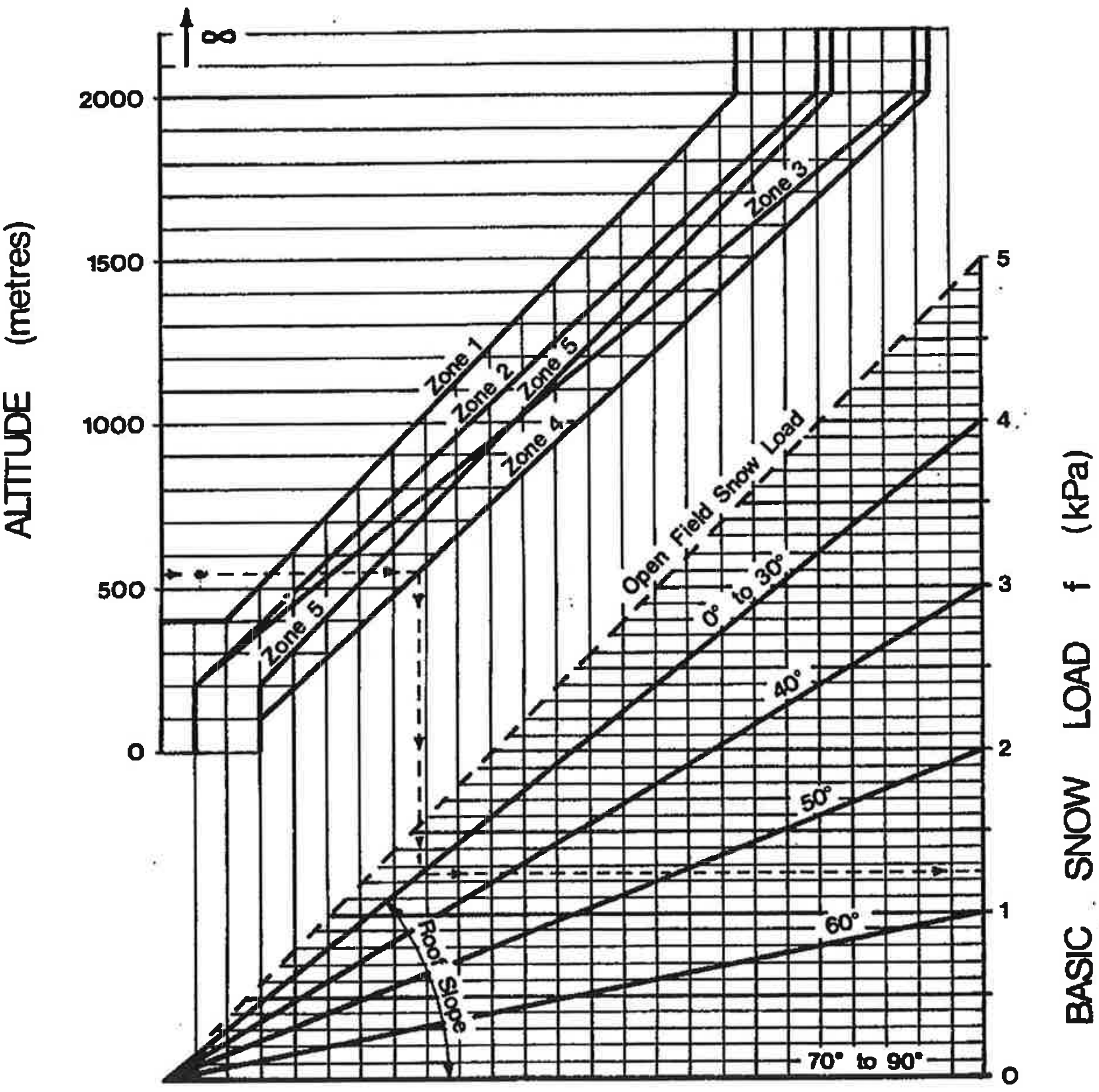


Fig. 2 BASIC SNOW LOAD f

PART 3 EARTHQUAKE PROVISIONS

3.1 SYMMETRY

3.1.1 The main elements of a building that resist seismic forces shall, as nearly as is practicable, be located symmetrically about the centre of mass of the building.

C3.1.1 It is recognized that the aim to achieve structural symmetry is frequently in conflict with the purpose and architectural design of a building. For high buildings, symmetry is one of the most basic requirements in achieving a structure of predictable performance. Simple geometry is essential for obtaining symmetry in practice. Notwithstanding the availability of modern computers, considerable uncertainty exists in selecting a mathematical model representing the true behaviour of complex arrangements such as combinations of geometrically dissimilar shear walls and unsymmetrical combinations of shear walls and frames. Geometrically dissimilar resisting elements are unlikely to develop their plastic hinges simultaneously, and ductility demand may also be increased by torsional effects.

3.2 DUCTILITY

3.2.1 The building as a whole, and all of its elements that resist seismic forces or movements, or that in case of failure are a risk to life, shall be designed to possess ductility; provided that this shall not apply to small buildings complying with clause 3.4.8.1 designed in accordance with clause 3.4.8.2 nor to tied veneers (item 3 (b) of table 8) and unreinforced or partially reinforced walls and partitions (item 4 of table 8) designed in accordance with clause 3.4.9.

3.2.2 Structural systems intended to dissipate seismic energy by ductile flexural yielding shall have "adequate ductility".

3.2.3 "Adequate ductility" in terms of clause 3.2.2 shall be considered to have been provided if all primary elements resisting seismic forces are detailed in accordance with special requirements for ductile detailing in the appropriate material code.

C3.2 Recent earthquake damage at Tokachi-oki (1968), San Fernando (1971), and elsewhere confirms that all seismic resisting systems, regardless of building height, designed to the seismic loadings of this standard must have ductility with the minor exceptions indicated in clause 3.2.1.

The general requirement for ductility must at present be qualitative rather than quantitative except for buildings designed to dissipate seismic energy by ductile flexural yielding. See also clause 3.2.3.

The requirement of clause 3.2.2 is in effect a practical approximation for the assessment of section curvature ductility demand. A more rigorous analytical approach, which is applicable only to reasonably regular symmetrical frames without sudden changes in storey stiffness, is a method using the following approximate criterion: the building as a whole should be capable of deflecting laterally through at least eight load reversals so that the total horizontal deflection at the top of the main portion of the building under the loadings of equations 4 and 5 and calculated on the assumption of appropriate plastic hinges, is at least four times that at first yield, without the horizontal load carrying capacity of the building being reduced by more than 20 percent. The horizontal deflection at the top of the building at first yield should be taken as that when yield first occurs in any main structural element or that at the earthquake load E *calculated on the assumption of elastic behaviour, whichever is the lesser.*

For buildings other than reasonably regular symmetrical frames without sudden changes in storey stiffness as provided in clause 3.4.7, maximum section curvature ductility demand should preferably be determined by a non-linear analysis using earthquake inputs appropriate to the site.

Primary members of the seismic resisting system subjected to the displacement or curvature ductility demand determined by the above procedure, are acceptable if they lose no more than 30 percent of their strength and provided the overall building ductility requirements are met.

Such members could be required to possess a ductility factor well in excess of 4 to satisfy the overall ductility demand during eight reversals.

A structure having strength greater than that required to meet the design earthquake load E *may have less ductile members, but in eight load reversals should be capable of reaching a total lateral deflection of at least four times that when the design earthquake load is applied.*

It will be necessary to check that members not required by design to be part of the horizontal force resisting system have adequate vertical load carrying capacity when the seismic resisting system deforms to resist the earthquake. Such checks should include a check on column and beam adequacy at four or more times the distortion from the specified loading.

It should be noted that the deflected shape of the building under earthquake loads can differ significantly from the shape under the specified static loads.

Unless its seismic resisting system is capable of dissipating large amounts of energy by hysteretic damping, a building designed to the earthquake loadings of this standard can be expected to be subject to excessively large section curvature ductility or deflection demands in earthquakes that:

Have energy inputs significantly in excess of 1940 El Centro N–S in zone A (or 85 percent of that in zone B, 67 percent in zone C); or

Are of different power spectral density from 1940 El Centro N–S, including those with a very pronounced peak at one or more periods.

Clause 3.2.3 should be read in conjunction with clauses 3.3.1 and C3.3.

C3.3 The earthquake provisions of this standard are based on the assumption that during its lifetime a building in zone A will probably experience (a) one or more earthquakes of high intensity and long duration, and (b) several earthquakes of moderate intensity and duration. It is further assumed that probability of such occurrences is less in zone B than zone A, and less in zone C than zone B.

There will be particular earthquake motions that are most likely to damage a particular building. However, simple descriptions of these are not yet available and so a variety of earthquake motions has been considered. For class III buildings on rigid or intermediate soils in seismic zone A the earthquake motions considered included those of the 1940 El Centro N–S type, shorter motions of higher accelerations and motions of longer duration; however, motions of somewhat differing spectral frequency content were not excluded.

Some New Zealand design standards for particular structural materials do not at present (1975) contain detailed requirements based on the design principles of clause 3.3, and therefore do not contain the special requirements for

3.3 DESIGN PRINCIPLES

3.3.1 General

3.3.1.1 Design shall be in accordance with the appropriate material code subject to the general principles of design set out below:

ductile detailing that are referred to in clause 3.2.3. It is intended that such requirements will be incorporated in revisions of the New Zealand standards concerned. Until these revisions are completed designers will have to apply accepted engineering principles to determine the additional precautions that are necessary, particularly in structural detailing, in the light of section 3.3.

C3.3.2.1 Elastic damped response is inadequate for resisting earthquakes of moderate or greater intensities. For example, a building responding elastically with 2 percent damping may amplify El Centro N–S 1940 type motion by a factor of 3 or more; if the building had a yield capacity of 0.15 g therefore, it would need an inelastic energy dissipating reserve capacity in order to withstand an earthquake having only one-sixth (approximately) of the intensity of El Centro N–S 1940 type ground motion; earthquakes of such intensity are likely to occur relatively frequently in many parts of New Zealand. The ductility requirements of section 3.2 are intended to meet the design principles of clause 3.3.2.1.

C3.3.2.3 It is usually adequate to design the beams of two-way framing or wall-frame systems for seismic loadings considered separately along the two principal axes of the structure. However, because structures deform significantly in other directions and for a significant fraction of the time of severe attack, simultaneous hinging of all beams or yielding of all diagonal braces framing into a column or wall will probably occur. This is the concurrency effect to be considered in the flexural, axial load, and shear design of columns and walls, in the design of joints, and in the design of foundations. Secondary elements need be designed for induced forces only.

C3.3.2.4 As an example of the operation of clause 3.3.2.4 consider the design of the ductile moment-resisting frame with beam hinges: The flexural beam reinforcement would be determined by the strength method using the design load U given by clause 1.3.2.3. For shear effects due to capacity design procedures, clause 3.3.2.4 permits the contribution of gravity load to be derived from the combination ($1.0\,D + 1.0\,L_R$) and either those shear effects, or the shear effects determined by the strength method, whichever was the greater would govern.

C3.3.3.1 The relative stiffness and strength of members should be such that column hinge mechanisms are avoided (see clause 3.3.3.4.1) and that member displacement ductility demand does not greatly exceed overall building ductility demand.

C3.3.3.2 The contribution to the flexural capacity of beams made by the tensile strength of the floor slabs in either cracked or uncracked state is subject to further investigation. At present, the contribution of the tensile strength of the concrete is generally neglected, but in special cases the effect might need to be considered.

3.3.2 Energy dissipation, capacity design and concurrency effect

3.3.2.1 Buildings shall be designed to dissipate significant amounts of energy inelastically under earthquake attack.

3.3.2.2 Buildings designed for flexural ductile yielding, or for yielding in diagonal braces, shall be the subject of capacity design. In the capacity design of earthquake-resistant structures, energy-dissipating elements or mechanisms are chosen and suitably designed and detailed, and all other structural elements are then provided with sufficient reserve strength capacity to ensure that the chosen energy-dissipating mechanisms are maintained throughout the deformations that may occur.

3.3.2.3 Columns or walls, including their joints and foundations, which are part of a two way horizontal force resisting system shall be designed for the concurrent effects resulting from the simultaneous yielding of all beams or diagonal braces framing into the column or wall from all directions at the level under consideration and as appropriate at other levels.

3.3.2.4 When capacity design procedures are applied, the gravity load acting on the structure may be taken as ($1.0\,D + 1.0\,L_R$).

3.3.3 Ductile frames

3.3.3.1 *General*

3.3.3.1.1 Ductile frames (items 1 and 2 of table 5) shall be capable of dissipating seismic energy in a flexural mode at a significant number of beam hinges, except that dissipation of seismic energy in column hinges is permitted for buildings which comply with clause 3.3.3.5.

3.3.3.2 *Beams*

3.3.3.2.1 Non-ductile failure in beams shall be avoided. The flexural overstrength of beams shall be assessed for the actual material quantities in the beams and any adjacent portions of slabs that are likely to be strained simultaneously.

C3.3.3.3 Premature failure or excessive deformation of beam-column zones can occur because of shear, buckling, bond effects, extensive yielding of reinforcement, or weld failure. It is important that the joint shears due to all beams framing into a junction should be considered.

C3.3.3.4 Except in the case of very low structures, sidesway mechanisms with plastic hinges at the top and bottom of the columns of a single storey make extremely high demands on column ductility.

(a) Recent studies show that when plastic hinges have formed in many beams additional sidesway may take place. As a result, column moments could rise above those determined from elastic analysis and lead to the formation of column hinges. Even for buildings with conservatively designed columns of normal proportions, current methods of analysis and design cannot exclude the possibility that hinges might form at the top and bottom of a column within a storey. Pending further studies, therefore, the possibility of this condition should be considered when column shear demands are assessed.

(b) Present indications are that in space frames hinges might form simultaneously in all directions framing into a column for at least six adjacent storeys (the beams in any remaining storeys may be at high levels of stress).

C3.3.3.5 Hinges for columns forming column hinge mechanisms must be designed and detailed to be ductile.

C3.3.3.5.1 Clause 3.3.3.5.1 is intended to permit the formation of some ductile column hinges during an earthquake.

Allowances shall be made for probable sources of overstrength including overstrength of materials (as compared with specified strength) and strain hardening of steel; this allowance shall depend on the probable extent of the yield excursion and the stress–strain characteristics for the steel concerned under cyclic loading (Bauschinger effect).

3.3.3.3 *Beam–column junctions*

3.3.3.3.1 Failure of beam–column junction zones and the formation of plastic hinges in beam–column junction zones shall be avoided.

3.3.3.4 *Columns*

3.3.3.4.1 Non-ductile failure of columns shall be avoided. Columns shall be designed to have adequate overstrength to avoid the formation of hinges and column hinge mechanisms, except as permitted by clause 3.3.3.5. Column overstrengths shall be sufficient to allow for the following:

(a) Inelastic effects leading to a distribution of beam flexural overstrength into columns different from the distribution derived from elastic analysis; and

(b) Column axial loads appropriate to the simultaneous formation of beam hinges in several storeys.

3.3.3.5 *Column hinging and column hinge mechanisms*

3.3.3.5.1 In "adequately redundant" structures the formation of a column hinge because of bending moment and axial tension, or bending moment and low axial compression, is permissible provided the shear capacity of the column is maintained. For the purposes of this clause a structure may be considered "adequately redundant" if for every column with a potential plastic hinge at least three other columns of the main horizontal load resisting system, interconnected by a rigid diaphragm and within the same storey height, can be shown to remain elastic under the capacity design horizontal load.

3.3.3.5.2 In single or two-storey buildings and in three storey buildings where the mass of the roof and walls of the top storey does not exceed 150 kg/m^2 of the floor area, and in the top storey of a multi-storey building, column hinge mechanisms are permitted.

3.3.4 Shear walls

3.3.4.1 *Ductile coupled shear walls*

C3.3.4.1 Well-proportioned ductile coupled cantilever shear walls could well be the best earthquake resisting structural systems available in reinforced concrete. The overall behaviour is similar to that of a moment resisting frame but with the advantages that, because of its stiffness, the system affords a high degree of protection against non-structural damage, even after considerable yielding in the coupling beams. In addition the coupling beams usually carry only small gravity loads and are reparable.

The major difference between the simple cantilever shear wall designed for ductile flexural yielding and the ductile coupled shear wall is that in the latter the coupling system can be made the major energy-dissipating device.

Permanent damage, such as mis-alignment of the building, is thus delayed, and disaster due to instability is unlikely even after all the overall ductility has been utilized.

3.3.4.1.1 Ductile coupled shear walls (item 3 of table 5) shall consist of two or more ductile cantilever shear walls (see clauses 3.3.4.1.2 and 3.3.4.2) interconnected by beams of substantial capacity to transfer shear forces from one wall to the other(s).

3.3.4.2. *Ductile cantilever shear walls*

C3.3.4.2.1 The values of S given for items 2 and 3 in table 5 apply only to walls complying with clause 3.3.4.2.1. The risk of non-ductile forms of energy dissipation rises with increasing axial and shear stresses, and in reinforced concrete and masonry walls with high percentages or unsymmetrical arrangements of main reinforcement.

Seismic shear predicted by capacity design should be increased in order to allow for the real dynamic situation where various modes superimpose in such a way as to produce hinging with the centre of inertial load considerably lower than that corresponding to the code distribution of loads. Guidance is given in materials codes for the appropriate level of dynamic shear magnification.

3.3.4.2.1 Ductile cantilever shear walls (items 2 and 3 of table 5) shall be suitably designed and detailed to ensure that energy dissipation will be by ductile flexural yielding and that the wall will not fail prematurely in a non-ductile manner.

3.3.5 Buildings with diagonal bracing capable of plastic deformation in tension only

3.3.5.1 Buildings with diagonal bracing capable of plastic deformation in tension only (item 7 of table 5) shall be designed with particular attention to clause 3.3.2.2 so that all structural elements in the building have capacities exceeding the yield capacity of the braces allowing for all sources of overstrength.

3.3.6 Elastically responding structures and structures of limited ductility

3.3.6.1 General. Within the limits of table 5, buildings may be designed in accordance with either the elastic response design procedure of clause 3.3.6.2, or the limited ductility design procedure of clause 3.3.6.3, as an alternative to the ductile design procedure.

3.3.6.2 Elastic response design procedure. Elastically responding structures shall be designed and detailed in accordance with the appropriate materials code.

3.3.6.3 Limited ductility design procedure. Structures of limited ductility, not specifically designed to ensure ductile flexural yielding through the application of the principles of capacity design shall be suitably designed and detailed in accordance with the appropriate materials code.

3.3.7 Foundations

C3.3.7 Foundation design requirements are separated into those for factored load design and those for capacity design, the latter being necessary for structures required to yield in order to dissipate adequate amounts of seismic energy.

3.3.7.1 *Design for factored loads*

C3.3.7.1 The factored loads given in clause 1.3.2 are for use in conjunction with strength design methods. Pending revision of NZS 4205P, it is recommended that a factor safety of 1.8 should be applied to the measured soil strength for determining permissible soil pressures under the given factored load combinations. (In designing footings for overturning moments, a rectangular stress block with uniform contact stress extending over a portion of the area is used in the strength method.)*

If the alternative method (see clause 1.3.3) is used, the usual factors of safety apply.

3.3.7.1.1 For buildings in the following categories:

Items 4 and 6 of table 5 and single storey ductile columns

Items A2, A3, and A4 of table 5A

foundation systems shall be designed to resist the forces and moments resulting from the factored load combinations of clause 1.3.2 or clause 1.3.3 as appropriate.

C3.3.7.1.2 If it is found that the resultant at the base of the wall falls outside the end of the wall, as will often be the case with small gravity loads, then it is considered reasonable to reduce the eccentricity of the resultant by arbitrarily increasing the vertical load. The additional vertical load could well be mobilized by factors not included in the design but, even if such additional vertical load does not act, the reduction in resistance to overturning is considered permissible.

3.3.7.1.2 For cantilever walls in item 4 of table 5 where, under the application of the lateral loads corresponding to $S = 2.0$ and the application of the minimum design gravity load, the resultant falls beyond the end of the wall at its base, the design gravity load may be increased by not more than 20 percent for design of the foundation system only.

3.3.7.2 *Capacity design*

C3.3.7.2 Where design loadings on the foundation system are determined by the overstrength of other parts of the structure, a factor of safety of 1.1 for soil pressures is suggested, pending revision of the foundation code, because at this extreme condition partial yielding of the subsoil might not be significantly damaging and it provides an additional energy dissipating mechanism.

*See list of related documents.

3.3.7.2.1 For buildings in the following categories:

Items 1, 2, 3, and 5 (i) of table 5.

foundation systems shall be designed to preclude foundation failure, or uplift of an entire foundation element, at loadings corresponding to yielding of the earthquake energy dissipating elements, taking concurrency effects into account where applicable.

C3.3.7.2.2 Under yielding conditions, analysis could show that some column footings will lift. This will reduce the number of energy-dissipating hinges. The clause is intended to preclude an excessive reduction in the number of hinges.

3.3.7.2.2 Notwithstanding the provisions of clause 3.3.7.2.1, for reasonably regular frame buildings in items 1 and 2 of table 5, uplift of the entire foundation element may be permitted for not more than one quarter of the total number of such elements for building deformations in any direction.

C3.3.7.3 It is believed that in some instances, particularly where the yield capacity of shear walls exceeds by a significant amount the design level required by this standard, some yielding of wall foundations is acceptable. This will increase the fundamental period of the structure and hence, in general, reduce response. The resulting increased shears in the foundation structure may need to be considered.

3.3.7.3 *Load limitation for foundation design*

3.3.7.3.1 Notwithstanding the provisions of clauses 3.3.7.1 and 3.3.7.2 no foundation system need be designed to resist forces and moments greater than those resulting from a horizontal force corresponding to $SM = 2$.

C3.3.8 Examples of structural systems not covered under clauses 3.3.3 to 3.3.7 inclusive may include long span structures where it is impractical to make the columns stronger than the beams, framing systems with shear walls above, and systems which combine cross-bracing and moment resisting framing. Special studies should be made for such structures and appropriate S values assessed accordingly.

3.3.8 Other structural systems

3.3.8.1 Notwithstanding clauses 3.3.3 to 3.3.7 inclusive structural systems which are ductile or which have limited ductility but are not covered elsewhere by this standard may be designed subject to a special study.

3.3.9 Methods of analysis

3.3.9.1 Buildings shall be analysed by the equivalent static force method specified in section 3.4. In addition a spectral modal analysis as specified in clause 3.5.2 shall be approved for any building and may be required by the Engineer for any building where, in his opinion, special circumstances exist, for example where a building is of particular importance to the community, or where a special study is required by this standard and spectral modal analysis is an appropriate ingredient of the special study for that building.

3.4 EQUIVALENT STATIC FORCE ANALYSIS

3.4.1 General

3.4.1.1 The horizontal seismic forces specified in this section shall be applied simultaneously at each floor and roof level.

C3.4.1.2 For symmetrical buildings and for buildings whose seismic resisting elements are oriented in two perpendicular directions, application of the specified forces in a direction other than a principal direction will produce the most adverse effects in some cases. In general, consideration of such other directions should not seriously affect the structural performance and is not required.

The direction of application of horizontal force should not be confused with the concurrency requirements of clause 3.3.2.3, which is concerned with capacity design.

3.4.1.2 For buildings symmetrical about at least one axis and for buildings with seismic resisting elements located along two perpendicular directions, the specified force 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.

Table 4

RISK FACTOR *R*

Category	*Description*	*R*
1	Structures containing highly hazardous contents.	2.0
2a	Buildings which are intended to remain functional in the Emergency Period for major earthquakes.	1.6
2b	Buildings whose failure could cause high loss of life in the surrounding area.	1.6
3a	Buildings which should be functioning in the Restoration Period for major earthquakes.	1.3
3b	Buildings whose contents have a high value to the community.	1.3
4	Buildings with normal occupancy or usage.	1.0

C3.4.2 To avoid excessively high values resulting from the combination of high R and S values, an upper limit for C_d has been introduced. Structures designed to the values of C_d corresponding to the upper limit should respond elastically to motions of approximately half the design earthquake if they possess 3 percent damping.

The ductility demands on buildings with fundamental periods of less than 0.3 s, designed to withstand the forces of clause 3.4.2.1, can be very high. Where practicable, short-period buildings should be provided with the capacity to withstand greater horizontal loads.

Points to note about the various seismic force factors are:

(a) Risk factor R (see table 4)

The intent of the risk factor is generally to increase the design seismic force in cases where structural failure would lead to an unusually high level of loss either in terms of economic or other losses to the community. The phases of disaster response are referred to in table 4. The Emergency Period covers the time of the earthquake itself and the time immediately after in which predominant activities are search and rescue, emergency food supply or evacuation, and so on, while during the Restoration Period public utilities and commercial services are patched together and damaged buildings are made safe so that the community can become functional once more. The list is of typical complexes of buildings within which individual non-essential buildings may be designed to Category 4.

Category 1: *Structures and installations for the direct support and containment of highly toxic or corrosive products, or of molten metal. (Note that Category 1 is primarily concerned with the safety of those working beside or nearby a particular installation, while Category 2b is concerned with wider effects on a surrounding urban area.)*

Category 2a: *Designated civilian emergency centres and civil defence centres;*
Essential hospital and medical facilities for example, surgery and treatment areas and their support facilities;
Ambulance centres;
Fire stations;
Police stations;
Radio and television facilities;
Telephone exchanges;
Local authority and utility engineering facilities;
Structures housing heavy rescue equipment and its fuel supply.

3.4.2 Total horizontal seismic force

3.4.2.1 Every building shall be designed and constructed to withstand a total horizontal seismic force (V) in each direction under consideration in accordance with the following formula.

$$V = C_d W_t \quad (27)$$

where

$$C_d = CRSM \quad (28)$$

and

C shall be determined from fig. 3 (page 45) in accordance with the seismic zone as shown in fig. 4 (page 46), the subsoil flexibility (see clause 3.4.3), and the period (T) in the direction under consideration (see clause 3.4.4). The minimum values for C shall be those for T = 1.2 s.

S shall be as given in table 5 or rationally deduced therefrom . The value of S shall be determined separately for each direction.

M shall be as given in table 6 or rationally deduced therefrom.

R shall be the largest applicable value given in table 4.

W_t shall be as specified in section 2.4 for loads above the level of lateral restraint imposed by the ground. Tanks, reservoirs, and the like shall be considered to contain their full contents;

provided that the value of C_d need not be taken greater than 4.8 CR for steel and prestressed concrete, and 4 CR for structures of other materials, and shall in no case be taken as less than 0.04.

Category 2b: *Storage and distribution facilities for products such as LPG, CNG, Natural Gas and other highly flammable, explosive, or poisonous materials in urban areas.*

Category 3a: *Central and local government facilities of particular importance following disaster;*
Defence establishments;
Hospital and medical facilities that are not essential facilities;
Electricity and gas supply authority facilities;
Prisons and other places of restraint;
Post Offices (major);
Airport buildings.

Category 3b: *Major art galleries, museums, libraries, and archival record depositories.*
It is recommended that buildings of particular cultural significance be designed to this Standard.

Category 4: *All other buildings.*

(b) Structural type factor S *(see tables 5 and 5B)*

The structural type factor S *is intended to reflect the potential seismic performances of different structural systems. The specified level of* S *primarily takes into account the ability of the structural type concerned to dissipate energy in a number of load cycles, and secondarily its degree of redundancy where appropriate, on the assumption that the bulk of the chosen energy dissipating members in all the principal resisting elements of a given structural type will participate in the dissipation of seismic energy.*

Where the earthquake resistance of a building must be provided by a combination of structural types, the designer is required to select an appropriate value for S *by rational deduction from table 5, and this will necessitate consideration of the degree to which the various elements will contribute to the dissipation of seismic energy in severe earthquakes.*

A method of determining rational design actions for buildings having horizontal force resisting systems in parallel, with differing S *and* M *values in the direction being considered, is as follows:*

Analyse the building, including torsion allowances, assuming S *and* M *equal 1 for all sub-assemblies and then design each using the load effect derived from this analysis and modified by multiplying it by the* S *and* M *values appropriate to the sub-assembly.*

This method is based on the premise that the values of S *and* M *for a given sub-assembly should reflect its available ductility. As at 1984, this method has not been fully researched and therefore should be used*

Table 5 STRUCTURAL TYPE FACTOR *S*

Item	*Description*	*S*
1	Ductile frames	0.8
2	Ductile coupled shear walls:	
	(a) $A \geqslant 0.67$	$0.8Z \leqslant 1.6$
	(b) $A \leqslant 0.33$	$1.0Z \leqslant 2.0$
	(c) $0.33 \leqslant A \leqslant 0.67$	By linear interpolation between 2(a) and 2(b)
3	Ductile cantilever shear walls. Single-storey ductile columns	
	(a) Two or more elements linked together	$1.0Z \leqslant 2.0$
	(b) Single element	$1.2Z \leqslant 2.0$
4	Frames of limited ductility of maximum height four storeys or 18 m or, with top storey, roof and wall mass less than 150 kg/m^2, five-storeys or 22.5 m. Cantilevered shear walls of limited ductility.	2.0
5	Buildings with diagonal bracing:	
	(i) Capable of plastic deformation in tension only:	
	(a) Single storey	2.0
	(b) Two or more storeys	2.5 or by special study
	(c) More than three storeys	By special study
	(ii) Capable of plastic deformation in both tension and compression.	1.6 or by special study
6	Single-storey cantilevered buildings supported by face loaded walls constructed of reinforced masonry or concrete.	2.0
7	Elastically responding structures:	
	(a) Reinforced concrete	5.0
	(b) Reinforced masonry	4.0
	(c) Prestressed concrete	5.0
	(d) Steel	6.0

Where A is the proportion of total overturning moment resisted by all beams (moments referred to the centroidal axes of all walls) and where:

Z = $3.0 - h_w/l_w$ subject to $1 \leqslant Z \leqslant 2$

h_w is the height from base of the wall to top of uppermost principal storey.

l_w is the horizontal length of the wall in the direction of the applied load.

with prudence particularly for buildings over 3 storeys high. It is noted that this method will generally result in a modified base shear V as prescribed by equation 27.

Other points to note about tables 5 and 5B are:

Chimneys can come under one of the items 1 to 8 but very tall, slender structures, where second and higher mode effects can become significant, come outside the code and must be designed by special study. The same applies to tanks, but depending on the size and proportion of the tank, the sloshing action of the content will become significant in loading in which case they must be designed by special study.

(i) Item 1: *The structural type factor of 0.8 applies to all ductile frames subject to capacity design procedures.*

(ii) Item 2: *Requirements for coupling beams are given in clause 3.3.4.1, and requirements for shear walls designed for ductile flexural yielding are given in clause 3.3.4.2. As the proportion of total base overturning moment resisted by the beam diminishes, the structural type factor increases, in recognition of the degree to which energy dissipation in the more vulnerable elements, the walls, is concentrated.*

For the situation where only two walls are present,

$$A = T\ell/M_o$$

Where

T *is the axial load induced in the walls by the coupling beams,*

ℓ *is the horizontal length between the centroids of the walls,*

M_o *is the total overturning moment at the base of the structure due to the same loads used in the determining of* T.

(iii) Item 3: *Single storey ductile columns provide seismic resistance by cantilever action.*

(See also item 2 of table 9.) Requirements for shear walls designed for ductile flexural yielding are given in clause 3.3.4.2. The parameter Z, incorporating the wall aspect ratio hw/lw *is introduced in recognition of the reduced energy dissipation, for a given displacement occurring where shear effects are significant such as in squat walls.*

(iv) Item 4: *Requirements are given in clause 3.3.6.*

(v) Item 5(i): *Design requirements are given in clause 3.3.5. The deformation modification factors given in clause 3.8.1 take account of the characteristics of these structures.*

(vi) Item 5(ii): *These systems, when suitably designed and detailed, may give reduced displacement respon-*

Table 5B

SM or $S_p M_p$ FACTORS FOR TIMBER

Item	*Description*	*SM or S_pM_p*
B1	Shear walls or diaphragms:	
	(a) Ductile	1.0
	(b) Ductile and stiffened with elastomeric adhesive	1.0
	(c) Limited ductility fixed with elastomeric adhesive	1.2
B2	Moment resisting frames:	
	(a) Ductile with an adequate number of possible plastic beam hinges	1.2
	(b) As for item B2 (a) but with connections of limited ductility	1.5
B3	Diagonally braced with timber members capable of acting as struts or ties:	
	(a) With ductile end connections	1.7
	(b) With end connections having limited ductility	2.0
B4	Elastically responding structures	2.4

ses, but at present special studies are required to avoid undesirable effects such as column hinge mechanisms.

(vii) Item 6: *Resistance to earthquake loadings is provided by the wall bending out of plane (see also item 2 of table 9). This item does not apply to retaining walls which should be subject to a special study.*

(viii) Item 7: *Elastically responding structures of reinforced concrete or reinforced masonry are expected to have equal maximum force levels corresponding to SM = 4. Because of the different* M *factor for the two materials, it has been necessary to set different* S *factors. Prestressed concrete and steel structures typically have lower elastic damping resulting in higher levels of elastic response, reflected in higher* S *factors in item 7 of table 5.*

(ix) Item B1 (a): *This item includes shear walls and diaphragms which are formed by either diagonal or orthogonal timber boarding, or sheet materials, fastened to timber framing and boundary members with a large number of metal dowel fasteners (nails, screws, bolts and the like) which act in shear. The building behaviour is 'ductile' as defined in clause 3.2.3. Boundary fastenings and members are required to be capacity designed.*

(x) Item B1 (b): *This item includes shear walls and diaphragms that would be included in item B1 (a) except that the design load is carried by the metal fasteners but the structure is stiffened by the use of elastomeric adhesives. Capacity design of boundary fastenings up to the strength of the adhesive is required, but they need not be designed for* SM *greater than 2.4.*

(xi) Item B1 (c): *The design load is carried by the elastomeric adhesive but nailing (or similar) provided for the clamping of glued joint during assembly must also be capable of carrying 50 percent of the design load.*

(xii) Item B2 (a): *This item includes moment resisting joints formed by nails or bolts acting in shear and the joint possessing substantial ductility. Structure ductility must be as required by clause 3.2.3 and there must be an adequate number of possible plastic beam hinges as defined by clause C3.4.2(b).*

(xiii) Item B2 (c): *This item includes moment resisting joints formed by fasteners having limited ductility such as pressed metal plates, shear plates, split ring connectors and the like.*

(xiv) Item B3 (a): *Members must be able to act as both struts and ties to the capacity of end fastenings. These fastenings to be metal dowels in shear possessing substantial ductility.*

Table 6

STRUCTURAL MATERIAL FACTOR M or M_p

Item	*Material*	*M or M_p*
1	Structural steel	0.8
2	Structural timber: As given by table 5B	
3	Reinforced non-prestressed concrete	0.8
4	Prestressed concrete	1.0
5	Reinforced masonry	1.0

(xv) Item B3 (b): *As for item B3 (a) but with end fasteners having limited ductility. Examples are pressed metal plates, shear plates and the like.*

(c) Structural material factor M and M_p *(see table 6)*

The value of M for ductile prestressed concrete structures is 25% higher than for ductile reinforced concrete structures to allow for the increased response of prestressed concrete structures.

C3.4.3 Recent earthquakes, particularly Caracas 1967 and Manila 1967 and 1970, have again demonstrated that flexible ground can increase the intensity of earthquake attack. Increased intensity on flexible ground is also indicated by analysis based on the measured dynamic characteristics of soils. The increase might not occur for very high intensity earthquakes.

These microzone effects will occur in all seismic zones, but for moderate earthquakes in zones A and B the inherently greater capacity of buildings in these zones affords greater protection.

Plastic deformation of the soil is expected to limit microzone increases on flexible ground for high intensity earthquakes. Hence the microzone effects will be much larger in areas where the design earthquake has a moderate intensity, namely seismic zone C.

Since the previous C values in NZS 1900: Chapter 8 were intended as an envelope for both hard and intermediate ground, additional provisions introduced in this standard are primarily for soft ground.

3.4.3 Subsoil flexibility

3.4.3.1 A building shall be considered to be on flexible subsoil if there are uncemented soils exceeding one of the following depths below the lowest continuous horizontal subsystem; that is, interconnecting tie beams or continuous slab forming a diaphragm:

6 m of cohesive soils with average undrained shear strength not exceeding 50 kPa.

8.5 m of cohesive soils with average undrained shear strength not exceeding 100 kPa.

12 m of cohesive soils with average undrained shear strength not exceeding 200 kPa.

15 m of cohesionless sands or gravels.

This shall apply whether or not there are piles or piers extending to some deeper hard stratum.

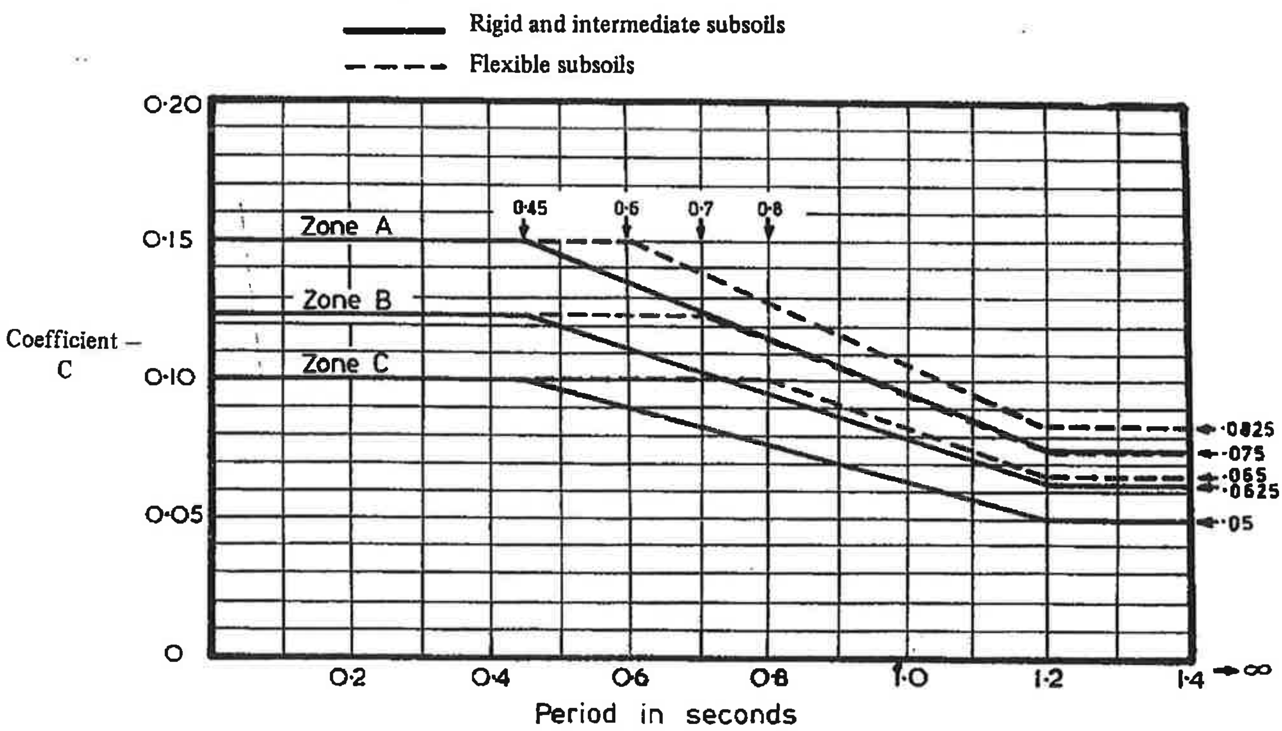


Fig. 3 BASIC SEISMIC COEFFICIENT *C*

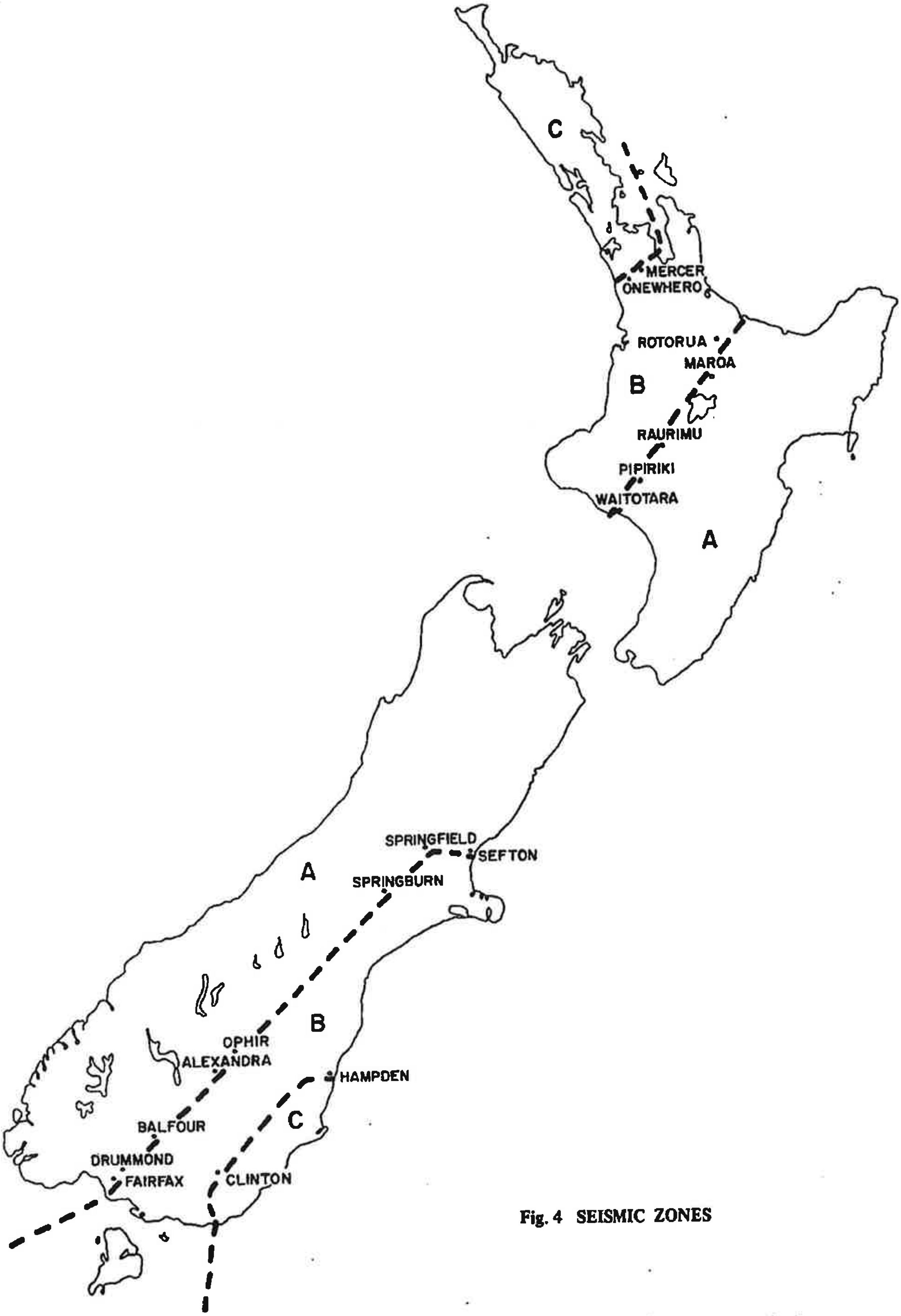


Fig. 4 SEISMIC ZONES

Although it will be some time before microzone effects can be defined accurately, the urgency of the problem justifies some provision at this stage. Almost all national codes make substantial microzone provisions.

The effect of this provision for "deformable sites" is to increase the period range for which the maximum seismic coefficient applies.

For sites where there are soils of erratically varying strengths, or including both cohesive and cohesionless soils, some judgment is necessary to fulfil the intent of the provision.

For long period structures on very deep uncemented soils this provision might not be adequate and special studies should be made.

C3.4.4.1 If in equation (29), the system of forces, F_x, *is replaced by some multiple of itself and the deflections* d_x *are replaced by the deflection responses to the replacement load system, the quotient is unaffected. So an arbitrarily chosen total horizontal force, distributed in the manner specified for* V *in clause 3.4.6, can be used in analysis to establish* T, *thus* C *and* V. *The ratio of* V *to the arbitrarily chosen total force is then a qualifier which converts all the analysis results to values required for design, which are therefore available from the same analysis that determined* T. *The value of* T *given by equation (29) is reliably close to the free vibration period in the fundamental mode whenever the horizontal force distribution system reasonably approximates the fundamental mode's inertia pattern. For a seriously irregular building, the* T *estimate will be shorter than the actual period by a moderate amount. For any reasonably regular building, an approximate value of* T *can be estimated from the formula:*

$$T = 0.063\sqrt{\Delta}$$

where Δ *is in millimetres and means the horizontal movement at the top of the main portion of the building under the following set of loads:*

The horizontal load at each level (x) is obtained by multiplying W_x *at that level by a factor* (h_x/h_n) *that varies linearlv from zero at the bottom to unity at the top.*

3.4.4 Building period

3.4.4.1 The period T in seconds shall be established from properly substantiated data or computation or both.

T may be calculated by the Rayleigh Formula:

$$T = 2\pi\sqrt{\frac{\Sigma\,(w_x d_x{}^2)}{g\,\Sigma\,(F_x d_x)}} \quad \ldots\ldots\ldots\ldots\ (29)$$

where d_x is the horizontal displacement of the mass at level x under the horizontal seismic loading specified by this Standard.

3.4.5 Clause deleted.

3.4.6 Distribution of horizontal seismic forces

3.4.6.1 The total horizontal seismic force V shall be distributed over the height of the building in accordance with the following formula:

$$F_x = \frac{V\,W_x h_x}{\Sigma\,W_x h_x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (30)$$

provided that:

(a) Where the height to depth ratio of the horizontal force resisting system is equal to or greater than 3, then 0.1 V shall be considered as concentrated at the top storey and the remaining 0.9 V shall be distributed in accordance with equation 30.

(b) For chimneys and smoke-stacks resting on the ground, 0.2 V shall be considered as concentrated at the top and the remaining 0.8 V shall be distributed in accordance with equation 30.

(c) For buildings with set-backs the load distribution shall comply with clause 3.4.11.

(d) The distribution of horizontal seismic forces in major buildings that have highly irregular shapes, large differences in lateral resistance or stiffness between storeys, or other unusual structural features shall be determined in accordance with the dynamic analysis procedure of section 3.5.

C3.4.6.1 (d) The provisions of clause 3.4.6.1 (d) will result in base shears similar to those that would be given by the equivalent static force method of analysis, but the distribution of forces will be more appropriate to the particular features of the irregular structures.

Nevertheless it will be necessary to make an equivalent static force analysis for irregular structures in order to obtain the limiting values in clauses 3.5.2.4 and 3.5.2.5. The term "major buildings" is intended to exclude low buildings that do not warrant the more complex procedure of a dynamic analysis. See also section 3.1.

3.4.6.2 At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level.

3.4.6.3 Floors and roofs acting as diaphragms and other principal members distributing seismic forces shall be designed in accordance with clause 3.4.9. Allowance shall be made for any additional forces in such members that may result from redistribution of storey shears.

*3.4.7 Horizontal torsional moments

3.4.7.1 The applicable method of design for torsional moments shall be:

(a) For structures not more than four storeys high or for reasonably regular structures more than four storeys high which are symmetric or of moderate eccentricity, horizontal torsion effects shall be taken into account either by the static method of clause 3.4.7.2, or by the two-dimensional modal analysis method of clause 3.5.2.2.1 (which also uses clause 3.4.7.2), or by the three-dimensional modal analysis method of clause 3.5.2.2.2.

(b) For reasonably regular structures more than four storeys high with a high degree of eccentricity, horizontal torsional effects shall be taken into account either by the static method of clause 3.4.7.2, or by the two-dimensional modal analysis method of clause 3.5.2.2.2. However, it is recommended that the three-dimensional modal analysis of clause 3.5.2.2.2 be used for such structures.

(c) For irregular structures more than four storeys high, horizontal torsional effects shall be taken into account by the three-dimensional modal analysis method of clause 3.5.2.2.2.

C3.4.7.1 Horizontal torsional effects are difficult to estimate. Both excitation and response are known with far less certainty than for translational behaviour. The effects are important however; a number of failures have been caused by horizontal torsion particularly at the ends and corners of buildings, and at re-entrant angles.

A designer's first aim should be to achieve symmetrical structures of similar resisting elements.

Three types of design approach are considered in this standard: a wholly static approach; a combined approach in which the vertical distribution of horizontal forces is given by a two-dimensional modal analysis (clause 3.5.2.2.1) and torsional effects are obtained from the static provisions of clause 3.4.7, and a three-dimensional spectral modal analysis (clause 3.5.2.2.2).

The static method given in clause 3.4.7.2 is intended to apply to reasonably regular buildings such as square, circular, or rectangular structures which have no major re-entrant angles and which are substantially uniform in plan.

Structures of moderate eccentricity are those for which the torsional component of shear load in the element most unfavourably affected does not exceed three quarters of the lateral translational component of shear load.

For exceptionally flexible buildings which are highly irregular and not more than four storeys high (clause 3.4.7.1 (a)) it is recommended that a three-dimensional modal analysis should be used, as the dynamic behaviour in such cases is likely to be more complex than for stiff buildings.

It should be noted that even a three-dimensional modal analysis may not always give good predictions of the dynamic behaviour of very irregular buildings, and may indeed seriously under-estimate earthquake effects in some cases.

It should also be remembered that in torsional situations energy dissipation cannot usually be distributed evenly among resisting elements.

For severely eccentric buildings with L, T, U, or similar irregular plan form, seismic separation of the wing is recommended regardless of the method of analysis.

As less ductile buildings are particularly vulnerable to torsional effects, buildings of more than three storeys and with a structural type factor S equal to or greater than 1.6 should be so designed as to have no worse than moderate eccentricity.

C3.4.7.2 The torsional effects which would be obtained by applying the horizontal force at the centre of mass at each level may be increased due to a number of causes. Some of these are of an accidental nature, such as those resulting from the limitations of stiffness calculation, degree of accuracy of assumptions, mass variation, construction variations, and, in severe earthquakes, asymmetric failure of torsion resisting elements.

Interaction between torsional and translational modes can lead to amplification effects, and torsional ground motion is a further cause of building torsion. The term 0.1b is introduced to allow for all these effects.

C3.4.9 In the preparation of clause 3.4.9 the following assumptions were made:

(a) For practical purposes pseudo-acceleration is an acceptable measure of damage potential to fixed parts and portions in a building. (See also clause 3.8.4.)

(b) The maximum accelerations in a building prior to yielding are related to its elastic damped response, and after yielding to the C_d values.

(c) Because of the requirements of New Zealand standards for design in various materials, and because of design and detailing practices (such as under-capacity factors, use of minimum yield stresses, rounding-up of sizes and the like), building over-capacities corresponding to 1.5 C_d should be common in future buildings, resulting in accelerations greater than $K_x C_d$.

(d) These accelerations will be reached in earthquakes of moderate intensity and will thus be relatively frequent in modern buildings having low damping. (This

3.4.7.2 To provide for shear resulting from torsional motion, the horizontal force at the level considered shall be applied in turn at each of two points distant 0.1*b* from the centre of mass at that level and either side of it, measured perpendicular to the direction of loading.

3.4.8 Clause deleted.

3.4.9 Parts or portions of buildings

3.4.9.1 Except as provided by clause 3.4.9.3 and subject to section 3.6, any part or portion of a building shall be designed for a seismic force F_p in each direction under consideration as given by:

$$F_p = C_p W_p \quad (33)$$

where

C_p shall be as given by clause 3.4.9.2 and

W_p shall be determined in the same manner as W_t (see clause 3.4.2.1).

assumption is supported by considerable evidence of building response from earthquakes in New Zealand and overseas.)

The absolute upper limits of this standard will rarely govern in the case of frame structures, but frequently in the case of structures for which the standard requires high C_d values, that is, "shear failing" systems, and damping values presumed appropriate to these have been selected. To establish the upper limits the following assumptions have been made:

(i) The strueture though highly strained responds elastically with 5 percent damping in accordance with the "Skinner" spectrum for rigid and medium soils scaled to g/3 ground acceleration.

(ii) The value of K_x for the top of medium height multi-storey buildings is 1.7. For high structures this value is obviously too low but for these the selected amplification factors are too high and the effects cancel.

It therefore follows that for single-storey structures maximum roof accelerations will not exceed 1.8 x g/3 = 0.6 g and for multi-storey structures 2.33 x 1.7 x g/3 = 1.33 g. The required $C_{p\ max.}$ values of the standard have been derived using these accelerations multiplied by the risk factor R_p and reduced in accordance with a factor that takes the ductility of the part into account. The maximum values of 1.33 R_p are thus applicable to brittle parts located at the top of multi-storey buildings. For the same part located on a single-storey building, a value of 0.6 R_p would apply. For parts possessing ductility, lower levels for $C_{p.\ max.}$ were set.

Ductility for the purpose of this clause was considered to be capable of being achieved with far less stringent detailing requirements than for the principal members of a structure required to dissipate significant amounts of seismic energy.

Ductile parts such as adequately reinforced walls, suitably reinforced wall beams, and the like are allowed to yield at about the same levels of response as will ductile buildings (with some overstrength) if designed to this standard (one-fifth of the 2 percent damped response of 1940 El Centro type motions). The resulting values for $C_{p.\ max.}$ for these parts are thus 0.12 R_p but not less than 0.2 R_p (single-storey) and 0.3 R_p (multi-storey).

Between the extremes of completely brittle parts and ductile parts, categories for partially ductile and low ductility elements have been introduced. As always, performance data from earthquakes have been used as a yardstick and the generally rare occurrence of damage to reinforced concrete floor diaphragms has resulted in their being classified as "ductile" regardless of proportion. In any case, excessively high design levels for reinforced concrete diaphragms would have the undesirable effect of increasing perimeter beam capacities (due to introduction of additional slab steel) and thus further complicating the problem of resisting beam–column joint shears.

Points to note about items in table 8 are:

Item 1 (a): *Except where explicitly required otherwise in table 9, centrally reinforced walls meet this requirement.*

3.4.9.2 C_p in equation 33 shall be evaluated thus:

(a) For items for which a value for R_p is given in table 9,

$$C_p = 1.5\,K_x S_p M_p R_p C_d$$

subject to $\alpha K_x Z R C_{p\ min.} \leqslant C_p \leqslant \alpha K_x Z R C_{p\ max.}$

(b) Where no value for R_p is given

$$C_p = \alpha K_x Z R C_{p\ max.}$$

In these expressions

K_x is 1 for single storey buildings; and $\frac{h_x}{h_{cq}}$ but not less than 1, for multi-storey buildings

α is 1 for single storey buildings; and $\frac{h_{cq}}{h_n}$ for multi-storey buildings

S_p is from table 8

M_p is from table 6

$C_{p\ max.}$ $C_{p\ min.}$ R_p are from table 9

R is from table 4

Z is 1 for seismic zone A, $\frac{5}{6}$ for seismic zone B, $\frac{2}{3}$ for seismic zone C

C_d is for the supporting building (see clause 3.4.2.1)

Table 8

STRUCTURAL TYPE FACTOR S_p FOR A PART OR PORTION OF A BUILDING

Item	*Description*	S_p
1	(a) Adequately reinforced walls and partition walls subjected to face loads (b) Floor and roof diaphragms (see clause 3.4.6.3) (c) Other principal members distributing seismic forces such as reinforced concrete or reinforced masonry wall bands detailed for ductility (d) Other reasonably ductile parts	1.0
2	Diaphragms with diagonal bracing capable of deformation in tension only and detailed in accordance with clause 3.3.2.2	1.2
3	(a) Principal members distributing seismic forces such as reinforced concrete or reinforced masonry wall bands not detailed for ductility (b) Tied veneers subject to face loadings	2.0
4	Unreinforced or partially reinforced walls and partitions subject to any loading	3.0

Item 1 (b): *Diaphragms such as reinforced concrete slabs should be reinforced in two directions at reasonably close spacings.*

Item 1 (c): *Members should be reinforced as required for ductile yielding. Where this is not practical* $S_p = 2.0$ *applies (item 3 (a)).*

Four broad categories of risk were established, ranging from normal risk $R_p = 1$ *(for example, heavy partitions) to extreme risk* $R_p = 2$ *(for example, failure resulting in release of toxic substances, or veneer overhanging streets). Intermediate risk categories were for* $R_p = 1.33$ *(for example, heavy partitions adjacent to means of egress and other items with* $R_p = 1.5$ *(see table 9).*

Members were placed into four categories depending on their likely performance when stressed into the inelastic range and in accordance with this presumed performance a level of $C_{p.\,max}$ *was set.*

Thus brittle elements in class III buildings in seismic zone A should have the capacity to resist the forces resulting from a maximum amplified El Centro type ground motion elastically.

Designers should be aware that in structures where the structural capacity is not related to the seismic design because wind load governs, assumption C3.4.9 (b) is not valid.

The real post-elastic situation is complex, and floor-motion studies should be made in special cases.

3.4.9.3 Parts or portions of buildings that are supported directly on the ground and are completely separated from the building shall be treated as independent buildings.

C3.4.9.3 Parts or portions completely separated from the building or supported directly on the ground or floor that effectively moves with the ground can in general be considered to be independent of the dynamic response of the building. In such cases, no amplification of the ground motion need be allowed for and this structure should be designed as a separate building.

Points to note about items in table 9 are:

Item 2: *Vertical cantilevers include cantilevered storeys, walls, partitions, and parapets.*

Item 6: *An example of primary elements distributing seismic forces is a wall beam located at floor or roof level which in the absence of an effective diaphragm at that level is designed to fulfil its function. Secondary distribution* *beams such as bond beams in walls spaced at 1200 mm centres or less should be designed for the loadings appropriate to reinforced walls.*

Item 7: *For particularly flexible and vital equipment a special study should be made. The values given for this item in table 9 are adequate only where the parts are reasonably ductile.*

Items 9 and 10: *As far as possible, provisions should be made to safeguard building contents not covered by table 9; in particular, stored items should be prevented from falling off shelves.*

C3.4.11.1 It is intended that other forms of irregularity be treated in accordance with appropriate clauses.

C3.5 In general, if modes other than the fundamental are likely to be significant a dynamic analysis should be considered. The two main methods of dynamic analysis in current use are spectral modal analysis and numerical integration response analysis. This standard allows only spectral modal analysis to give relative load distributions for quantitative analysis because numerical integration response analysis has been insufficiently calibrated for code purposes. Other dynamic methods are currently under development. Random vibration analysis, for example, shows promise where a seismic risk analysis is being carried out. Such methods may, however, be used only to supplement results obtained by the static or dynamic provisions of this standard, they may not be used as alternative procedures.

In general, if modes other than the fundamental are significant a spectral modal analysis will give a more realistic distribution of loads than that specified in clause 3.4.6.

C3.5.2.1 Unscaled values of C_d *are not suitable for use as a design spectrum since they are related to the total building mass rather than the masses associated with modal responses. Moreover, the* C_d *values include some allowance for the effects of inelastic deformation on the building response.*

For very long periods this spectrum might not be appropriate, and a special study is recommended in such cases.

3.4.10 Overturning moments

3.4.10.1 The seismic overturning moments shall be derived from the horizontal seismic forces and the distribution of horizontal seismic forces according to clauses 3.4.2, 3.4.6, and 3.4.7 without reduction.

3.4.11 Set-backs

3.4.11.1 For buildings with set-backs where the plan dimension of the tower in each direction is at least 75 percent of the corresponding plan dimension of the lower part, the effect of the set-back may be neglected for the purpose of determining seismic forces by the equivalent static force method (see also clause 3.4.6.1).

3.4.11.2 For other conditions of set-backs: the distribution of horizontal seismic forces shall be determined in accordance with the dynamic analysis procedure of section 3.5.

3.5 DYNAMIC ANALYSIS

3.5.1 General

3.5.1.1 Dynamic analysis shall be by spectral modal analysis in accordance with clause 3.5.2.2, which may be supplemented by numerical integration response analysis in accordance with clause 3.5.3.

3.5.2 Spectral modal analysis

3.5.2.1 *Design spectrum*

3.5.2.1.1 The structural design spectrum shall be given by the expression KC for each mode where C is given by fig. 3 and K is as given by clause 3.5.2.4.1 and is the same for all modes.

Table 9

SEISMIC FORCE FACTORS FOR PARTS AND PORTIONS OF BUILDINGS (see clause 3.4.9)

Item	*Part or portion*	*Direction of force*	C_p *max.*	R_p	C_p *min.*
1	Walls, partition walls, infilling panels, exterior prefabricated panels:	Normal to face			
	(a) Adjacent to an exitway, street, or public place, or required to have a fire resistance rating of 2 h or more.				
	(i) Single-storey buildings:				
	S_p less than or equal to 1		0.3		
	S_p greater than 1		0.9		
	(ii) Multi-storey buildings:				
	S_p less than or equal to 1		0.5	1.1/3	0.3
	S_p greater than 1		1.8	1.1/3	0.6
	(b) Other:				
	(i) Single-storey buildings:				
	S_p less than or equal to 1		0.2		
	S_p greater than 1		0.6		
	(ii) Multi-storey buildings:				
	S_p less than or equal to 1		0.4	1.0	0.3
	S_p greater than 1		1.3	1.0	0.6
2	Vertical cantilevers and elements fastened to them.	Normal to face of wall			
	(a) Horizontal restraint from ductile cantilevered columns or walls on				
	(i) One-storey building, and cantilevered top storey in a two-storey building		0.3		
	(ii) Multi-storey buildings and other cantilevered top storeys		0.6	1.5	0.3
	(b) Horizontal restraint of lesser ductility				
	(i) One storey building, and cantilevered top storey in a two-storey building		0.6	2.0	0.4
	(ii) Multi-storey buildings and other cantilevered top storeys		1.3	2.0	0.9
3	Ornamentations, tied veneers, appendages:	Any horizontal			
	(i) Single-storey buildings		1.0		1.0
	(ii) Multi-storey buildings		2.0		2.0
4	Connections for exterior pre-fabricated panels (item 1) and all item 3:				
	(i) Single-storey buildings		1.0		
	(ii) Multi-storey buildings		2.0		
5	Horizontally cantilevered floors, beams etc. (see clause 3.6.3):	Vertically downward or upward	0.9		
6	Floors and roofs acting as diaphragms and other primary elements distributing seismic forces, see clause 3.4.6.3:	Any horizontal			
	(a) Single-storey buildings				
	S_p equal to 1.0		0.2		
	S_p equal to 1.2		0.25		
	S_p greater than 1.2		0.4		
	(b) Multi-storey buildings				
	S_p equal to 1.0		0.3		
	S_p equal to 1.2		0.4		
	S_p greater than 1.2		0.6		

Item	Part or portion	Direction of force	C_p max.	R_p	C_p min.
7*†	Towers not exceeding 10% of the mass of the building. Tanks and full contents, not included in item 8 or item 9; chimneys and smoke stacks and penthouses connected to or part of the building except where acting as vertical cantilevers:	Any horizontal			
	(a) Single-storey buildings where the height to depth ratio of the horizontal force resisting system is:				
	(i) Less than or equal to 3		0.2		
	(ii) Greater than 3		0.3		
	(b) Multi-storey buildings where the height to depth ratio of the horizontal force resisting system is:				
	(i) Less than or equal to 3		0.3		
	(ii) Greater than 3		0.5		
8*†	Containers and full contents and their supporting structures; pipelines, and valves:	Any horizontal			
	(a) For toxic liquids and gases, spirits, acids, alkalis, molten metal, or poisonous substances, liquid and gaseous fuels including containers for materials that could form dangerous gases if released:				
	(i) Single-storey buildings		0.6	2.0	0.5
	(ii) Multi-storey buildings		1.3	2.0	0.9
	(b) Fixed fire fighting equipment including fire sprinklers, wet and dry riser installations and hose reels:				
	(i) Single-storey buildings		0.5	1.5	0.3
	(ii) Multi-storey buildings		1.0	1.5	0.6
	(c) Other:				
	(i) Single-storey buildings		0.3	1.0	0.2
	(ii) Multi-storey buildings		0.7	1.0	0.4
9*†	Furnaces, steam boilers, and other combustion devices, steam or other pressure vessels, hot liquid containers; transformers and switchgear; shelving for batteries and dangerous goods:	Any horizontal			
	(i) Single-storey buildings		0.6	2.0	0.5
	(ii) Multi-storey buildings		1.3	2.0	0.9
10*†	Machinery; shelving not included in item 9; trestling, bins, hoppers, electrical equipment not specifically included in other items 8, 9 or 11; other fixtures:	Any horizontal			
	(i) Single-storey buildings		0.3	1.0	0.2
	(ii) Multi-storey buildings		0.7	1.0	0.3
11†	Lift machinery, guides etc; emergency standby equipment.	Any horizontal	0.6		
12†	Connections for items 8 to 11 inclusive shall be designed for the specified forces provided that the gravity effects of dead and live loads shall not be taken to reduce these forces.				
13†	Suspended ceilings including attached equipment, lighting and attached partitions, see clause 3.6.5.	Any horizontal	0.6		
14	Communications, detection or alarm equipment for use in fire or other emergency:				
	(i) Single-storey buildings,	Any horizontal	0.5	1.5	0.3
	(ii) Multi-storey buildings,		1.0	1.5	0.6

* See clause 3.6.4.

† See clause 3.6.6.

3.5.2.2 *Torsional effects*

3.5.2.2.1 For symmetrical or moderately unbalanced buildings for which torsional effects are calculated by the static method of clause 3.4.7 account shall be taken of not less than the first three modes for each direction under consideration.

C3.5.2.2 See clause 3.4.7.2.

3.5.2.2.2 Where dynamic torsional effects are included in the spectral modal analysis, account shall be taken of not less than four modes for each direction under consideration, two of them predominantly translational and two predominantly torsional. The model shall include the effects of accidental eccentricities of $\pm 0.1\ b$. For moderately unbalanced buildings the torsional effect shall be not less than that calculated by the static method of clause 3.4.7.

3.5.2.3 *Shear*

3.5.2.3.1 The shear at any level shall be taken as the square root of the sum of the squares of the modal shears at that height.

3.5.2.4 *Scaling factor*

C3.5.2.4 When a building is designed to resist the more accurate distribution of loads given by spectral modal analysis then an improved performance will result. Base shear values are therefore reduced to 90 percent of the values given in section 3.4.

3.5.2.4.1 The value of the scaling factor K shall be chosen so that in accordance with clauses 3.5.2.1 to 3.5.2.3 inclusive the computed base shear V is not less than $0.9\ C_d W_t$

3.5.2.5 *Minimum shear values*

C3.5.2.5 At some levels of a building the spectral modal analysis might give load values much lower than those given by the equivalent static force method. These low local values are obtained partly as a result of neglecting some of the effects of inelastic deformation on the building response, and therefore full advantage cannot be taken of the apparent local reduction in loads.

3.5.2.5.1 At any level the shear derived in accordance with clause 3.5.2.3 shall be taken as not less than 80 percent of the values computed by the equivalent static forces method specified in section 3.4.

3.5.2.6 *Horizontal forces and overturning moments*

3.5.2.6.1 The horizontal forces and overturning moments shall be derived from the shears given by clauses 3.5.2.1 to 3.5.2.5 inclusive.

3.5.3 Numerical integration response analysis

C3.5.3 The value which should be selected for the equivalent static base shear (C_d) becomes increasingly uncertain as the fundamental period of the building increases beyond 1.5 s. For this period range the C values of fig. 3 are intended to be conservative. Such long-period buildings should have their horizontal seismic loads selected on the basis of special studies.

The following may be adopted as a guide in the selection of design earthquakes, the modal damping, and the reduction factor for ductility: If the building stiffness, or the building height, is scaled to reduce its fundamental period to 1.5 s then the procedures adopted for the dynamic analysis of the non-scaled building should result in a base shear for the scaled building which is not less than 90 percent of the equivalent static base shear; that is 0.9 $C_d W_t$.

3.5.3.1 Numerical integration response analysis may be used to obtain additional information on building behaviour, particularly in the post-elastic range, to supplement that obtained by spectral modal analysis.

3.6 SPECIFIC REQUIREMENTS FOR PARTICULAR ELEMENTS

3.6.1 General

3.6.1.1 Clause 3.4.9 shall be subject to the specific requirements of this section for the particular elements covered below.

3.6.2 Connectors

3.6.2.1 Connectors to ornamentations, veneers, appendages, and exterior panels, including anchor bolts, shall be corrosion-resisting and ductile, with adequate anchorages. In the case of precast concrete panels, anchorages shall be attached to or hooked round panel reinforcing.

3.6.2.2 For anchor bolts for equipment (item 12 of table 9) designed in accordance with clause 3.4.9.3 the value of C_{p2} may be taken as 1.

C3.6.2.2 Anchor bolts for rigid equipment can be expected to have little or no inelastic demand made upon them in cases where F_p has been derived from the following values of C_p for class III buildings in seismic zone A or the corresponding values (see clause 3.4.9) for other cases:

C_p *not less than 0.35 for equipment directly supported on the ground.*

C_p *not less than 0.6 for equipment in single-storey buildings.*

C_p *not less than 1.4 for equipment in multi-storey buildings.*

Where F_p has been derived from values of C_p less than those listed above, or for particularly important equipment, the design of anchor bolts and their method of embedment should be appropriate to avoid sudden premature inelastic failure.

3.6.3 Brittle components

3.6.3.1 Brittle components, items 7–10 of table 9, shall be designed to withstand twice the loading values obtained from clause 3.4.9. If separated from the structure by an energy dissipating system, the brittle components shall be designed to withstand a load equal to twice the working load of the dissipator.

C3.6.3.1 The purpose of this clause is to make certain that adequate provision is made for possible stress reversal which may arise from dynamic effects. Similar reasoning applies to item 12 of table 9. The provision is intended to ensure minimum anchorage for these items.

3.6.4 Horizontal cantilevers

3.6.4.1 The gravity effects of dead and live loads shall not be taken to reduce the specified vertically upward load on horizontal cantilevers.

3.6.5 Suspended ceilings

3.6.5.1 The support system for suspended ceilings shall be appropriately designed and constructed so as to avoid sudden or incremental failure or excessive local or cumulative deformations that would release ceiling components likely to cause a hazard to the occupants.

3.6.5.2 (Vacant)

C3.6.5 Failure of suspended ceiling systems has been the result of many earthquakes in New Zealand and overseas.

This type of damage can be a life hazard to occupants and can add to the potential for panic.

The functions of class I buildings and some class II buildings such as telephone exchanges have been disrupted by failure of suspended ceilings including integral lighting fixtures and hanging light fixtures.

Failure of suspended ceiling systems can be caused not only by failure of members and connections but also by local or cumulative deformations allowing elements to drop between supports.

Vertical ground accelerations approximately as intense as the horizontal ground accelerations have been recorded *in a number of earthquakes. These motions when amplified by the structure and ceiling framing can result in net upward forces. The ceiling framing should be constructed in such a manner that all joints and connections are positively, mechanically fixed to avoid disconnection under dynamic effects. The connections should preferably be such that components will fail prior to the joints.*

Holding-down devices for elements should allow their easy removal and replacement for access into the ceiling space.

Where the suspended system depends on lateral support from the structure, the members transmitting these loads need to be adequately spliced to transmit the specified forces without buckling and/or deforming at splices in a manner that will result in excessive cumulative deformations.

Hangers need not be designed to withstand compressive loads, but they do need to be connected to the ceiling and structure in a manner that allows relative upward movement without disengagement.

3.6.5.3 Primary and secondary support members shall have positive mechanical connections to each other and the building. In addition to normal gravity loadings and any horizontal load they may have to transmit, splices and connections to components, and connections to the building shall be designed for a shear or net upward reaction of one-third the gravity load or support reaction but not less than 50 N.

3.6.5.4 The support system for tiles and lighting fixtures shall be so designed that the distance between supports will not increase by more than 5 mm when the design load required by clause 3.4.9 is applied in any direction and manner tending to spread the supports. Where the tiles are positively connected to the supports, they may be considered as part of the restraining system in preventing the spread of the supports.

3.6.5.5 Elements, lighting fixtures, and other heavy fittings built into the suspended ceiling system shall be positively anchored to their supports against a net upward force equal to one-third their actual weight. Very light suspended ceiling systems where each tile is less than 2 kg and of a shape and type that on falling would not be a hazard to the occupants need not have the tiles positively fixed against earthquake forces.

C3.6.6 Quasi-resonance effects have frequently resulted in damage to flexible and flexibly mounted equipment and should be avoided where possible. The factor of 2 may be inadequate for lightly damped equipment high in a building. Such equipment should always be fitted with suitable mounts or snubbed to limit the acceleration and displacement. Snubbing devices should have resilient surfaces to reduce shock loading.

3.6.6 Flexible and flexibly mounted equipment and their fixings

3.6.6.1 Equipment with a ratio of first mode period to the building design fundamental period in the range 0.6 to 1.4 or equipment mounted on the ground with a first mode period between 0.05 and 2.0 seconds, and the fixings to such equipment, shall be designed to withstand at least twice the force determined using clause 3.4.9.

C3.7 The interconnection of foundations is an important construction requirement to ensure that the building will act as a unit in an earthquake. This is best achieved by connections at ground level where the disturbance originates and where the connections provide a measure of safety if ground movements occur.

For light buildings and retaining walls, foundations restrained by the passive pressure of the ground or by friction between the foundation and the soil can be adequate. Individual foundation blocks under timber floors require adequate fixings to the timber floor diaphragm, and no light building should derive its lateral resistance solely from such individual foundation blocks unless these are specifically designed for lateral loads on the building.

Foundations on sloping sites require special attention.

Piles driven through soft subsoils should be designed for bending due to lateral loads.

3.7 INTERCONNECTION AND HORIZONTAL SUPPORT

3.7.1 All parts of a building, including floors, roofs, partitions, ornamentations, machinery, tanks, veneers, precast elements, and the like, unless specifically designed to act otherwise, shall be tied and interconnected by adequate fixings designed to resist wind or earthquake loads. Connections shall have sufficient ductility and rotation capacity to preclude fracture or brittle failure.

3.7.2 Concrete or masonry walls shall be anchored to all floors and roofs that provide horizontal support for the wall or are required to provide stability for the wall. Such anchorages shall be capable of resisting the horizontal forces given by clause 3.4.9 or a minimum force of 3 kN per metre length of wall, whichever is the larger.

3.7.3 Individual foundations of a building shall be interconnected in two directions generally at right angles by members designed for a horizontal force equal to 10 percent of the vertical load on the foundations under seismic conditions averaged between the columns concerned. Alternatively, foundations may be restrained by other means against differential lateral movement during an earthquake.

3.8 DEFORMATIONS DUE TO EARTHQUAKE LOADS

3.8.1 Computed deformations

C3.8.1.1 Guidance on the calculations of deformations is given in the relevant material's codes.

3.8.1.1 Computed deformations shall be those resulting from the application of the horizontal actions specified in section 3.4 or 3.5 and multiplied by the factor K/SM appropriate to the structural type and material, where K = 2 for the method of section 3.4 and K = 2.2 for the method of section 3.5.

C3.8.1.2 Computed deformations vary proportionally with the value of C_d. Compared to the value of C_d derived from S = 1, M = 1, considerable reductions result for structures with the same importance and located within the same seismic zone, for example, M = 0.8 (structural steel) S = 0.8 (adequate redundancy) and up to 10 percent reduction in some cases where a dynamic analysis has been made. The justification for reductions in strength does not apply to deformations. For those systems for which the principle of equal displacements for the inelastic system and elastic system with the same initial stiffness applies, neither reductions nor increases in C_d values will affect the total displacement in an earthquake. The modification factor is aimed at achieving this, that is, separation requirements proportional to CR.

Designers should be aware that for structures dissipating energy in a ductile flexural mode the separation requirement of this standard gives average damage protection to a class III building with 5 percent damping in seismic zone A at levels of motions up to one-third 1940 El Centro N–S only. Furthermore, buildings where energy dissipation tends to be localized in some storeys are prone to large deformations. Thus wherever practical a greater degree of separation should be provided. Measured responses in New Zealand and overseas confirm the large deformations suffered by modern framed structures owing to their low damping characteristics.

3.8.1.2 Computed deformations shall be calculated neglecting foundation rotations.

3.8.2 Building separation

3.8.2.1 Each building separated from its neighbour shall have a minimum clear space from the property boundary, other than adjoining a public space, either 1.5 times the computed deflections as given in clause 3.8.1 or 0.002 times its height, whichever is the larger, and in any case, not less than 12 mm. Parts of buildings, or buildings on the same site separated from each other shall have a minimum clear space from each other either of 1.5 times the sum of their computed deflections as given by clause 3.8.1, or of 0.004 times their height whichever is the larger and in any case not less than 25 mm. Separation spaces need not extend into the foundations except where the Engineer may direct.

C3.8.2.2 The provisions of clauses 3.8.2, 3.8.3 and 3.8.4 are intended to avoid major structural damage caused by hammering of buildings and interference between structural and non-structural elements; they are also intended to reduce non-structural damage and the resulting hazard to occupants. Panic amongst occupants caused by large building sway has been a further consideration.

Earthquake damage in the Anchorage earthquake (failure of precast claddings and windows) and the Caracas earthquake (serious structural failure due to weak non-structural hollow clay partitions) required a review of separation requirements.

Non-separation of elements is now permitted only in very rigid buildings. The practical difficulties and the expense of large separations have not been ignored, and the required minimum separation distances are significantly smaller than the deformations that could result from the imposed forces of a major earthquake.

3.8.2.2 Separation spaces shall be cleared of construction debris and detailed so as to remain clear during normal use. Construction tolerances shall make allowance for the clear space provisions. Space coverings shall be durable and shall allow three-dimensional movement. Where compressible space fillings are used, specified clearances shall be appropriately increased and the forces resulting from the compression of the filler material allowed for in the design.

C3.8.3 The intention of clause 3.8.3 is to determine a building deformation that is significant in relation to damage to the non-structural elements listed in clause 3.8.4. In general the inter-storey deflection as defined by clause 3.8.3.1 may be taken as an adequate measure of the damage potential to the listed elements.

In special cases, for example where the elements are located in bays containing members subject to large axial deformations, other criteria may apply, such as where elements are located in bays adjacent to relatively slender shear walls. In other cases the deformation of horizontal members may also need to be considered.

A class III reinforced concrete shear wall building with just sufficient stiffnesses to qualify for non-separation may suffer inter-storey deflections of the order of 3 mm for a storey-height of 3.6 m in a 1940 El Centro N–S type motion if it has 10 percent damping and suffers no damage.

The limit storey slope of 1 % is intended to control Pδ effects in seismic zone A. To effect the same measure of control so that limit Pδ generated member forces have the same relativity to primary seismic member forces in other seismic zones as they have in zone A, it is necessary to introduce zone factors to the analysis slope limits.

3.8.3 Inter-storey deflections

3.8.3.1 Inter-storey deflections computed in accordance with 3.8.1 between two successive floors measured horizontally shall not exceed 0.0006 of the storey height where non-structural elements are not separated as specified in 3.8.4 nor 0.010 of the storey height multiplied by the zone factor in any case where the zone factor is:

1 for seismic zone A
$\frac{5}{6}$ for seismic zone B
$\frac{2}{3}$ for seismic zone C.

C3.8.4 It is at present believed that for some parts of New Zealand the damage risk and life risk resulting from the exclusions contained in clause 3.8.4.1 (c) are acceptable. The effects of more distant earthquakes may make this assumption less valid for structures of a relatively longer period than for those of a shorter period. The mode of

3.8.4 Separation of elements

3.8.4.1 Except as provided by clause 3.8.3.1, the following elements shall be effectively separated from the structure in accordance with clause 3.8.4.2.

(a) Elements, such as stairways, rigid partitions, and in-

failure of windows and other brittle claddings subjected to in-plane loadings is uncertain. If, as they have done in some experimental investigations, they suffer explosive failure this would add to the hazard.

Further investigation of these and other aspects is required. Windows falling the height of only one storey should constitute a lesser direct risk but broken glass in the streets obviously is a hazard to people attempting to leave a building following an earthquake and also to those engaged in rescue operations in general. In seismic zones where moderate earthquakes are likely to occur more frequently, damage control aspects in addition to measures required to limit life risk, require the separation of all exterior brittle elements in all storeys.

Inserts in concrete should be attached to or hooked around reinforcing steel, or otherwise terminated to effectively transfer seismic forces. Required anchors in masonry walls of hollow units or cavity walls should enter a reinforced grouted structural element of the wall.

In earthquakes of low Modified Mercalli Intensity, the major component of damage cost observed is associated with internal partitions (for example, in the MM VI zone of the San Fernando 1971 earthquake). Such earthquakes occur relatively frequently (for example, MM VI in Zone A is equalled or exceeded about once every six years on average) and hence it is recommended that internal partitions be detailed to permit an inter-storey deformation of one quarter that computed by clause 3.8.1 without damage.

fillings, that are capable of altering the intended structural behaviour to a significant degree.

(b) Precast concrete claddings and other claddings of similar mass.

(c) Glass windows and other rigid brittle exterior claddings, except in the case of claddings on class III buildings in seismic zone C that in the case of failure cannot fall through a height greater than the storey in which they were installed.

3.8.4.2

(a) Elements described in 3.8.4.1(a) shall be so separated from the structure that there is no impact when the structure deforms to twice the extent computed by clause 3.8.1.

(b) Elements described in 3.8.4.1 (b) and (c) shall be so separated that there is no impact when the structure deforms to the extent computed by clause 3.8.1.

C3.9 It is sometimes thought that the substitution of a larger or a stiffer element for that shown in the drawings is self-evidently acceptable. This is not so: any change, whether it involves an increase or a decrease in the strength or stiffness of an individual member from that shown in the drawings, may have a seriously detrimental effect on the earthquake performance of the building. It is the structural designer's responsibility to ensure that this is appreciated by all concerned.

3.9 STRUCTURAL VARIATIONS TO BE APPROVED BY DESIGNER

3.9.1 The structural designer shall make the builder or contractor aware of the dangers that can arise when structural elements are changed or varied without the specific approval of the structural designer.

PART 4 WIND LOADS

4.1 SCOPE AND INTERPRETATION

C4.1.1 This Part provides for wind loads to be determined both on the basis of established data as set out in this standard and elsewhere and also on the basis of special studies by wind tunnel or similar tests conducted by an approved authority. The facilities for carrying out such special studies exist in the University Schools of Engineering and in certain Government Departments. In outline, the procedure for determining wind loads follows the following steps:

(a) Determine the basic wind speed V *in accordance with the nature and location of the building.*

(b) Determine the design wind speed V_s *in accordance with the topography (factor* S_1*) and ground roughness, building height, and height above ground (factor* S_2*).*

(c) Determine the dynamic pressure q *corresponding to* V_s.

(d) Determine from q *the following pressures and forces as required:*

(i) The pressure p *at any point on the surface of the building.*

(ii) The resultant pressure force F *exerted normal to the surface of any structural element or cladding unit.*

(iii) The frictional force F' *acting along that flat surface in the direction of the wind.*

(iv) The resultant pressure force F *exerted on the building as a whole:*

either *by vectorial summation of the pressure forces on all the surfaces,*

or *by use of the appropriate force coefficient* C_f.

(v) The frictional force F' *exerted on the building as a whole.*

* Breadth and depth are dimensions measured in relation to the direction of the wind, whereas length and width are dimensions related to the plan form.

4.1.1 Scope

4.1.1.1 This Part includes provisions for slender exposed structural elements such as chimney stacks, observation towers, and the like, but it does not apply to wind loads on buildings of unusual shape or location or which may be subject to wind-induced oscillations or to steady transverse forces; the wind loads on such a building shall be determined by a special study.

4.1.2 Definitions

4.1.2.1 In this part, unless inconsistent with the context:

BREADTH* means the horizontal dimension of the building measured normal to the direction of the wind.

DEPTH* means the horizontal dimension of the building measured in the direction of the wind.

DYNAMIC PRESSURE of wind means the free stream dynamic pressure resulting from the design wind speed.

EFFECTIVE FRONTAL AREA means the area normal to the direction of the wind (the "shadow area").

ELEMENT OF SURFACE AREA means the area of surface over which the pressure coefficient is taken to be constant.

FORCE COEFFICIENT means a non-dimensional coefficient such that the total wind force on a body is a product of the force coefficient times the dynamic pressure of the incident wind times the effective frontal area.

GROUND ROUGHNESS means the nature of the earth's surface as influenced by small-scale obstructions such as trees and buildings (as distinct from topography).

HEIGHT:

HEIGHT OF A BUILDING means the height of a building above the adjacent ground.

HEIGHT ABOVE GROUND means the height above the general level of the ground to windward (see clause 4.4.4.4).

LENGTH* means the greater horizontal dimension of the building above the ground adjacent to that building.

RETURN PERIOD means the average number of years within which the given wind gust is expected to be equalled or exceeded.

TOPOGRAPHY means the nature of the earth's surface as influenced by the hill and valley configurations.

WIDTH* means the lesser horizontal dimension of a building above the ground adjacent to that building.

4.1.3 Symbols

4.1.3.1 In this Part symbols shall have the following meanings, provided that other symbols, or other meanings for symbols listed below, that are defined immediately adjacent to formulae or diagrams shall apply in relation to those formulae or diagrams only:

A	element of surface area
A_e	effective frontal area
b	breadth of building
C_{av}	average external pressure coefficient
C_f	force coefficient
C'_f	frictional coefficient
C_p	pressure coefficient
C_{pe}	external pressure coefficient
C_{pi}	internal pressure coefficient
d	depth of building or reference dimension
D	diameter
F	pressure force
F'	frictional force
h	height of building
l	length
p	pressure on a surface
p_e	external pressure
p_i	internal pressure
q	dynamic pressure of wind (stagnation pressure)
S_1	topography factor
S_2	ground roughness, building size, and height above ground factor
V	basic wind speed
V_s	design wind
W	width of building
ϵ	height of surface roughness of element

* See note on previous page.

4.2 GENERAL

4.2.1 The wind load on a building shall be calculated for:

(a) The building as a whole; *and*

(b) Individual structural elements such as roofs and walls; *and*

(c) Individual cladding units and their fixings.

C4.2.1 It is important to note that the wind load on a partially completed building will depend on the method and sequence of construction, and may be critical.

4.2.2 Buildings shall be so designed that the risk of wind-induced oscillations of the building as a whole and of individual structural elements and cladding units is negligible or that the consequences are unimportant.

C4.2.2 See clause 4.4.6 and Appendix B.

4.2.3 Buildings shall be designed with due attention to the effects of wind on the comfort of people inside and outside the buildings.

C4.2.3 Clause 4.2.3 requires attention to be paid to such points as:

(a) That unpleasant wind conditions are not created in forecourts, plazas, pedestrian walks, and the like. Wind tunnel tests on models can give guidance on preventing unpleasant wind effects. See references 1, 2, and 3 of Appendix D.

(b) That the maximum acceleration of the building in high winds is not alarming to occupants. See reference 13 of Appendix D.

(c) That wind-generated noise is minimized. Noise can be generated by the wind where vortices are shed by sharp edges and small gaps exist (for example, at window frames) and where flexible members vibrate.

(d) That sealing of doors and windows against draughts and the entry of water is effective.

4.3 WIND SPEEDS

4.3.1 Basic wind speed

4.3.1.1 The return period shall correspond to the nature of the building as given by table 11.

Table 11

RETURN PERIOD

Nature of building	*Return period (years)*
Structures used only during construction operations, such as formwork and falsework	5
Buildings representing a low degree of hazard to life and property in the event of failure, such as isolated towers in wooded areas and farm buildings other than residential buildings or residential accessory buildings	25
All other buildings	50

C4.3.1 The basic wind speed given by clause 4.3.1 is the 3 second gust expected to be equalled or exceeded on the average once in the given return period. An estimate of the basic wind speed with a return period of 100 years can be found by multiplying the 50 year return period wind by 1.06; the accuracy of such an estimate is not high. These basic wind speeds have been estimated by means of a statistical analysis of gust data from continuous wind records at meteorological stations in New Zealand. They refer to the 3 second gust speed at a height of 10 m above the ground in open level conditions. Occasionally in New Zealand winds greatly in excess of the given values occur in tornadoes, of which there are probably at least 30 a year. However, they are small scale phenomena having on the average damage paths 10 to 30 m wide and 1 to 3 km long. Because of their infrequency and small scale they are not taken into account in design.

The 5 year return period in table 11 is based on the assumption that temporary structures will remain in position for 6 months or less.

C4.3.1.2 (a) The average length of record for the cities and towns listed in table 12 is 26 years. Over rugged topography the wind is very gusty and varies considerably from site to site. Around Wellington, for example, the highest recorded winds occurred on 10 April 1968 during the storm in which the vessel T.E.V. Wahine *sank in Wellington Harbour. The maximum recorded gusts for various locations in the city area were:*

	m/s		m/s
Kelburn	*55*	*Hawkins Hill*	*66*
Wellington Airport	*52*	*Oteranga Bay*	*75*
Gracefield	*50*		

It is not possible to give an accurate return period for a storm of this kind, which was the most severe in over 100 years of observations. It is thought to be about 200 years.

C4.3.1.2 (c) There are very few continuous wind records from the high country of New Zealand and therefore fig. 6 has no validity for the central mountain chain.

4.3.1.2 The basic wind speed V shall correspond to the location of the building as follows:

(a) For locations listed in table 12: As given by table 12 for the appropriate return period;

(b) For other locations except mountainous areas: The wind speed given by interpolation from fig. 6 multiplied by:

Return period 50 years, all locations:	1.0
Return period 25 years, all locations:	0.94
Return period 5 years:	
Locations in North Auckland north of Warkworth:	0.76
All other locations:	0.82

(c) For mountainous areas: As determined by a special study in consultation with the Meteorological Service.

Table 12

BASIC WIND SPEEDS FOR SOME CITIES AND TOWNS

City or town	*V* (m/s) *for return period of:*		
	5 years	*25 years*	*50 years*
Auckland	28	31	33
Blenheim	28	31	33
Christchurch	33	38	40
Dunedin	32	36	38
Gisborne	29	33	35
Hokitika	31	35	37
Invercargill	36	40	42
Kaitaia	36	44	48
Napier	31	35	37
Nelson	29	35	37
New Plymouth	34	39	41
Palmerston North	31	35	36
Rotorua	28	32	34
Tauranga	27	30	32
Timaru	31	37	39
Wanganui	36	43	46
Wellington	40	47	50
Westport	30	34	36
Whangarei	32	41	44

4.3.2 Design wind speed

C4.3.2.1 The basic wind speeds (see clause C4.3.1) take no account of the effects of large local variations in the ground surface (topography), ground roughness, building size, and the variation of strong winds with height. These effects are allowed for by the use of the factors S_1 and S_2 to determine design wind speeds.

4.3.2.1 The design wind speed V_s shall be

$$V_s = S_1 S_2 V \quad \text{(38)}$$

where

S_1 is as given by clause 4.3.3 and

S_2 is as given by clause 4.3.4.

4.3.3 Topography factor S_1

C4.3.3 The local effects of mountain and valley configuration are allowed for by S_1. The main mountain chain in New Zealand lies across the prevailing wind flow, and it is common for exposed peaks and ridges above the general level of the surrounding terrain to give rise to accelerated winds; valleys and breaks in the mountain chain in which funnelling of the wind occurs also cause accelerations. Sites so affected are often known locally for their abnormal winds. It is impossible to give more detailed guidance on sites for which S_1 should be taken as 1.1 or higher, so that the choice of S_1 must generally depend on local knowledge and experience of the site concerned. For particularly abnormal sites, and in cases where the designer is doubtful about the value of S_1 to be used, the advice of the Meteorological Service should be sought.

4.3.3.1 The topography factor S_1 shall be as given by table 13 or as determined by a special study in consultation with the Meteorological Service.

Table 13

TOPOGRAPHY FACTOR S_1

Topography	S_1
Valleys and gorges shaped to produce funnelling of the wind; exposed hillsides, peaks, and ridges where acceleration of the wind is known to occur; especially abnormal sites.	1.1 to 1.2 or higher
All other	1.0

4.3.4 Ground roughness, building size, and height above ground factor S_2

4.3.4.1 *General*

4.3.4.1.1 The ground roughness, building size, and height above ground factor shall be as given by table 14 according to the ground roughness category as given by clause 4.3.4.2, the building size class as given by clause 4.3.4.3, and the height above ground as given by clause 4.3.4.4 or Appendix A.

4.3.4.2 *Ground roughness*

C4.3.4.2 The ground roughness categories of clause 4.3.4.2.1 generally correspond to those used in BSCP 3: Chapter V: Part 2 and AS 1170 Part 2*, but "ground roughness 4" has not been included because it is not considered applicable to New Zealand conditions. The applicable ground roughness category may vary with the direction of wind being considered. In cases of doubt the Meteorological Service should be consulted.*

The height of the few obstructions present in ground roughness 1 will generally be less than 1.5 m. Airfields are included in this category because many of the wind records from which basic wind speeds were determined come from such sites; nevertheless, local conditions may well be such that ground roughness 2 should be used for the design of buildings at or adjacent to airfields.

4.3.4.2.1 The ground roughness category shall be whichever of the following most closely corresponds to the area in which the building site is located:

Ground roughness 1: Open fetches of level or nearly level country with no shelter. Examples are flat coastal fringes, airfields, and swamps.

Ground roughness 2: Flat or undulating country with obstructions such as hedges or walls around fields, scattered windbreaks, and occasional buildings. Examples are wasteland and most agricultural land that is not well wooded.

*See list of related documents.

The height of the well-scattered obstructions present in ground roughness 2 will generally be between 1.5 m and 10 m.

It is expected that ground roughness 3 will apply to most buildings.

Ground roughness 3: Surfaces covered with numerous large obstructions. Examples are well wooded farmland and forest areas, towns, and cities.

4.3.4.3 *Building size*

4.3.4.3.1 The building size class shall be whichever of the following applies to the building (or part of a building) under consideration:

Class A: All units of cladding, glazing, and roofing and their immediate fixings; individual members of unclad structures.

Class B: All buildings for which neither the greatest horizontal nor the greatest vertical dimension exceeds 50 m.

Class C: All buildings for which either the greatest horizontal or the greatest vertical dimension (or both) exceeds 50 m.

Table 14

GROUND ROUGHNESS, BUILDING SIZE, AND HEIGHT ABOVE GROUND FACTOR S_2

Height above ground (m)	*Ground roughness*								
	1			*2*			*3*		
	Building class			*Building class*			*Building class*		
	A	*B*	*C*	*A*	*B*	*C*	*A*	*B*	*C*
[illegible] or less	0.83	0.78	0.73	0.72	0.67	0.63	0.64	0.60	0.55
[illegible]	0.88	0.83	0.78	0.79	0.74	0.70	0.70	0.65	0.60
10	1.00	0.95	0.90	0.93	0.88	0.83	0.78	0.74	0.69
15	1.03	0.99	0.94	1.00	0.95	0.91	0.88	0.83	0.78
20	1.06	1.01	0.96	1.03	0.98	0.94	0.95	0.90	0.85
30	1.09	1.05	1.00	1.07	1.03	0.98	1.01	0.97	0.92
40	1.12	1.08	1.03	1.10	1.06	1.01	1.05	1.01	0.96
50	1.14	1.10	1.06	1.12	1.08	1.04	1.08	1.04	1.00
60	1.15	1.12	1.08	1.14	1.10	1.06	1.10	1.06	1.02
80	1.18	1.15	1.11	1.17	1.13	1.09	1.13	1.10	1.06
100	1.20	1.17	1.13	1.19	1.16	1.12	1.16	1.12	1.09
120	1.22	1.19	1.15	1.21	1.18	1.14	1.18	1.15	1.11
140	1.24	1.20	1.17	1.22	1.19	1.16	1.20	1.17	1.13
160	1.25	1.22	1.19	1.24	1.21	1.18	1.21	1.18	1.15
180	1.26	1.23	1.20	1.25	1.22	1.19	1.23	1.20	1.17
200	1.27	1.24	1.21	1.26	1.24	1.21	1.24	1.21	1.18

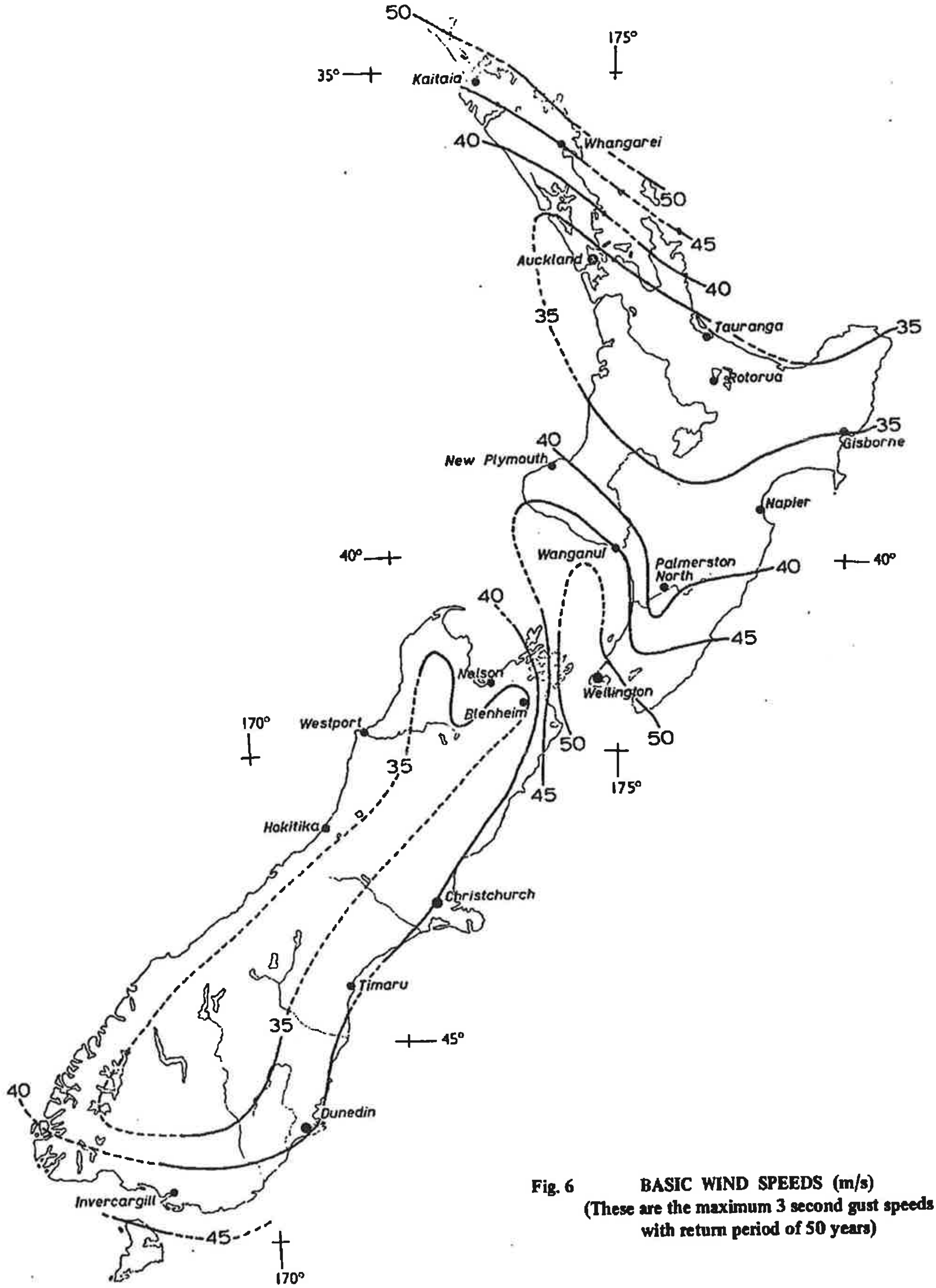


Fig. 6 **BASIC WIND SPEEDS (m/s)**
(These are the maximum 3 second gust speeds with return period of 50 years)

4.3.4.4 *Height above ground*

4.3.4.4.1 The height above ground shall be measured from the general level of the ground to windward and shall be either:

(a) The height h as shown in the applicable diagram in this standard; *or*

(b) Where there is no such diagram:

The height to the eaves for a building with a pitched roof;

The height to the top of the building for a building with a flat roof;

provided that for buildings on or near the edge of an escarpment or a relatively sudden change in ground level the method of Appendix A shall be used.

C4.3.4.4 It should be noted that although the height above ground is measured to the top of a building or part of a building for the determination of S_2, the height to the eaves is relevant to the determination of pressure and force coefficients from some of the tables.

4.3.4.4.2 If the wind load on the whole of the building is to be determined by dividing the building into convenient parts and finding the wind load on each part separately, then:

(a) The height of any such part shall not be less than three times its breadth; and

(b) The factor S_2 used for each part shall correspond to the height above ground of the top of that part.

4.4 PRESSURES AND FORCES

4.4.1 The dynamic pressure q of the wind shall be determined from

$$q = 0.613\ V_s^2 \qquad (39)$$

where V_s is given by section 4.3. Table 15 gives values of q for values of V_s from 10 to 70 m/s in 1 m/s intervals.

C4.4.1 The coefficient 0.613 applies only when V_s is expressed in metres per second (m/s) and q in pascals (Pa). The value of this coefficient has been chosen as suitable for the ambient temperature and relative humidity that can be assumed for design purposes throughout most of New Zealand; a different value may sometimes be necessary to suit the conditions at a particular site.

4.4.2 The pressure p at any point on the surface of the building shall be determined from

$$p = C_p q \qquad (40)$$

where C_p is as given by section 4.5.

C4.4.2 If the value of the pressure coefficient C_p is negative this indicates that p is a suction as distinct from a positive pressure.

4.4.3 The resultant pressure force F acting on an individual structural element or cladding unit normal to its surface shall be determined from

$$F = C_p q A \qquad (41A)$$

or

$$F = (C_{pe} - C_{pi})\ qA \qquad (41B)$$

where C_p, C_{pe}, and C_{pi} are as given by section 4.5 and A is the surface area of the structural element or cladding unit.

C4.4.3 Equation 41B applies only to structural elements or cladding units for which the surface exposed to external pressure has the same area A as, and is parallel to, the surface exposed to internal pressure. A negative value for F indicates that the force acts outwards.

4.4.4 The resultant frictional force F' acting along any flat surface in the direction of the wind shall be

$$F' = C_f' qA \quad (42)$$

where C_f' is as given by section 4.11.

C4.4.4 The frictional force on flat surfaces may form an appreciable part of the total wind load when the area of such flat surfaces along the direction of the wind is large compared to the frontal area of the structure.

4.4.5 The resultant force on the building as a whole shall be determined by the vectorial summation of the forces on all the surfaces, provided that the resultant pressure force F on the building as a whole may be determined from

$$F = C_f qA_e \quad (43)$$

where C_f is as given by section 4.8 and A_e is the effective frontal area of the building.

C4.4.5 Equation 43 gives the pressure force acting in the direction of the wind. The value of C_f differs for the wind acting on different faces of the building, so that in order to determine the critical value it is necessary to determine F for each wind direction.

If applicable values of C_f are not given in this standard, or if by virtue of the building shape or wind direction the resultant pressure force is inclined to the wind, then the resultant pressure force on the building as a whole will have to be determined by vectorial summation. If the loading is to be determined by this method then coefficients from other sources (such as references 9 and 15 of Appendix D) should be used. The average pressure coefficients for pitched roofs and for walls of rectangular buildings, as given by clauses 4.7.2 and 4.9.1.1, are to be used to obtain the overall force acting on a building or an individual frame within a building.

In the shear flow, which is implied by the increasing values of S_2 with height in table 14, positive pressure on the windward face follows the local variation of q with height. Suctions on leeward faces tend to be constant with height and given by q at the top of the building. This effect can be allowed for by using an effective dynamic pressure

$$q_e \doteq 0.9\, q_h$$

where q_h is the dynamic pressure at the top of the building of height h and q_e gives the correct force on the building.

In order to calculate moments about ground level the force F can be taken to act at a centre of pressure located at a point 0.55 h above ground level.

4.4.6 If the natural period of oscillation of the building as a whole or of an individual structural element (see clause 3.4.4) is in the range 2 to 6 s, dynamic overshoot shall be allowed for by multiplying the design wind load for the building or element by 1.7.

C4.4.6 Pressure and force coefficients describe the effect of the design gust as if it were applying a steady load to the building. However, the basis for the design wind speed is a gust of 3 seconds duration, and such a transient load causes a dynamic overshoot so that the maximum deflection and the accompanying stresses exceed those caused by a steady load of the same magnitude. When the gust duration and the natural period of oscillation of the building are approximately equal, this magnification effect is substantial. Dynamic overshoot is likely to affect only the design of slender structures such as high-rise buildings, chimneys, and towers. See also clause 4.2.2, Appendix B, and references 10, 13 and 17 of Appendix D. For structures having a natural period of oscillation exceeding 6 seconds, a more detailed analysis of gust reponse is indicated.

4.5 PRESSURE COEFFICIENTS: GENERAL

4.5.1 Pressure coefficients shall be obtained from:

(a) Wind tunnel or similar tests conducted by an approved authority; *or*

C4.5 The nature of the flow around shapes such as are used in buildings is extremely complicated and not amenable to full theoretical prediction. Small changes in shape *can give rise to significant changes in the flow. The flow is further affected by factors such as the intensity of wind turbulence and the size of the building in relation to the average size of the turbulent eddies, transient effects during the passage of wind gusts, the gradient of wind speed with height, and the exact nature of the immediate surrounding buildings. Accurate values of pressures and forces for particular cases can best be predicted from wind tunnel studies on scale models. In many cases resort to such testing is not practicable and an alternative basis for design is adequate. For this purpose typical force and pressure coefficients can be tabulated for guidance. It should be accepted that at best such tabulated figures are not more reliable than ±15 percent, and where a greater accuracy is called for wind tunnel tests or reference to other sources of information is indicated.*

Separate values of pressure coefficients for windward and leeward areas have not been presented, since in the final design analysis a surface could be subjected to both of the conditions, dependent on the wind direction. Values presented are such that the worst conditions to which the surface would be subjected are covered.

Pressure coefficients are given for a particular surface or part of the surface of a building. The area of that surface or part of the surface when multiplied by the pressure coefficient and the dynamic pressure q gives the wind loading in a direction normal to that particular surface or part thereof. When calculating wind load on individual structural elements such as roofs and walls, and individual cladding units and their fixings, it is essential to take account of the pressure difference between opposite faces of such elements or units. For clad structures it is therefore necessary to know the internal pressure as well as the external pressure. The following distinguishing pressure coefficients are therefore used:

C_{pe} = *external pressure coefficient.*

C_{pi} = *internal pressure coefficient.*

Areas of high local suction frequently occur near the edges of walls and roofs. These coefficients are shown separately and should be used only to calculate the loads on these local areas (including all elements, such as purlins, girts, and their fixings, as well as cladding units and their fixings, lying wholly within the areas to which the local coefficients apply). They should not be used for calculating additive loads on entire structural elements such as roofs and walls.

See also references 5, 6, 7, 8, 9, 14 and 15 of Appendix B.

C4.6 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 is a ridge ventilator on a low-pitch roof, and this, under all directions of wind, can reduce the uplift force on the roof.

(b) Sections 4.6 and 4.7 of this standard; *or*

(c) Other approved sources.

4.5.2 Each individual structural element or cladding unit shall be designed for the worst possible combination of pressure coefficients. The dynamic pressure *q* shall be taken at the height specified for each case.

4.6 INTERNAL PRESSURE COEFFICIENTS

4.6.1 The internal pressure coefficients for a clad building of rectangular plan shall be:

(a) Openings uniformly distributed in all walls:

C_{pi} = ±0.3

(b) Any single wall only having openings, with other

Damage to windows, cladding or doors on windward walls or in areas of high local suction could give rise to the conditions specified in clauses 4.6.1 (b) and 4.6.1 (d). In many cases however, the use of $C_{pi} = \pm 0.3$ *is considered to be an acceptable risk. For calculation of deflections, values of* C_{pi} *larger than* ± 0.3 *need not be used.*

Values of C_{pi} *for open-ended cylinders are given in clause 4.7.7.2. The variation of internal pressure throughout the building may be significant and may cause pressure loads on internal partitions.*

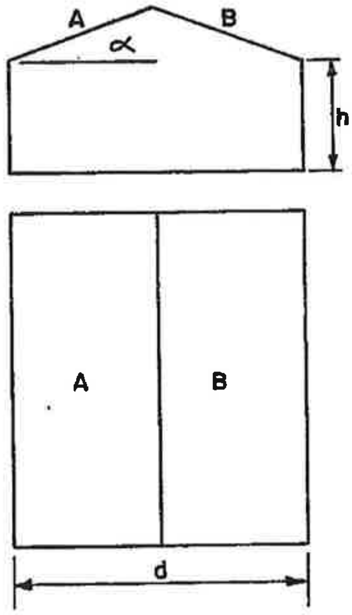


Fig. 7 PITCHED ROOFS OF CLAD BUILDINGS: NOTATION

C4.7.2 The pressure coefficients given by clauses 4.7.2.2 and 4.7.2.3 shall be vectorially summed to obtain the overall force acting on the roof structure and the building as a whole.

walls and roof impermeable or having openings in the aggregate less than one-sixth of the area of those in the dominant wall:

$C_{pi} = \pm 0.8$

(c) Dominant openings in the roof:

$C_{pi} = C_{pe}$ for roofs as given by clause 4.7.2.

(d) A dominant opening in a region of high local suction:

$C_{pi} = C_{pe}$ for that region as given by clause 4.7.3.

4.6.2 For buildings with mechanically induced internal pressurization, this additional pressure shall also be allowed for in design.

4.7 EXTERNAL PRESSURE COEFFICIENTS

4.7.1 Walls

4.7.1.1 Each wall of a rectangular building shall be designed for both

$C_{pe} = 0.8$ and -0.7.

4.7.1.2 The dynamic pressure q shall be taken at the height of the eaves.

4.7.2 Roofs of clad buildings

4.7.2.1 Each section A and B as shown in fig. 7 and fig. 9 and the roof as a whole and the supporting structure shall be designed for the worst possible combination of pressure coefficients. The dynamic pressure q shall be taken at the height h as shown in fig. 7 and fig. 9.

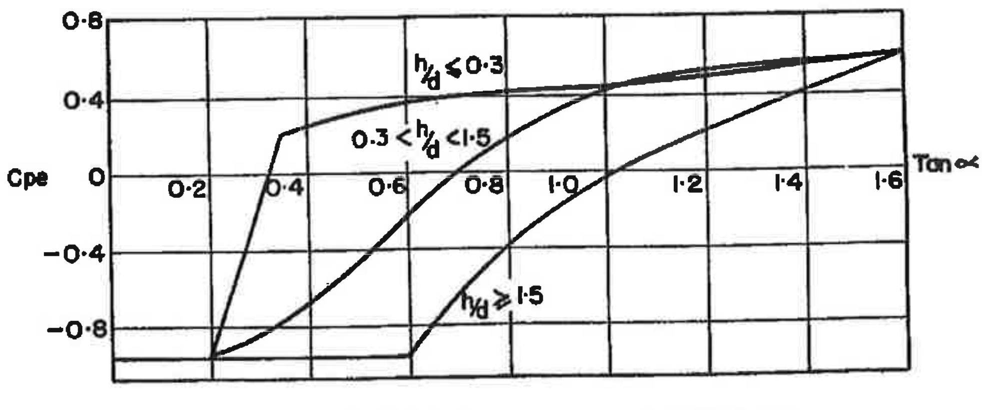


Fig. 8 EXTERNAL PRESSURE COEFFICIENTS FOR PITCHED ROOFS OF CLAD BUILDINGS (see clause 4.7.2.2)

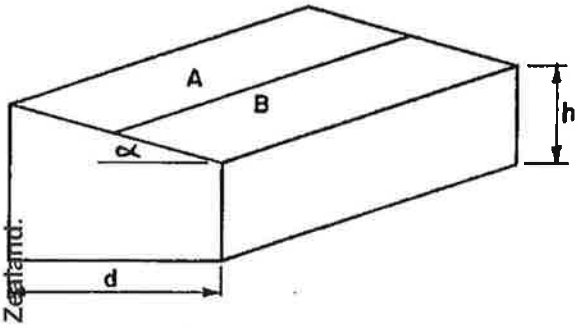


Fig. 9 MONOSLOPE ROOFS OF CLAD BUILDINGS: NOTATION

C4.7.3 The local pressure coefficients as given by clause 4.7.3 are not to be used in determining the overall forces acting on a structure.

C4.7.3.2 (c) The net maximum value of C_p *for roof overhangs is:*

Roofs of slope $\geqslant 10°$*:*

2.3 for sections F
3.3 for sections G

Roofs of slope $< 10°$*:*

2.8 for sections F
3.8 for sections G

In each case the resultant force acts upward normal to the roof.

4.7.2.2 For pitched roofs of clad buildings the external pressure coefficients shall be:

(a) Roof angle α less than or equal to 20°:

C_{pe} = −0.9 on both sections A and B.

(b) Roof angle α greater than 20°:

C_{pe} = −0.8 on both sections A and B.

(c) All roof angles

C_{pe} = −0.7 on either section and the value given by fig. 8 on the other section.

4.7.2.3 For monoslope roofs the external pressure coefficients shall be:

(a) Roof angle α less than or equal to 15°:

C_{pe} = −1.0 on both sections A and B.

or

C_{pe} = −1.0 on either section and −0.4 on the other section.

(b) Roof angle α greater than 15° but less than 30°:

C_{pe} = −0.9 on both sections A and B for a distance $d/2$ from each end and −0.5 on the remainder.

or

C_{pe} = −1.0 on section B and −0.2 on section A.

or

C_{pe} = −1.0 on section A and −0.6 on section B.

(c) Roof angle α equal to or greater than 30°:

C_{pe} = −0.8 on both sections A and B for a distance of $d/2$ from each end and −0.5 on the remainder.

or

C_{pe} = −0.6 on section B and 0.0 on section A.

or

C_{pe} = −1.0 on section A and −0.6 on section B.

4.7.3 Edges of roofs and corners of buildings

4.7.3.1 Each section F and G as shown in fig. 10 (which applies both to pitched roofs, as shown, and to monoslope roofs) and roof over hangs shall be designed for the worst possible combination of pressure coefficients. The dynamic pressure q shall be taken at the height h as shown in fig. 10.

4.7.3.2 The external pressure coefficients shall be:

(a) For each section F:

C_{pe} = −1.5 for roofs of slope $\geqslant 10°$

C_{pe} = −2.0 for roofs of slope $< 10°$

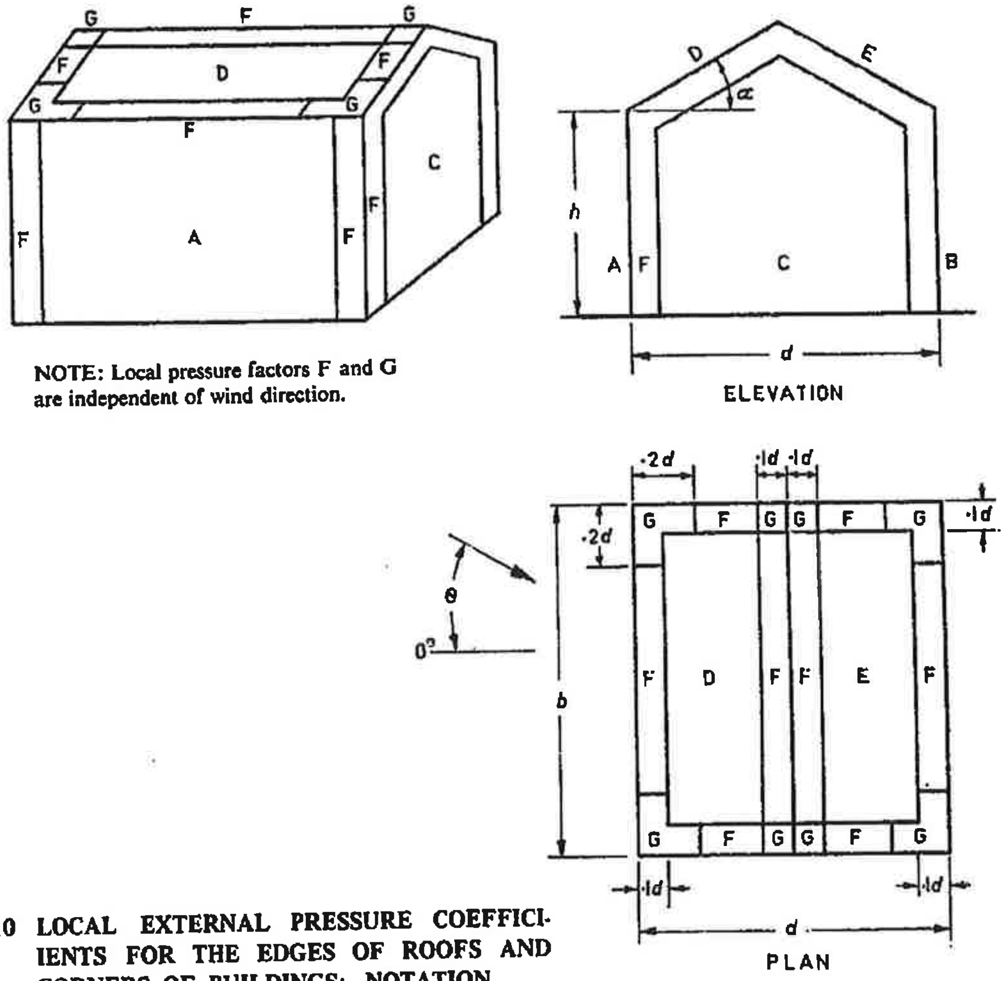
(b) For each section G:

C_{pe} = −2.5 for roofs of slope $\geqslant 10°$

C_{pe} = −3.0 for roofs of slope $< 10°$

(c) For the underside of roof overhangs:

C_{pe} = +0.8.



NOTE: Local pressure factors F and G are independent of wind direction.

d = plan dimension of roof surface.

Fig. 10 LOCAL EXTERNAL PRESSURE COEFFICIENTS FOR THE EDGES OF ROOFS AND CORNERS OF BUILDINGS: NOTATION. (APPLIES TO BOTH PITCHED ROOFS, AS SHOWN, AND MONOSLOPE ROOFS.)

4.7.4 Free roofs

4.7.4.1 Monoslope free roofs for which the minimum clear height under the roof is greater than one-half of the depth in the direction of the wind shall be assumed to be acted upon by a combination of resultant pressures normal to the roof in the direction given by fig. 11 such that

$$p = C_p q$$

where C_p varies linearly across the width from $C_p = 2.0$ at the windward edge (C_{tw}) to $C_p = \tan \alpha$ at the leeward edge (C_{tl}), and no variation shall be made for local effects.

C4.7.4.1 The subscripts "tw" and "tl" are used to denote total (that is, combined internal and external) pressure coefficient at the windward and leeward edges respectively. The angle of the roof, α, is the angle in degrees from the horizontal and is always considered to be positive.

4.7.4.2 For pitched and troughed free roofs the resultant pressures normal to the roof shall be taken as $p = C_p q$ where C_p is given by fig. 12 to fig. 17 inclusive. No increases shall be made for local effects except as indicated.

C4.7.4.2 Further guidance on pitched and troughed free roofs is given in table 13 of BSCP 3: Chapter V: Part 2 †It should be noted that the effective roof angle can be, say ±10° different from the nominal angle as a result of sloping terrain; this can be particularly significant when such roofs are near the edge of a sharp height change, such as a quayside.

d = *plan dimension of roof surface.*

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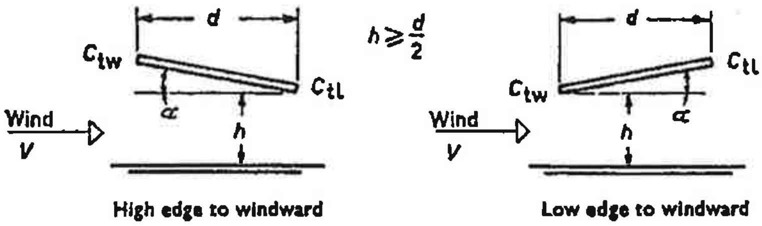
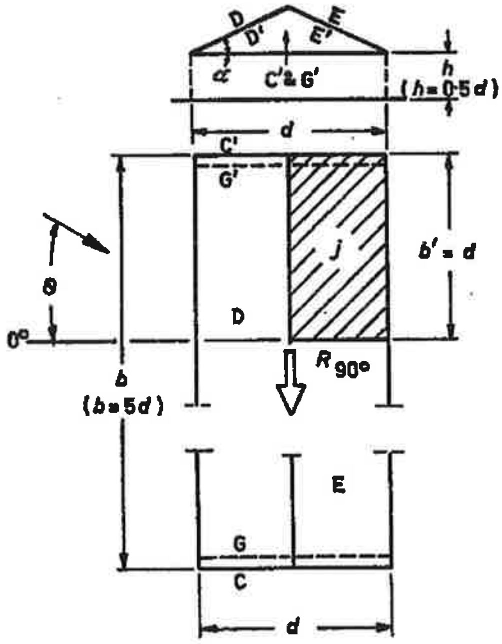
† See list of related documents.



Exposure	*Roof pitch (tan α)*	*Direction resultant pressure*
High edge to windward	$\tan\alpha \leqslant 0.2$	*both {up, down}
	$0.2 < \tan\alpha \leqslant 2.0$	up
Low edge to windward	$\tan\alpha \leqslant 0.2$	*both {down, up}
	$0.2 < \tan\alpha \leqslant 2.0$	down

* Both alternatives to be considered

Fig. 11 MONOSLOPE FREE ROOFS

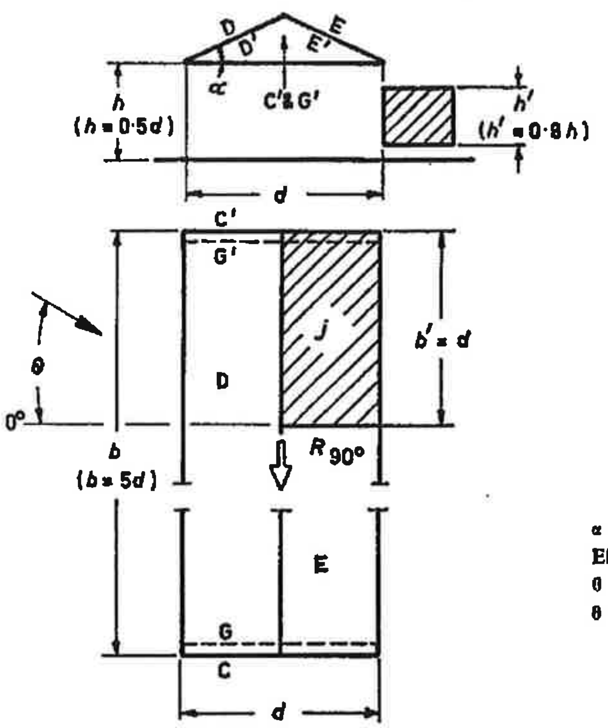


α = 30°
θ = 0° – 45° D – E' full length
θ = 90° D – E' part length b'

θ	External pressure coefficients C_{pe}							
					End surfaces			
	D	D'	E	E'	C	C'	G	G'
0°	0.6	-1.0	-0.5	-0.9				
45°	0.1	-0.3	-0.6	-0.3				
90°	-0.3	-0.4	-0.3	-0.4	-0.3	0.8	0.3	-0.4

45° For j: C_p top = 1.0; C_p bottom = -0.2
90° Tangentially acting friction: $R_{90°} = 0.05\ qbd$

Fig. 12 PITCHED FREE ROOFS, α = 30°

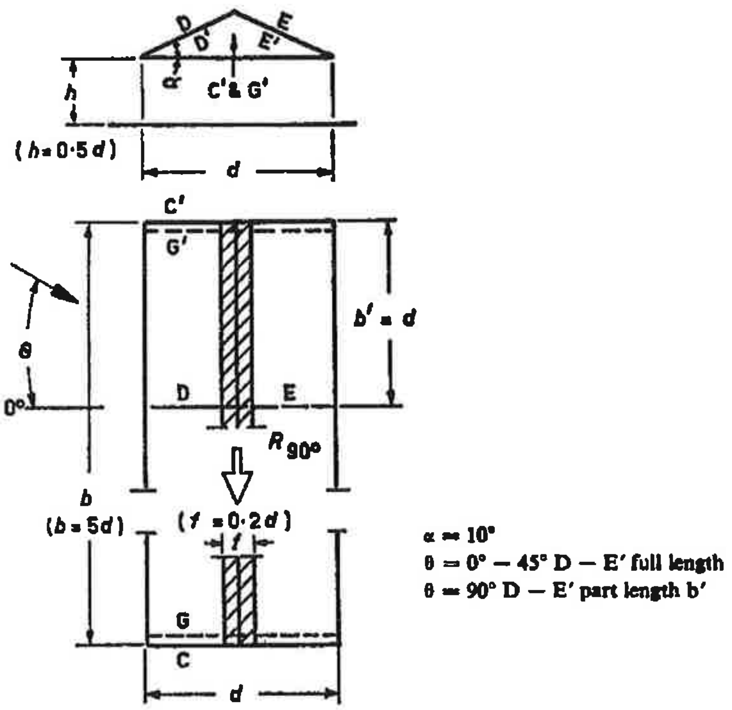


α = 30°
Effects of trains or stored materials:
θ = 0° – 45° – 180° D – E' full length
θ = 90° D – E' part length b'

θ	External pressure coefficients C_{pe}							
					End surfaces			
	D	D'	E	E'	C	C'	G	G'
0°	0.1	0.8	-0.7	0.9				
45°	-0.1	0.5	-0.8	0.5				
90°	-0.4	-0.5	-0.4	-0.5	-0.3	0.8	0.3	-0.4
180°	-0.3	-0.6	0.4	-0.6				

45° For j: C_p top = -1.5; C_p bottom = 0.5
90° Tangentially acting friction, see fig. 12

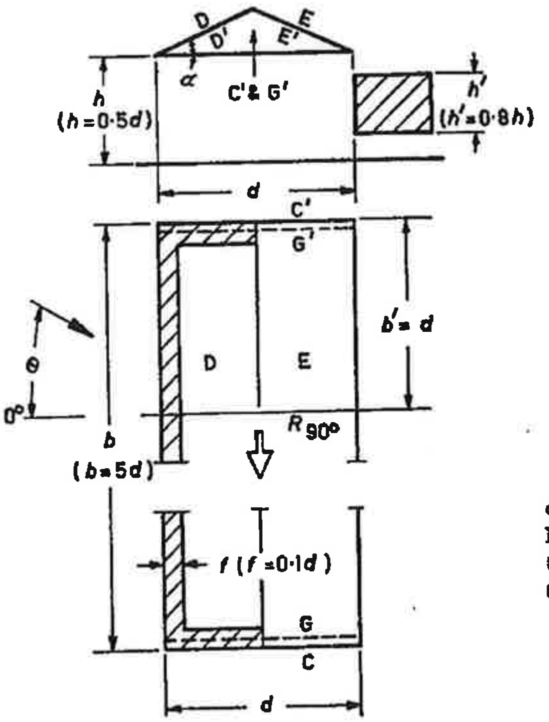
Fig. 13 PITCHED FREE ROOFS, α = 30°, WITH EFFECTS OF TRAINS OR STORED MATERIALS



θ	External pressure coefficients C_{pe}							
					End surfaces			
	D	D'	E	E'	C	C'	G	G'
0°	-1.0	0.3	-0.5	0.2				
45°	-0.3	0.1	-0.3	0.1				
90°	-0.3	0	-0.3	0	-0.4	0.8	0.3	-0.6

0° For *f*: $C_p = -1.0$; C_p bottom = 0.4
0° to 90° Tangentially acting friction, $R_{90°} = 0.1\ qbd$

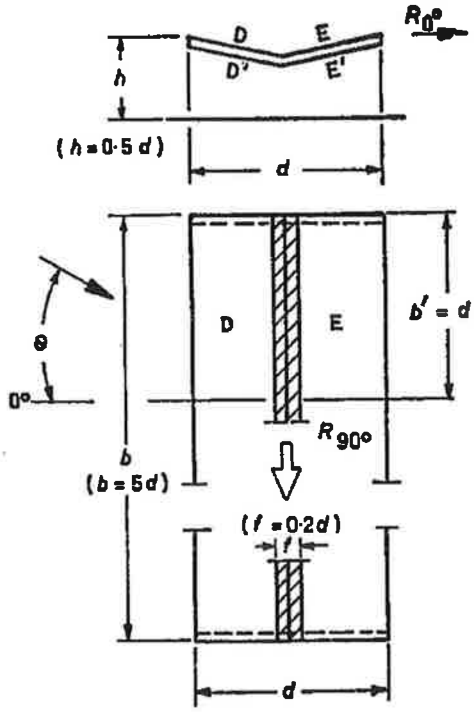
Fig. 14 PITCHED FREE ROOFS, $\alpha = 10°$



$\alpha = 10°$
Effects of trains or stored materials:
$\theta = 0° - 45° - 180°$ D — E' full length
$\theta = 90°$ D — E' part length b'

θ	External pressure coefficients C_{pe}							
					End surfaces			
	D	D'	E	E'	C	C'	G	G'
0°	-1.3	0.8	-0.6	0.7				
45°	-0.5	0.4	-0.3	0.3				
90°	-0.3	0	-0.3	0	-0.4	0.8	0.3	-0.6
180°	-0.4	-0.3	-0.6	-0.3				
0°	For f: C_p top = 1.6; C_p bottom = 0.9							
0° to 180°	Tangentially acting friction, see fig. 14							

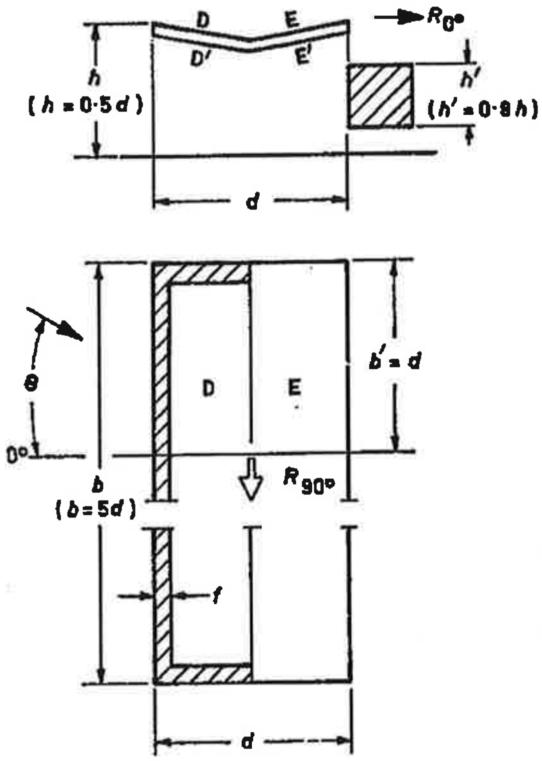
Fig. 15 PITCHED FREE ROOFS, $\alpha = 10°$, WITH EFFECTS OF TRAINS OR STORED MATERIALS



Roof slope α — 10°
θ = 0° — 45° D — E' full length
θ = 90° D — E' part length b'

θ	*External pressure coefficients C_{pe}*			
	D	D'	E	E'
0°	0.3	-0.7	0.2	-0.9
45°	0	-0.2	0.1	-0.3
90°	-0.1	0.1	-0.1	0.1
0°	For f: C_p top = 0.4; C_p bottom = -1.5			
0° to 90°	Tangentially acting friction: $R_{90°} = 0.1\,qbd$			

Fig. 16 TROUGHED FREE ROOFS

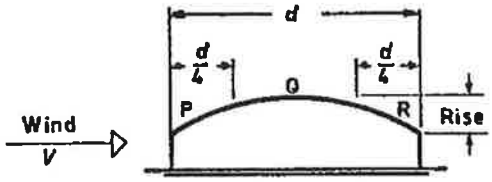


Roof slope α = 10°
Effects of trains or stored materials:
θ = 0° — 45° — 180° D — E′ full length
θ = 90° D — E′ part length b′

θ	*External pressure coefficients C_{pe}*			
	D	D′	E	E′
0°	-0.7	0.8	-0.6	0.6
45°	-0.4	0.3	-0.2	0.2
90°	-0.1	0.1	-0.1	0.1
180°	-0.4	-0.2	-0.6	-0.3

0° For *f*: C_p top = -1.1; C_p bottom = 0.9
0° to 180° Tangentially acting friction: $R_{90°} = 0.1\,qbd$

Fig. 17 TROUGHED FREE ROOFS WITH EFFECTS OF TRAINS OR STORED MATERIALS



Exposure	Rise-to-span ratio, G $G = \frac{rise}{d}$	External pressure coefficients C_{pe} Windward quarter P	Centre half Q	Leeward quarter R
Roof on elevated structure	$0 < G < 0.2$ $0.2 \leqslant G < 0.3$ $0.3 \leqslant G \leqslant 0.6$	-0.9 $1.5G - 0.3$* $2.75G - 0.675$	$-0.7 - G$ $-0.7 - G$ $-0.7 - G$	-0.5 -0.5 -0.5
Roof springing from ground level	$0 < G < 0.6$	$1.4G$	$-0.7 - G$	-0.5

*The alternative coefficient $6G - 2.0$ shall also be used.

Fig. 18 CURVED ROOFS

4.7.5 Curved roofs

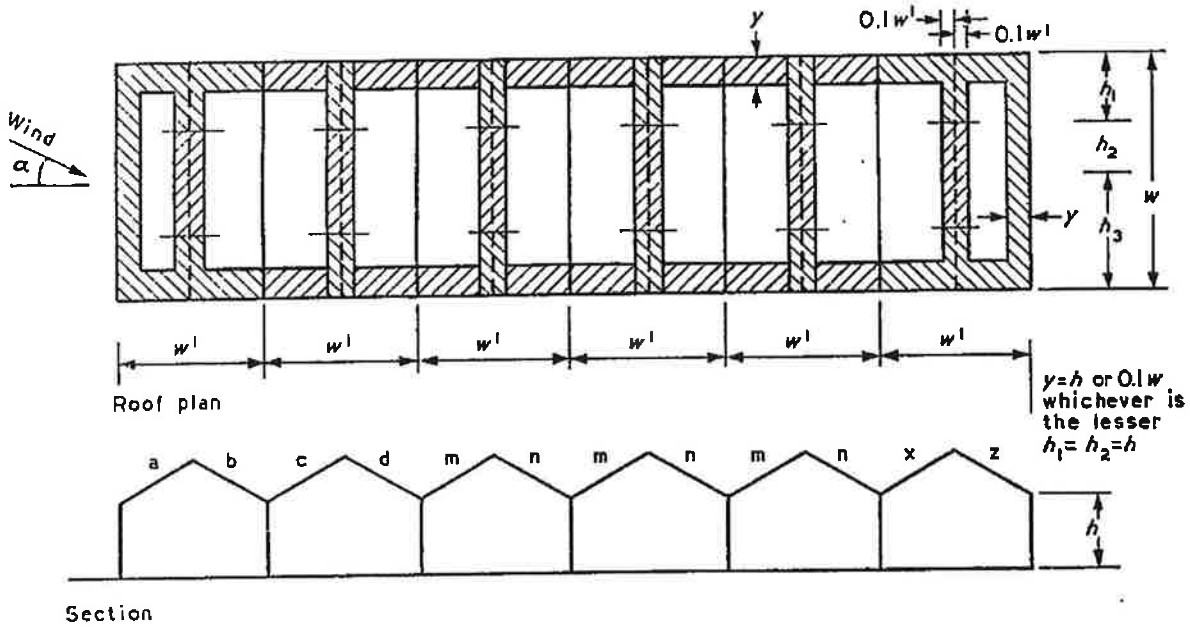
4.7.5.1 For curved roofs the external pressure coefficients shall be as given by fig. 18. Allowance for local effects shall be made in accordance with clause 4.7.3.

4.7.6 Pitched and saw-tooth roofs of multi-span buildings

C4.7.6 Evidence on multi-span buildings is fragmentary, and any departures from the cases given in fig. 19 and fig. 20 should be investigated separately.

4.7.6.1 For pitched and saw-tooth roofs of multi-span buildings the external pressure coefficients shall be as given by fig. 19 and fig. 20 respectively provided that all spans shall be equal and the height to the eaves shall not exceed the span ($h \not> w'$ on fig. 19 and fig. 20).

4.7.6.2 For wind along the axis of the building frictional force is allowed for in fig. 19 and fig. 20, but for wind perpendicular to the axis of the building frictional force shall be allowed for in accordance with clause 4.4.4.

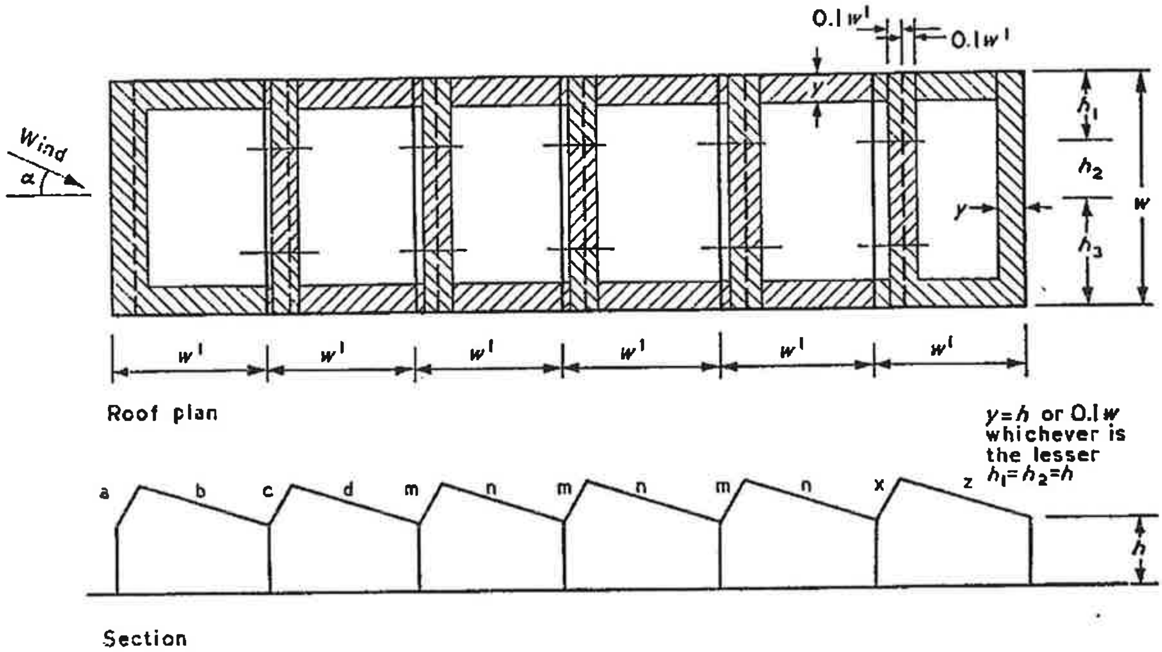


Roof angle	Wind angle α	First span a	First span b	First intermediate span c	First intermediate span d	Other intermediate spans m	Other intermediate spans n	End span x	End span z	Local coefficient (▧)	Local coefficient (▨)
degrees 5	degrees 0	-0.9	-0.6	-0.4	-0.3	-0.3	-0.3	-0.3	-0.3		
10		-1.1	-0.6	-0.4	-0.3	-0.3	-0.3	-0.3	-0.4	-2.0	-1.5
20		-0.7	-0.6	-0.4	-0.3	-0.3	-0.3	-0.3	-0.5		
30		-0.2	-0.6	-0.4	-0.3	-0.2	-0.3	-0.2	-0.5		
45		+0.3	-0.6	-0.6	-0.4	-0.2	-0.4	-0.2	-0.5		

Roof angle	wind angle α	Distance h_1	h_2	h_3
degrees Up to 45	degrees 90	-0.8	-0.6	-0.2

Frictional force: when wind angle $\alpha = 0^\circ$ horiztonal forces due to frictional force are allowed for in the above values;
when wind angle $\alpha = 90^\circ$ allow for frictional force in accordance with section 4.11.

Fig. 19 EXTERNAL PRESSURE COEFFICIENTS C_{pe} FOR PITCH ROOFS OF MULTIPLE-SPAN BUILDINGS (ALL SPANS EQUAL) WITH h NOT GREATER THAN w'.



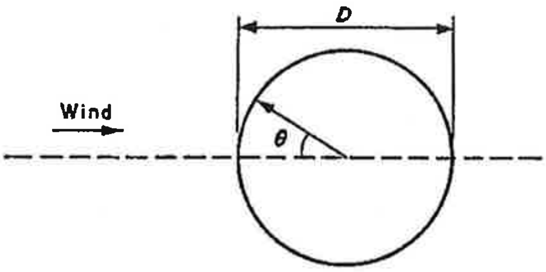
Wind angle α	First span		First intermediate span		Other intermediate spans		End spans		Local coefficient	
	a	b	c	d	m	n	x	z		
degrees										
0	+0.6	-0.7	-0.7	-0.4	-0.3	-0.2	-0.1	-0.3	-2.0	-1.5
180	-0.5	-0.3	-0.3	-0.3	-0.4	-0.6	-0.6	-0.1		

Wind angle α	Distance h_1	h_2	h_3
degrees			
90	-0.8	-0.6	-0.2
270	Similarly, but handed		

Frictional force: when wind angle $\alpha = 0°$ horizontal forces due to frictional force are allowed for in the above values; when wind angle $\alpha = 90°$ allow for frictional force in accordance with section 4.11.

Fig. 20 EXTERNAL PRESSURE COEFFICIENTS C_{pe} FOR SAW-TOOTH ROOFS OF MULTIPLE-SPAN BUILDINGS (ALL SPANS EQUAL) WITH h NOT GREATER THAN w'.

C4.7.7 Fig. 21 will apply only when the diameter D *exceeds 0.3 m. It can be used for wind blowing normal to the axis of vertical cylinders such as chimneys and silos, and of horizontal cylinders such as horizontal tanks sufficiently clear of the ground. The limitation* D > *0.3 m is because fig. 21 applies only to cylinders in supercritical flow.*



4.7.7 Cylindrical structures

4.7.7.1 For the purpose of calculating wind forces that act in such a way as to deform a cylindrical structure the values of C_{pe} given in fig. 21 shall be used provided that:

(a) The diameter D of the cylinder shall be greater than 0.3 m.

(b) The length h shall be the height of a vertical cylinder or the length of a horizontal cylinder except that where there is a free flow of air about both ends h shall be taken as half the length when calculating h/D.

(c) For cylinders having their axis parallel to the ground plane the clearance between the cylinder and the ground shall be not less than D.

4.7.7.2 In the calculation of the load on the periphery of the cylinder, the value of C_{pi} shall be taken into account. For open-ended cylinders C_{pi} shall be taken as:

–0.8 where h/D is not less than 0.3

–0.5 where h/D is less than 0.3.

Position on periphery θ	*Pressure coefficient C_{pe}*			
	Surface: rough or with projections.		*Surface: smooth*	
degrees	h/D = 10	$h/D \not> 2.5$	h/D = 10	$h/D \not> 2.5$
0	+1.0	+1.0	+1.0	+1.0
10	+0.9	+0.9	+0.9	+0.9
20	+0.7	+0.7	+0.7	+0.7
30	+0.4	+0.4	+0.35	+0.35
40	0	0	0	0
50	-0.5	-0.4	-0.7	-0.5
60	-0.95	-0.8	-1.2	-1.05
70	-1.25	-1.1	-1.4	-1.25
80	-1.2	-1.05	-1.45	-1.3
90	-1.0	-0.85	-1.4	-1.2
100	-0.8	-0.65	-1.1	-0.85
120	-0.5	-0.35	-0.6	-0.4
140	-0.4	-0.3	-0.35	-0.25
160	-0.4	-0.3	-0.35	-0.25
180	-0.4	-0.3	-0.35	-0.25

Fig. 21 PRESSURE DISTRIBUTION AROUND CYLINDRICAL STRUCTURES

4.8 FORCE COEFFICIENTS: GENERAL

4.8.1 Force coefficients shall be obtained from:

(a) Wind tunnel or similar tests conducted by an approved authority; *or*

(b) Sections 4.9 and 4.10 and Appendix C of this standard; *or*

(c) Other approved sources.

Fig. 22 AVERAGE EXTERNAL WALL-PRESSURE COEFFICIENTS FOR BUILDINGS HAVING h/b NOT GREATER THAN 5, AND OF RECTANGULAR PLAN

$\theta = 0$

	Mark in fig 10	d/b	*Average pressure coefficient* C_{av} (1)
Windward external wall surface	A	All values	+0.8
Leeward external wall surfaces	B	0 to 1 (2) 4 or more	-0.5 -0.2
Side external wall surface	C	All values	-0.7
(2) For d/b values intermediate between the limits given, interpolate linearly.		(1) All values of C_{av} shall be used with the value of q applying at height = h.	

NOTE – The pressure coefficients given in fig. 22 are average values for use in establishing overall wind loadings. The values quoted are applicable for sharp-edged rectangular buildings when the wind is blowing normal to one face. The values also 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 quoted in this table. Under these conditions the appropriate coefficients can be determined only from correctly scaled wind-tunnel tests. As detailed in clause 4.7.3, local peaks of negative pressure may be much higher than these average values.

Table 16

FORCE COEFFICIENTS FOR LOW WALLS AND HOARDINGS

Width to height ratio

Above ground	*On ground*	C_f
From 0.5 to 6	From 1 to 12	1.2
10	20	1.3
16	32	1.4
20	40	1.5
40	80	1.75
60	120	1.8
80 or more	160 or more	2.0

*4.9 FORCE COEFFICIENTS FOR CLAD BUILDINGS

4.9.1 Buildings of uniform section

C4.9.1 Shapes for individual structural members are given in fig. 25, and these may be used for building shapes in suitable cases (note that clause 4.10.2.1 (a) will apply in such cases).

Table 17 REDUCTION FACTOR K

l/d	2	5	10	20	40	50	100	∞
Circular cylinder, subcritical flow	0.58	0.62	0.68	0.74	0.82	0.87	0.98	1.0
Circular cylinder, supercritical flow	0.80	0.80	0.82	0.90	0.98	0.99	1.0	1.0
Flat plate perpendicular to wind	0.62	0.66	0.69	0.81	0.87	0.90	0.95	1.0

4.9.1.1 Force coefficients for rectangular clad buildings shall be obtained by summing the average pressure coefficient as given by fig. 22. For other clad buildings of uniform section the force coefficients are given by fig. 23.

4.9.2 Circular shapes

4.9.2.1 Force coefficients for circular shapes shall be as given by fig. 24 and Appendix C.

4.9.3 Low walls and hoardings

4.9.3.1 Force coefficients for walls and hoardings less than 15 m high shall be as given by table 16 provided that the height shall be measured from the ground to the top of the wall or hoarding, and provided that for walls or hoardings above ground the clearance between the wall or hoarding and the ground shall be not less than 0.25 times the vertical dimension of the wall or hoarding.

4.9.3.2 To allow for oblique winds the design shall also be checked for the net pressure normal to the surface varying linearly from the maximum of C_p = 1.7 C_f at the upwind edge to C_p = 0.44 C_f at the downwind edge.

4.10 FORCE COEFFICIENTS FOR UNCLAD BUILDINGS

4.10.1 General

4.10.1.1 This section applies to permanently unclad buildings and to frameworks of buildings while temporarily unclad.

4.10.1.2 For buildings that, because of their size and the design wind speed, are in the super-critical flow regime, a check shall be made on the loads occurring at lower wind speeds when the flow is subcritical.

C4.10.1.3 It is only rarely that the effects of icing will need to be considered in New Zealand. For guidance on these effects designers should consult Appendix F of BSCP 3: Chapter V: Part 2.*

4.10.1.3 The effects of icing are not allowed for in this standard.

4.10.2 Individual members

C4.10.2.1 For members of circular section d = *diameter* D.

4.10.2.1 The force coefficients for individual members given below refer to members of infinite length. For members of finite length those force coefficients shall be multiplied by the appropriate value of K as given by table 17 according to the length l of the member and its width d across the wind, provided that:

(a) Where any member abuts on to a plate or wall in such a way that free flow of air around that end of the member is prevented, then the ratio l/d shall be doubled for the purpose of determining K; and

(b) When both ends of a member are so obstructed the ratio l/d shall be taken as infinity for the purpose of determining K.

* See list of related documents.

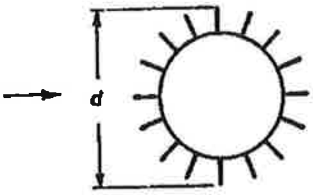
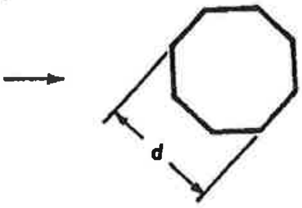
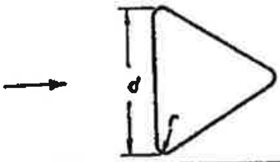
Plan shape	Description	$h/d \leq 1$	$h/d=5$	$h/d=20$
	Ribbed cylinder, ribs> 0.01 d	0.8	1.0	1.2
		1.0	1.2	1.4
	$r/d = 0.25, dV_s < 7$ $r/d = 0.25, dV_s > 7$ $r/d = 0.01$	0.8 0.4 1.0	1.0 0.5 1.2	1.3 0.7 1.6
	$r/d = 0.25, dV_s < 7$ $r/d = 0.25, dV_s > 7$ $r/d = 0.01$	0.9 0.4 1.3	1.2 0.5 1.6	1.5 0.7 2.2

Fig. 23 FORCE COEFFICIENTS FOR SOLID SHAPES MOUNTED ON A SURFACE

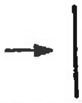
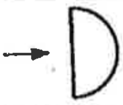
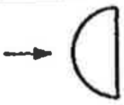
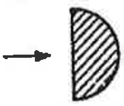
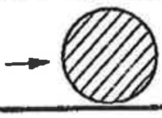
Side elevation	Description of shape	C_f
	Circular Disc	1·2
	Hemispherical Bowl	1·4
	Hemispherical Bowl	0·4
	Hemispherical Solid	1·2
	Spherical Solid	$b\ V_s < 7 - 0.5$ $b\ V_s > 7 - 0.2$

Fig. 24 FORCE COEFFICIENTS FOR CIRCULAR SHAPES

C4.10.2.2 The force coefficients in fig. 25 are given for two mutually-perpendicular directions relative to a reference axis on the member. They are designated C_{fn} *and* C_{ft} *and give the forces normal and transverse, respectively, to the reference plane as shown in fig. 25:*

Normal force $F_n = C_{fn}qkld$

Transverse force $F_t = C_{ft}qkld$

It should be noted that in fig. 25 the force coefficient relates to the dimension d *as shown and not, as in other cases, to the effective frontal area* A_e.

4.10.2.2 Force coefficients for wind normal to the longitudinal axis of flat-sided members shall be as given by fig. 25.

C4.10.2.3 The values of C_f *given in table 18A are suitable for all surfaces of evenly distributed roughness of height less than 0.01 diameter, that is, for all normal surface finishes. A description of supercritical flow and Reynolds number* Re *is given in Appendix C.*

4.10.2.3 Force coefficients for members of circular sections shall be as given by table 18A according to the diameter D of the member and the design wind speed V_s.

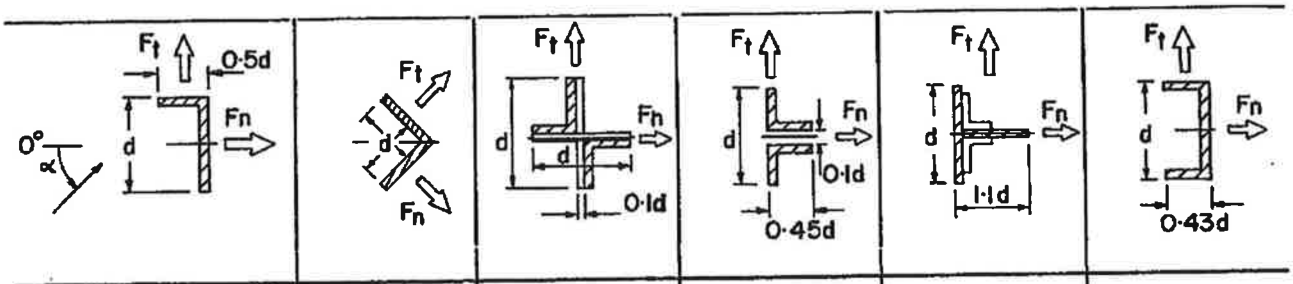
Table 18 **FORCE COEFFICIENTS FOR CIRCULAR MEMBERS**

A: Individual members of circular section and infinite length having roughness of height less than 0.01 diameter.

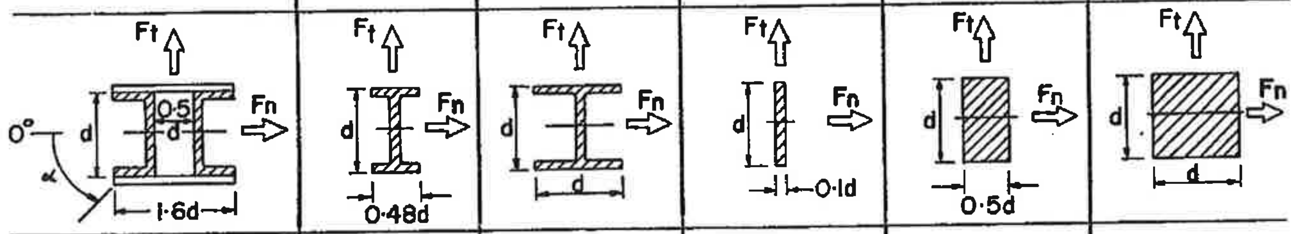
	Flow regime	C_f
Subcritical flow	$DV_s < 6$ m²/s $R_e < 4.1 \times 10^5$	1.2
Supercritical flow	$6 \leq DV_s < 12$ m²/s $4.1 \times 10^5 \leq Re < 8.2 \times 10^5$	0.6
	$12 \leq DV_s < 33$ m²/s $8.2 \times 10^5 \leq Re < 22.6 \times 10^5$	0.7
	$DV_s \geq 33$ m²/s $Re \geq 22.6 \times 10^5$	0.8

B: Wires and cables of infinite length having roughness of height greater than 0.1 diameter.

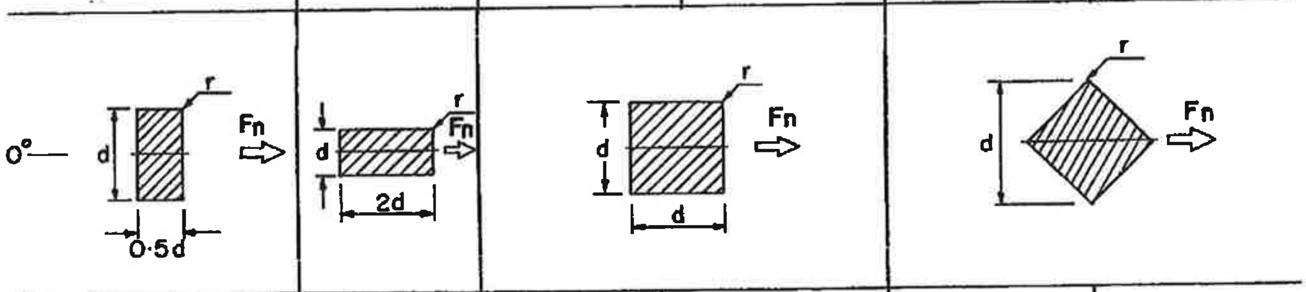
Flow regime	C_f for:			
	Smooth surface wire	*Moderately smooth wire (galvanized or painted)*	*Fine stranded cables*	*Thick stranded cables*
$DV_s < 0.6$ m²/s	–	–	1.2	1.3
$DV_s \geq 0.6$ m²/s	–	–	0.9	1.1
$DV_s \leq 6$ m²/s	1.2	1.2	–	–
$DV_s \geq 6$ m²/s	0.5	0.7	–	–



α	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}
degrees												
0	+1.9	+0.95	+1.8	+1.8	+1.75	+0.1	+1.6	0	+2.0	0	+2.05	0
45	+1.8	+0.8	+2.1	+1.8	+0.85	+0.85	+1.5	-0.1	+1.2	+0.9	+1.85	+0.6
90	+2.0	+1.7	-1.9	-1.0	+0.1	+1.75	-0.95	+0.7	-1.6	+2.15	0	+0.6
135	-1.8	-0.1	-2.0	+0.3	-0.75	+0.75	-0.5	+1.05	-1.1	+2.4	-1.6	+0.4
180	-2.0	+0.1	-1.4	-1.4	-1.75	-0.1	-1.5	0	-1.7	±2.1	-1.8	0



α	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}
degrees												
0	+1.4	0	+2.05	0	+1.6	0	+2.0	0	+2.1	0	+2.0	0
45	+1.2	+1.6	+1.95	+0.6	+1.5	+1.5	+1.8	+0.1	+1.4	+0.7	+1.55	+1.55
90	0	+2.2	+0.5	+0.9	0	+1.9	0	+0.1	0	+0.75	0	+2.0



	$\frac{r}{d} = 0.16$	$\frac{r}{d} = 0.16$	$\frac{r}{d} = 0.16$	$\frac{r}{d} = 0.33$	$\frac{r}{d} = 0.125$	$\frac{r}{d} = 0.25$
dV_s	C_{fn}	C_{fn}	C_{fn}	C_{fn}	C_{fn}	C_{fn}
<7	+1.8	+0.7	+1.2	+1.0 ($dV_s < 4$)	+1.6	+1.5
>7	+0.7	+1.2	+0.7	+0.5 ($dV_s > 4$)	+1.6	+0.7

Fig. 25 FORCE COEFFICIENTS FOR INDIVIDUAL STRUCTURAL MEMBERS OF INFINITE LENGTH

C4.10.3 Since the wind can come in any direction the most unfavourable load condition must be taken. In general, the wind load on a single frame should be calculated for the condition where the wind is at right angles to the frame unless it can be shown that another wind angle is appropriate.

Table 19

FORCE COEFFICIENTS FOR SINGLE FRAMES COMPLYING WITH CLAUSE 4.10.3.1

ϕ	C_f for:		
	Flat sided members	*Circular members in:*	
		Subcritical flow $DV_s < 6\ m^2/s$	*Supercritical flow* $DV_s \geq 6\ m^2/s$
0.1	1.9	1.2	0.7
0.2	1.8	1.2	0.8
0.3	1.7	1.2	0.8
0.4	1.7	1.1	0.8
0.5	1.6	1.1	0.8
0.75	1.6	1.5	1.4
1.0	2.0	2.0	2.0

C4.10.3.2 Where single frames are composed of circular-section members it is possible that the larger members will be in the supercritical flow regime ($DV_s \geq 6\ m^2/s$*) and the smaller members will not (*$DV_s < 6\ m^2/s$*), and there may also be some details fabricated from flat-sided sections.*

4.10.2.4 Force coefficients for wires and cables shall be as given by table 18B according to the diameter D, the design wind speed V_s, and the surface roughness.

4.10.3 Single frames

4.10.3.1 Force coefficients for a single frame having *either:*

(a) All flat-sided members; *or*

(b) All circular members

and in which all members of the frame have *either*:

(c) DV_s less than 6 m²/s; *or*

(d) DV_s greater than 6 m²/s

shall be as given by table 19 according to the type of member, the diameter D, the design wind speed V_s, and the solidity ratio ϕ, where ϕ is equal to the effective area of a frame normal to the wind direction divided by the area enclosed by the boundary of the frame normal to the wind direction.

4.10.3.2 Force coefficients for a single frame not complying with clause 4.10.3.1 shall be calculated from:

$$C_f = ZC_{f.\,super} + (1 - Z)\frac{A_{circ.\,sub}}{A_{sub}} C_{f.\,sub} + (1 - Z)\frac{A_{flat}}{A_{sub}} C_{f.\,flat} \quad \ldots\ldots\ldots\ldots (44)$$

where:

$C_{f.\,super}$ = force coefficient for the supercritical circular members as given by table 18

$C_{f.\,sub}$ = force coefficient for the subcritical circular members as given by table 18

$C_{f.\,flat}$ = force coefficient for the flat sided members as given by fig. 25

$A_{circ.\,sub}$ = effective area of the subcritical circular members

A_{flat} = effective area of the flat-sided members

A_{sub} = $A_{circ.\,sub} + A_{flat}$

Z = (Area of the frame in a supercritical flow)/A_e.

C4.10.4 Clause 4.10.4 applies to unclad buildings having two or more parallel frames where the windward frame may have a shielding effect on the frames to leeward. Where there are more than two frames of similar geometry and spacing, the wind load on the third and subsequent frames should be taken as equal to that on the second frame. The loads on the various frames should be added together to obtain the total load on the structure. The use of the aerodynamic solidity ratio β enables all cross-sections of single members to be incorporated.

4.10.4 Multiple-frame buildings

4.10.4.1 Force coefficients for multiple-frame unclad buildings shall be:

(a) For the windward frame and any unsheltered parts of other frames: As given by clause 4.10.3.

(b) For frames sheltered by the windward frame: As given by clause 4.10.3 multiplied by a shielding factor η as given by table 20 according to the spacing ratio of the frames as given by clause 4.10.4.2 and the aerodynamic solidity ratio β of the windward frame as given by clause 4.10.4.3.

4.10.4.2 The spacing ratio shall be the distance, centre to centre, of the frames, beams, or girders divided by the least overall dimension of the frame, beam, or girder measured at right angles to the direction of the wind. For triangular framed unclad buildings or rectangular framed unclad buildings diagonal to the wind the spacing ratio shall be calculated from the mean distance between the frames in the direction of the wind.

4.10.4.3 The aerodynamic solidity ratio β shall be the solidity ratio ϕ as given by clause 4.10.3.1 multiplied by:

1.6 for flat-sided members;

1.2 for circular members in the subcritical regime and for flat-sided members in conjunction with such circular members;

0.5 for circular members in the supercritical regime and for flat-sided members in conjunction with such circular members.

C4.10.5 Lattice towers of square and equilateral-triangle section constitute a special case of multiple-frame unclad buildings for which it may be convenient to use an overall force coefficient in the calculation of wind load.

4.10.5 Lattice towers

4.10.5.1 Force coefficients for lattice towers of square or equilateral-triangle section with flat-sided members for the wind blowing against any face shall be as given by table 21.

Table 20 **SHIELDING FACTOR η**

Spacing ratio	η for β of:							
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8 and over
up to 1.0	1.0	0.96	0.90	0.80	0.68	0.54	0.44	0.37
2.0	1.0	0.97	0.91	0.82	0.71	0.58	0.49	0.43
3.0	1.0	0.97	0.92	0.84	0.74	0.63	0.54	0.48
4.0	1.0	0.98	0.93	0.86	0.77	0.67	0.59	0.54
5.0	1.0	0.98	0.94	0.88	0.80	0.71	0.64	0.60
6.0 and over	1.0	0.99	0.95	0.90	0.83	0.75	0.69	0.66

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

4.10.5.3 For equilateral-triangle lattice towers with flat-sided members the load may be assumed to be constant for any inclination of the wind to a face.

Table 21

FORCE COEFFICIENTS FOR SQUARE AND EQUILATERAL–TRIANGLE SECTION LATTICE TOWERS OF FLAT–SIDED MEMBERS

ϕ	C_f for: *Square towers*	*Equilateral-triangle towers*
0.1	3.8	3.1
0.2	3.3	2.7
0.3	2.8	2.3
0.4	2.3	1.9
0.5	2.1	1.5

C4.10.5.4 It is only in very few cases that lattice towers with circular members will have all members in the same flow regime, that is, either all subcritical or all supercritical. For these cases tables 22 and 23, which are based on actual measurements, give somewhat lower values than would be obtained by applying clause 4.10.4.

4.10.5.4 Force coefficients for lattice towers of square section with circular members all in the same flow regime may be as given by table 22.

Table 22

FORCE COEFFICIENTS FOR SQUARE SECTION LATTICE TOWERS OF CIRCULAR MEMBERS ALL IN THE SAME FLOW REGIME

ϕ *of front face*	C_f for: *All members in flow* $DV_s < 6\ m^2/s$		*All members in supercritical flow* $DV_s \geqslant 6\ m^2/s$	
	On to face	*On to corner*	*On to face*	*On to corner*
0.05	2.4	2.5	1.1	1.2
0.1	2.2	2.3	1.2	1.3
0.2	1.9	2.1	1.3	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

C4.10.5.5 Clause 4.10.5.4 applies also to clause 4.10.5.5.

4.10.5.5 Force coefficients for lattice towers of equilateral-triangle section with circular members all in the same flow regime may be as given by table 23.

Table 23

FORCE COEFFICIENTS FOR EQUILATERAL-TRIANGLE SECTION LATTICE TOWERS OF CIRCULAR MEMBERS ALL IN THE SAME FLOW REGIME

ϕ of front face	C_f for: All members in subcritical flow $DV_s < 6m^2/s$ (all wind directions)	C_f for: All members in supercritical $DV_s \geqslant 6m^2/s$ (all wind directions)
0.05	1.8	0.8
0.1	1.7	0.8
0.2	1.6	1.1
0.3	1.5	1.1
0.4	1.5	1.1
0.5	1.4	1.2

C4.11 For rectangular clad buildings it is generally necessary to take frictional force into account only when d/h *or* d/b *is greater than 4. For other buildings the frictional force is indicated where necessary in the clauses, figures, and tables for pressure and force coefficients.*

4.11 FRICTIONAL FORCE

4.11.1 The frictional force F' in the direction of the wind for rectangular clad buildings shall be:

$$F' = C'_f qA \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (45)$$

where A is the area of the surface under consideration and C'_f has the following values:

C'_f = 0.01 for smooth surfaces without corrugations or ribs across the wind direction.

C'_f = 0.02 for surfaces with corrugations across the wind direction.

C'_f = 0.04 for surfaces with ribs across the wind direction.

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APPENDIX A

CALCULATION OF HEIGHT ABOVE GROUND FOR BUILDINGS ON OR NEAR THE EDGE OF AN ESCARPMENT OR A RELATIVELY SUDDEN CHANGE IN GROUND LEVEL

For a building on or near the edge of an escarpment or a relatively sudden change in ground level, such that $E/y > 0.2$ as shown in fig. 26 and fig. 27, the height above ground shall be measured from an artificial ground datum as shown (dotted line marked Z_g) in fig. 26 and fig. 27.

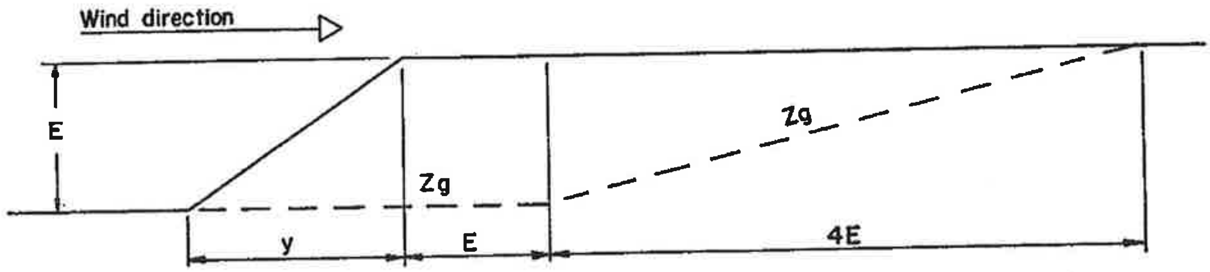

Fig. 26 ARTIFICIAL GROUND DATUM Z_g

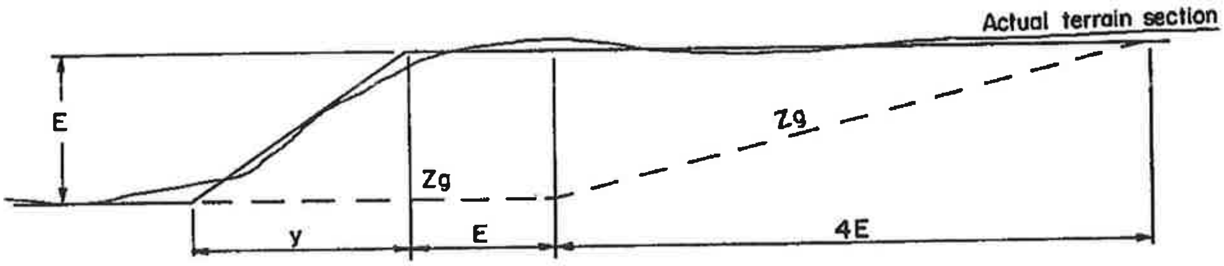

Fig. 27 TYPICAL CONSTRUCTION OF FIG. 26 ON ACTUAL TERRAIN SECTION

APPENDIX B

OSCILLATIONS CAUSED BY WIND

B1 General

The design of flexible structures such as chimneys, towers, tall lamp standards and some tall buildings, as well as some components of buildings, requires investigation of wind-induced oscillations. The risk of such oscillations occurring may, in principle, be reduced by making the structure stiffer and so raising its natural frequency or by increasing its damping. However, scope for making effective changes in this way is limited.

The following notes are given for guidance and approximate calculation. When refined estimates are required wind tunnel testing should be undertaken.

B2 Vortex shedding

When the wind blows past a bluff body vortices are shed alternatively from one side and the other of the body. This causes a periodic force to act at right angles to the direction of the wind on the body shedding the vortices and if the frequency of vortex shedding is close to a natural frequency of the structure, resonance is possible. The response of the structure is then a periodic deflection across the wind and the amplitude of this deflection, and the accompanying stresses, may be excessive.

The frequency of vortex shedding is given by:

$$n = \frac{SV}{b}$$

where

n = frequency of vortex shedding (Hz)

S = Strouhal number given below

V = mean wind speed (m/s)

b = reference dimension of structure as defined hereafter (m)

For circular cylinders of diameter b

S = 0.2	$Vb \leqslant 5\ \mathrm{m^2/s}$
S = between 0.2 and 0.4	$5\ \mathrm{m^2/s} < Vb \leqslant 50\ \mathrm{m^2/s}$
S = 0.4	$50\ \mathrm{m^2/s} < Vb$

In the range $5\ \mathrm{m^2/s} \leqslant Vb \leqslant 50\ \mathrm{m^2/s}$ vortex shedding is less regular and S depends on the motion of the structure.

For bodies with angular section, such as, for example, rectangular and rolled steel shapes,

S = 0.15

where b is the projected cross-wind dimension.

B3 Resonance

If n is put equal to a natural frequency of the structure or structural element, a critical velocity V_c can be estimated for which resonance is likely. A gust duration of about five periods is enough for almost full amplitude oscillations to develop. To avoid resonance the relevant frequency must be such as to make the critical wind speed greater than the speed in a gust of duration equal to, say, five times the natural period. The average speed (V_t) of the wind in gusts of 5 to 60 seconds duration may be determined from the three second gust using table 24 which is based on measurements made at heights of about 10 m, (see reference 16 of Appendix D). In the absence of results for other heights, they may be taken to apply at all heights.

If it is not possible to avoid resonance, its effects may be allowed for approximately by designing for an equivalent steady transverse force F given by

$$F = \frac{0.5}{\beta} C_L b L q_{cr}$$

where L is unsupported length,

C_L is a lateral force coefficient,

β is the critical damping ratio and,

q_{cr} is the dynamic pressure for the critical wind speed given by

$$q_{cr} = 0.613\, V_c^2$$

where q_{cr} is in Pa, and V_c in m/s.

Table 24 RATIO V_t/V

V_t = average speed in a gust of duration t

V = average speed in a gust of duration 3 seconds

t (second)	5	10	20	30	60
V_t/V	0.95	0.90	0.86	0.82	0.76

Conservative values of C_L are 0.2 for circular cylinders and 1.0 for rectangular cylinders of approximately square cross-section. Suggested realistic values of β are 0.01 for steel frames and 0.01 to 0.02 for reinforced concrete frames. On the other hand β may be as low as 0.001 for welded steel stacks. The regular character of vortex shedding may be destroyed by fitting aerodynamic "spoilers" such as helical strakes or a perforated shroud to the upper part of the structure. Such devices may be adequate to reduce the amplitude or oscillation to tolerable levels.

B4 Galloping

Galloping is another wind-induced oscillation that can occur in a steady wind. Structures likely to suffer from this form of wind driving have low mass, low damping, and cross-sections with sharp corners, such as triangular, square, rectangular or cruciform – for example a light lattice tower clad for appearance. Wind tunnel testing is recommended; testing for the onset speed is straightforward.

B5 Random loading

The natural air flow is unsteady and the wind load on any structure or structural element fluctuates with time. This is different from the effects of vortex shedding and galloping which are treated above and which arise from the interaction of a steady wind and a body in it. The dynamic overshoot treated in clause 4.6 allows for the response of the building to this kind of loading in the worst single gust. At lower wind speeds than this, there will be continuous random deflection about the mean deflected position. Such effects may be most severe near the wakes of prominent topographical features or other upstream buildings. The peak amplitude may be of concern. Statistical analysis of peak amplitudes has been investigated by Davenport (see references 13 and 17 of Appendix D).

APPENDIX C

WIND FORCES ON CIRCULAR SECTIONS

The wind force on any object is given by

$$F = C_f A_e q$$

where C_f is the force coefficient (see sections 4.4, 4.8, 4.9, and 4.10).

A_e is the effective area of the object normal to the wind direction, and

q is the dynamic pressure of the wind (see clause 4.4.1).

For most shapes the force coefficient remains approximately constant over the whole range of wind speeds likely to be encountered. However, for objects of circular cross-section it varies considerably.

For a circular section the force coefficient depends upon the way in which the wind flows around it and is dependent upon the velocity and kinematic viscosity of the wind and diameter of the section. The force coefficient is usually quoted against a non-dimensional parameter, called the Reynolds number, which takes account of the velocity and viscosity of the flowing medium (in this case the wind) and the member diameter.

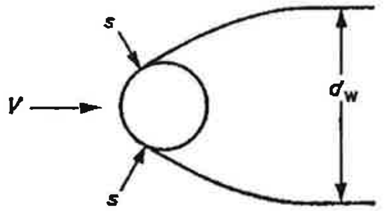

Fig. 28 WAKE IN SUBCRITICAL FLOW

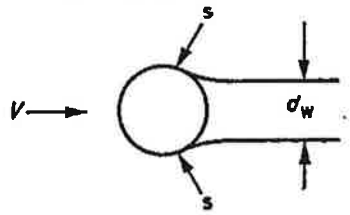

Fig. 29 WAKE IN SUPERCRITICAL FLOW

Reynolds number, $Re = \dfrac{DV_s}{\nu}$

where

D is the diameter of the member,

V_s is the design wind speed, and

ν is the kinematic viscosity of the air, which is 1.46×10^{-5} m^2/s at 15 °C and standard atmospheric pressure.

Since in most natural environments likely to be found in New Zealand the kinematic viscosity of the air is fairly constant, it is convenient to use DV_s as the parameter instead of Reynolds number and this has been done in this Standard.

The dependence of a circular section's force coefficient upon Reynolds number is due to the change in the wake developed behind the body.

At a low Reynolds number the wake is as shown in fig. 28 and the force coefficient is typically 1.2. As Reynolds number is increased the wake gradually changes to that shown in fig. 29, that is, the wake width d_w decreases and the separation point, s, moves from the front to the back of the body.

As a result the force coefficient shows a sudden drop at a critical value of Reynolds number, followed by a gradual rise as Reynolds number is increased still further.

The variation of C_f with the parameter DV_s is shown in fig. 30 for infinitely long circular cylinders having various values of relative surface roughness (ϵ/D), when subjected to a wind having an intensity and scale of turbulence typical of built-up urban areas. The curve for a smooth cylinder ($\epsilon/D = 1 \times 10^{-5}$) in a steady airstream, as found in a low-turbulence wind tunnel, is shown for comparison.

It can be seen that the main effect of free-stream turbulence is to decrease the critical value of the parameter DV_s. For subcritical flows, turbulence can produce a considerable reduction in C_f below the steady airstream values. For supercritical flows, this effect becomes significantly smaller.

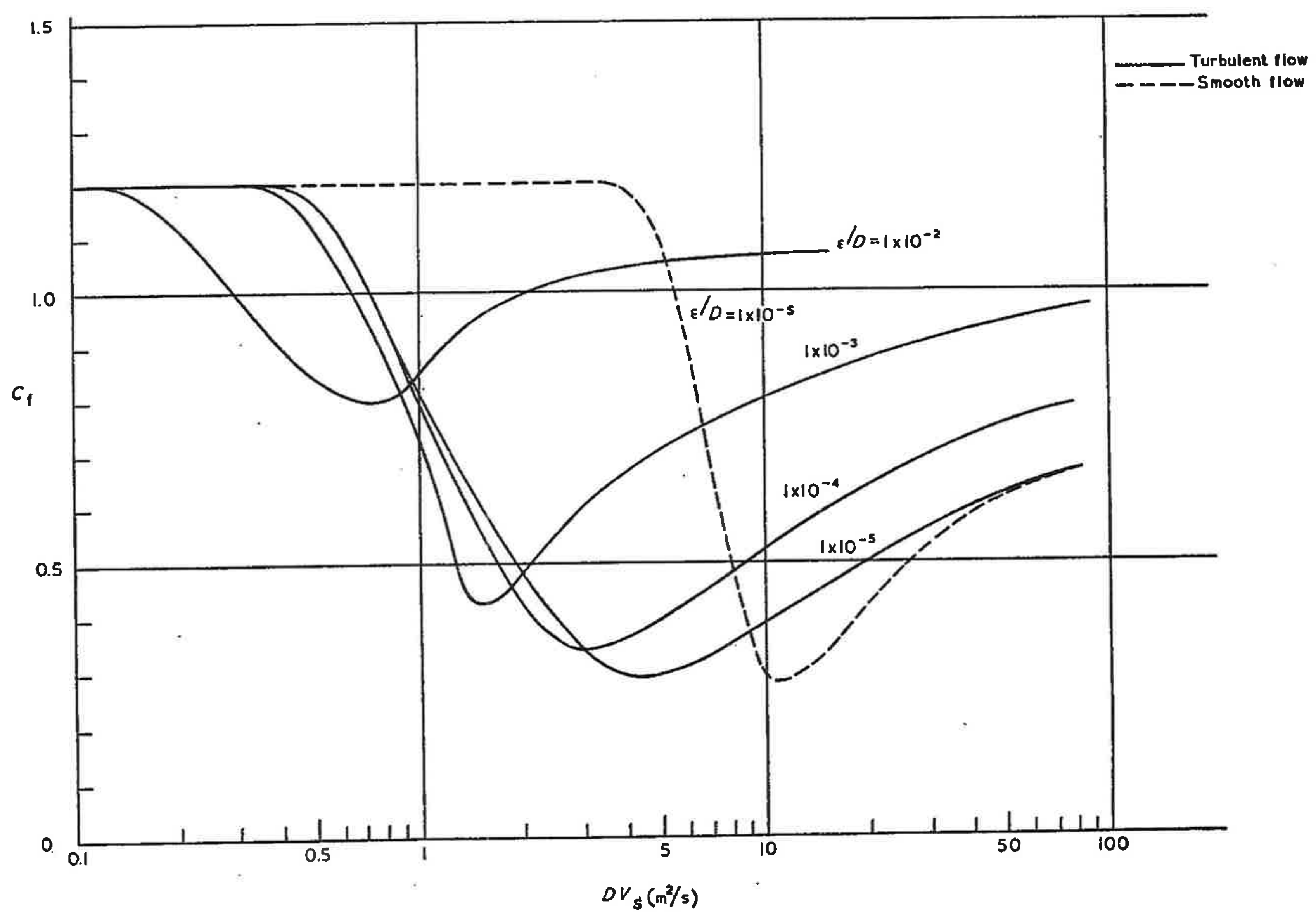

Fig. 30 VARIATION OF FORCE COEFFICIENT C_f WITH DV_s FOR INFINITELY LONG CIRCULAR CYLINDERS HAVING VARIOUS DEGREES OF SURFACE ROUGHNESS

APPENDIX D

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· NEW ZEALAND STANDARD

NZS 4203:1984

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

Pr 00

AMENDMENT No. 1

July 1992

EXPLANATORY NOTE – This Amendment applies when this Standard is used as a Verification Method that is referenced in Approved Document B1 Structure – General, to the New Zealand Building Code. The Amendment does not apply when this Standard is used under the Model Building Bylaw system which remains in operation until 31 December 1992.

To ensure receiving advice of the next amendment to NZS 4203:1984 please complete and return the amendment request form.

APPROVAL

Amendment No. 1 was approved in July 1992 by the Standards Council to be an amendment to NZS 4203:1984 pursuant to the provisions of section 10 of the Standards Act 1988.

(Amendment No. 1, July 1992)

1.1 SCOPE AND INTERPRETATION

1.1.1.1
In lines 4 and 5 **delete** the words "and is approved as a means of compliance with the relevant requirements of NZS 1900*" and **substitute** a new sentence: "It is cited as a Verification Method in Approved Document B1 Structure - General, of the New Zealand Building Code".

1.1.1.2
In line 2 **delete** the word "Engineer" and **substitute** "design engineer".

1.1.3.1
Add a new definition:

"DESIGN ENGINEER means any person who, on the basis of experience or qualifications, is competent to design structural elements of the building under consideration to safely resist the loads likely to be imposed on the building".

(Amendment No. 1, July 1992)

1.5 CONSTRUCTION REQUIREMENTS
Delete the section and **substitute:**

"**1.5 LOADING NOTICE**

1.5.1 Signs indicating allowable floor loadings shall be provided in accordance with NZBC F8".

(Amendment No. 1, July 1992)

Table 3
Delete the table and **substitute** the following:

Table 3

MINIMUM BASIC LIVE LOADS FOR ROOFS

Type of roof	Lu (kPa)	Lc (kN)
1. Roofs and verandas with access for: fire escape, roof garden, light storage and general pedestrian traffic, and also roofs and verandas where people can be expected to congregate on occasions irrespective of access, where no load will exceed 2.0 kPa.	2.0	1.0
2. Gantry roof over public way.		
(a) Where materials are stacked on, or crane loads are carried over the roof	7.0 (See note (1))	0.5 Plus impact (See note (2))
(b) Where it supports a site office	3.5 (See note (1))	0.5 Plus impact (See note (2))
3. Other roofs and verandas	0.25	1.0

NOTE –
(1) Loads are to be calculated, but design loadings shall be no less than shown.

(2) Impact shall be calculated on the basis of a compact mass (specific gravity = 1.0) equal to the value of the concentrated load specified falling from the top of the construction.

(Amendment No. 1, July 1992)

REQUEST FOR ADVICE OF NEXT AMENDMENT

Please fill in the request for the next amendment to this New Zealand Standard and mail to Standards New Zealand, Private Bag, Wellington.

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NZS 4203:1984
Amendment No. 2

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Provisions for wind loading:
New edition of NZS 4203 published

As reported in the April 1984 issue of **Standards**, Amendment No. 3 to NZS 4203 **General structural design and design loadings for buildings**, was not being made available separately, but was to be incorporated in a reprint of NZS 4203. This new edition was released at the end of last year.

Commenting on the new edition, Mr Latham Andrews, a member of the Probabilistic Design Committee said that it had been hoped to make major changes to Part 4 **Wind loads**, as much of this part is based on Australian Standard AS 1170 (SAA Loadings Code) Part 2 **Wind forces**, which has recently been revised. Unfortunately the publication of AS 1170 : Part 2 : 1983 came too late for the new material to be incorporated in NZS 4203, Amendment No. 3. Consequently, the wind load section is somewhat out of date and largely conservative.

Designers who wish to make use of more up to date information on wind loading are recommended to refer to an article published in **New Zealand Engineering** of February 1984 by Dr. S.J. Reid, New Zealand Meteorological Service, entitled **Basic wind speeds for engineering design** for use in conjunction with NZS 4203, table 6, and to AS 1170 : Part 2: 1983 for pressure coefficients.

It is recommended that if a designer proposes to use this approach, he first checks with the Local Authority concerned that the basis for his design for wind loading will be acceptable.

NZS 4203 : 1984
Code of practice for
GENERAL STRUCTURAL DESIGN AND
DESIGN LOADINGS FOR BUILDINGS

February 1985

ERRATUM

In the 1984 edition the following table was omitted.

Page 68, Clause 4.4 *Add* table 15:

Table 15 DYNAMIC PRESSURE q (Pa)

V_s m/s	*0*	*1.0*	*2.0*	*3.0*	*4.0*	*5.0*	*6.0*	*7.0*	*8.0*	*9.0*
10	61	74	88	104	120	138	157	177	199	221
20	245	270	297	324	353	383	414	447	481	516
30	552	589	628	668	709	751	794	839	885	932
40	981	1030	1080	1130	1190	1240	1300	1350	1410	1470
50	1530	1590	1660	1720	1790	1850	1920	1990	2060	2130
60	2210	2280	2360	2430	2510	2590	2670	2750	2830	2920
70	3000									