Structural Design Actions

Part 5 : Earthquake actions – New Zealand

NZS 1170.5:2004

NZS 1170.5:2004

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New Zealand Standard™

Structural design actions

Part 5: Earthquake actions – New Zealand

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PREFACE

This Standard was prepared by the Standards New Zealand Technical Committee BD/6/4/11. This was initially a Joint Standards Australia/Standards New Zealand Technical Subcommittee of Committee BD-006, General Design Requirements and Loading on Structures which was to develop a Joint Australia/New Zealand Earthquake Actions Standard. The development of a Joint Earthquake Standard proved to be impractical and hence separate country specific Standards were proceeded with as part of the 1170 suite of Standards. It follows the philosophy and principles that are set out in the Preface to Part 0.

This Standard, taken together with the other joint Parts of the 1170 suite is to supersede NZS 4203:1992, *General structural design and design loadings for buildings*. However, NZS 4203 will remain current in New Zealand for a transition period after publication to allow projects currently under way to be completed and for the Department of Building and Housing to follow its processes for the citation of the 1170 series of Standards as verification methods for the New Zealand Building Code.

The 1170 series, *Structural design actions*, comprises the following parts, each of which has an accompanying Commentary that is published as a Supplement:

AS/NZS 1170 *Structural design actions*

AS/NZS 1170.0:2002 Part 0: *General principles*

AS/NZS 1170.1:2002 Part 1: *Permanent, imposed and other actions*

AS/NZS 1170.2:2002 Part 2: *Wind actions*

AS/NZS 1170.3:2003: Part 3: *Snow and ice actions*

NZS 1170 *Structural design actions*

NZS 1170.5:2004 Part 5: *Earthquake actions – New Zealand*

(Also to be published is AS 1170.4, *Earthquake actions – Australia* which will have application only in Australia.)

The Commentary to this Standard is NZS 1170.5 Supplement 1, *Structural design actions—Earthquake actions – New Zealand – Commentary (Supplement to* NZS 1170.5:2004). The Commentary is intended to provide background to the various provisions in the Standard, to suggest approaches that may satisfy the intent of the Standard, and if appropriate, describe differences between this and previous editions of the Standard. References are provided for further reading and these are given at the end of each section of the Commentary.

The terms 'normative' and 'informative' have been used in this Standard to define the application of the Appendix to which they apply. A 'normative' Appendix is an integral part of a Standard, whereas an 'informative' Appendix is only for information and guidance. The Standard includes Appendix D 'Requirements for material design Standards' as an informative Appendix that is included to guide developers of material specific structural design Standards that are intended to be used in conjunction with NZS 1170.5. Appendix D does not have application to the design of a specific structure where a conforming material specific structural design Standard is being used.

CONTENTS

STANDARDS NEW ZEALAND

New Zealand Standard

Structural design actions

Part 5: Earthquake actions – New Zealand

SECTION 1 SCOPE AND GENERAL

1.1 SCOPE

This Standard, NZS 1170.5, sets out procedures and criteria for establishing the earthquake actions to be used in the limit state design of structures and parts of structures within New Zealand that are within the scope of AS/NZS 1170.0.

The design of the following structures for earthquake actions is outside the scope of this Standard:

(a) Bridges.

(b) Tanks containing liquids.

(c) Civil structures including dams and bunds.

(d) Offshore structures that are partly or fully immersed.

(e) Soil retaining structures.

This Standard does not address the effects of slope instability and/or liquefaction resulting from earthquake shaking.

This Standard is to be used in conjunction with material specific structural design Standards that comply with the requirements of Appendix D. Such a Standard is referred to here as an appropriate material Standard.

1.2 DETERMINATION OF EARTHQUAKE ACTIONS

Earthquake actions for use in design, E_u for ultimate limit state and E_s for serviceability limit state, shall be appropriate for the type of structure or element, its intended use, design working life and exposure to earthquake shaking. Earthquake actions determined in accordance with this Standard shall be deemed to comply with the requirements of this Clause.

1.3 LIMIT STATES

The requirements for the ultimate and serviceability limit states as defined in AS/NZS 1170.0 are deemed to be satisfied by compliance with this Standard.

1.4 SPECIAL STUDIES

A special study shall be carried out to justify any departure from or extension of the provisions of this Standard. Such studies are outside the scope of this Standard but some guidance is given in AS/NZS 1170.0, Appendix A. Where a special study is carried out to justify departure from this Standard, the minimum provisions of this Standard that are not specifically addressed by the special study shall still apply, including the maintenance of the underpinning principles and performance objectives of this Standard.

1.5 REFERENCED DOCUMENTS

The following documents are referred to in this Standard:

NZS

3101 Concrete Structures Standard

3101.1 & 3101.2:1995 Parts 1 & 2: The design of concrete structures

AS/NZS

1170 Structural design actions

1170.0:2002 Part 0: General principles

1170.1:2002 Part 1: Permanent, imposed and other actions

1170.3:2003 Part 3: Snow and ice actions

Building Industry Authority

The New Zealand Building Code Handbook and Approved Documents

1.6 UNITS

Except where specifically noted, this Standard uses SI units of kilograms, metres, seconds, Pascals and Newtons (kg, m, s, Pa, N).

1.7 DEFINITIONS

Definitions of the terms used in this Standard shall be as given in Appendix A.

1.8 NOTATION

The notation used in this Standard shall be as given in Appendix B.

SECTION 2 VERIFICATION

2.1 GENERAL REQUIREMENTS

2.1.1 General

All structures shall comply with the requirements for the ultimate limit state and the serviceability limit state as set out in Clauses 2.3 and 2.4 and the appropriate material Standard.

2.1.2 Structural systems

All structures shall be configured with a clearly defined load path, or paths, to transfer the earthquake actions (both horizontal and vertical) generated in an earthquake together with gravity loads to the supporting foundation soil. All elements shall be capable of performing their required function while sustaining the deformation of the structure resulting from the application of the earthquake actions determined for each limit state.

2.1.3 Localized actions

Structural elements and members shall be tied together to enable the structure to act as a whole in resisting seismic actions. Consideration shall be given to actions induced in individual elements due to the displaced shape and the gravity loads.

2.1.4 Earthquake limit state design performance requirements

The design performance requirements are as follows:

(a) Ultimate limit state for earthquake loading shall provide for:

- (i) Avoidance of collapse of the structural system; and
- (ii) Avoidance of collapse or loss of support to parts of categories P.1, P.2, P.3 and P.4 (Section 8); and
- (iii) Avoidance of damage to non-structural systems necessary for emergency building evacuation, that renders them inoperative.

(b) Serviceability limit states for earthquake loading are to avoid damage to:

- (i) The structure and the non-structural components that would prevent the structure from being used as originally intended without repair after the SLS1 earthquake as defined in Clause 2.4; and
- (ii) In a structure with a critical post earthquake designation (i.e. importance level 4) all elements required to maintain those operations for which the structure is designated as critical, are to be maintained in an operational state or are to be returned to a fully operational state within an acceptable short timeframe (usually minutes to hours rather than days) after the SLS2 earthquake as defined in Clause 2.4.

2.2 STRUCTURAL TYPES

2.2.1 Ductile structures

A ductile structure is one where the structural ductility factor is greater than 1.25 but does not exceed 6.0.

2.2.2 Structures of limited ductility

Structures of limited ductility are a subset of ductile structures. A structure of limited ductility is one where the structural ductility factor is greater than 1.25 but less than 3.0.

2.2.3 Nominally ductile structures

A nominally ductile structure is one where the structural ductility factor is greater than 1.0 and equal to or less than 1.25.

2.2.4 Brittle structures

A brittle structure is defined as a structure with structural components that are not capable of inelastic deformation without undergoing sudden and significant loss of strength. The structural ductility factor, μ, for brittle structures shall be taken as 1.0.

2.3 ULTIMATE LIMIT STATE VERIFICATION

2.3.1 General

Structures shall be designed to ensure the performance requirements of Clause 2.1.4(a) are satisfied.

The ultimate limit state performance requirements are met when a structure, analysed in accordance with Section 6 for ULS design actions, satisfies the following:

(a) The ULS deflection criteria in Section 7;

(b) The design strength is equal to or greater than the design action specified by AS/NZS 1170.0 and this Standard; and

(c) The material strains in potential inelastic zones do not exceed the limiting values given in the appropriate material Standards.

The ULS design actions shall be derived in accordance with Section 5 using the structural characteristics determined in Section 4 and the site hazard spectra from Section 3 for the return period specified in Section 3 of AS/NZS 1170.0.

2.3.2 Ductility requirements

The assignment of the structural ductility factor, μ, shall be consistent with the capability of the associated detailing from the appropriate material Standard, with consideration of:

(a) Plastic mechanisms that develop within the structure because of inelastic zones forming in structural members that are required to sustain action effects.

(b) Material strain demands within inelastic zones of individual structural members.

(c) Material strain limits for potential inelastic zones, given in the appropriate material Standard.

(d) The inelastic displacement profiles calculated in accordance with Clause 7.2.1.

(e) Where an appropriate material Standard is not available, the structural ductility factor, μ, shall be determined by special study.

2.3.3 Capacity design

2.3.3.1 *Ductile structures and structures of limited ductility*

The provisions of Clause 5.3.1.1 together with the requirements for capacity design in Clause 5.6 shall be applied to all ductile structures including structures of limited ductility.

2.3.3.2 *Nominally ductile and brittle structures*

The provisions of Clause 5.3.1.2 shall be applied to nominally ductile and brittle structures together with the requirements for capacity design where required by the appropriate material Standard.

2.4 SERVICEABILITY LIMIT STATE VERIFICATION

Structures shall be designed to ensure the performance requirements of Clause 2.1.4(b) are satisfied.

The serviceability limit state performance requirements are met when a structure, analysed in accordance with Section 6 for serviceability limit state design actions, satisfies the following:

(a) The serviceability limit state deflection criteria in Section 7; and

(b) The design strength, as specified by the appropriate material Standard, equals or exceeds the serviceability design actions specified in AS/NZS 1170.0 and in Clause 7.5.2.

For structural systems that experience inelastic response under serviceability limit state actions, the calculated deflections shall include the deflections resulting from the total accumulated inelastic deformation in each potential inelastic zone.

The serviceability limit state design actions for SLS1 and SLS2 as appropriate shall be derived in accordance with Section 5 using the structural characteristics determined in Section 4 and the site hazard spectra from Section 3 for the return period specified in Section 3 of AS/NZS 1170.0.

2.5 DEFORMATION CONTROL

2.5.1 Ultimate limit state

Structure deformations shall be determined in accordance with Section 7.

Deformation shall be limited at the ultimate limit state as provided in Clauses 7.4 and 7.5 so that:

(a) The structural system continues to perform its load-bearing functions; and

(b) Damaging contact with neighbouring structures is avoided; and

(c) Parts of all categories except P.5 and P.7 shall continue to be supported; and

(d) Non-structural systems necessary for emergency structure evacuation shall continue to function; and

2.5.2 Serviceability limit state

Deformation shall be limited at the serviceability limit state so that:

(a) At the SLS1 level, structural system members and parts of structures shall not experience deformations that result in damage that would prevent the structure from being used as originally intended without repair.

(b) At the SLS2 level for structures of importance level 4, all parts of the structure shall remain operational so that the structure performs the role that has resulted in it being assigned this importance level.

2.6 STRUCTURE PARTS

All structure parts shall meet the requirements of Section 8.

SECTION 3 SITE HAZARD SPECTRA

3.1 ELASTIC SITE SPECTRA FOR HORIZONTAL LOADING

3.1.1 Elastic site spectra

The elastic site hazard spectrum for horizontal loading, $C(T)$, for a given return period shall be as given by Equation 3.1(1):

$$C(T) = C_h(T)\, Z\, R\, N(T,D) \qquad \ldots 3.1(1)$$

where

$C_h(T)$ = the spectral shape factor determined from Clause 3.1.2

Z = the hazard factor determined from Clause 3.1.4

R = the return period factor R_s or R_u for the appropriate limit state determined from Clause 3.1.5 but limited such that ZR_u does not exceed 0.7

$N(T,D)$ = the near-fault factor determined from Clause 3.1.6

3.1.2 Spectral shape factor, $C_h(T)$

The spectral shape factor, $C_h(T)$, shall be selected from Table 3.1, for the site subsoil class defined in Clause 3.1.3. The spectral shape factor functions are graphed in Figure 3.1 for general cases and in Figure 3.2 for values for the modal response spectrum and the numerical integration time history methods, to determine the $C(T)$ values required for vertical loading, and to determine the $C_h(0)$ values required to evaluate $C(0)$ for parts in Clause 8.2.

TABLE 3.1
SPECTRAL SHAPE FACTOR, C_h (T)

Period, T (seconds)	Spectral shape factor, $C_h(T)$ (g)			
	Site subsoil class			
	A Strong rock and B rock	C Shallow soil	D Deep or soft soil	E Very soft soil
0.0	1.89 (1.00)[1]	2.36 (1.33)[1]	3.00 (1.12)[1]	
0.1	1.89 (2.35)[1]	2.36 (2.93)[1]	3.00	
0.2	1.89 (2.35)[1]	2.36 (2.93)[1]	3.00	
0.3	1.89 (2.35)[1]	2.36 (2.93)[1]	3.00	
0.4	1.89	2.36	3.00	
0.5	1.60	2.00	3.00	
0.6	1.40	1.74	2.84	3.00
0.7	1.24	1.55	2.53	3.00
0.8	1.12	1.41	2.29	3.00
0.9	1.03	1.29	2.09	3.00
1.0	0.95	1.19	1.93	3.00
1.5	0.70	0.88	1.43	2.21
2.0	0.53	0.66	1.07	1.66
2.5	0.42	0.53	0.86	1.33
3.0	0.35	0.44	0.71	1.11
3.5	0.26	0.32	0.52	0.81
4.0	0.20	0.25	0.40	0.62
4.5	0.16	0.20	0.32	0.49

NOTE:

1 Values in brackets correspond to spectral values for the modal response spectrum and the numerical integration time history methods, to the $C(T)$ values required for vertical loading, and to the $C_h(0)$ values required to evaluate $C(0)$ for parts in Clause 8.2.

3.1.3 Site subsoil class

The site subsoil class shall be determined as being one of classes A to E from Clauses 3.1.3.2 to 3.1.3.6.

3.1.3.1 *Hierarchy for site classification methods*

The preferred site classification method is from site periods based on four times the shear-wave travel-time through material from the surface to underlying rock. The next preferred methods are from borelogs including measurement of geotechnical properties or by evaluation of site periods from Nakamura ratios or from recorded earthquake motions. Lacking this information, classification may be based on boreholes with descriptors but no geotechnical measurements. The least preferred method is from surface geology and estimates of the depth to underlying rock.

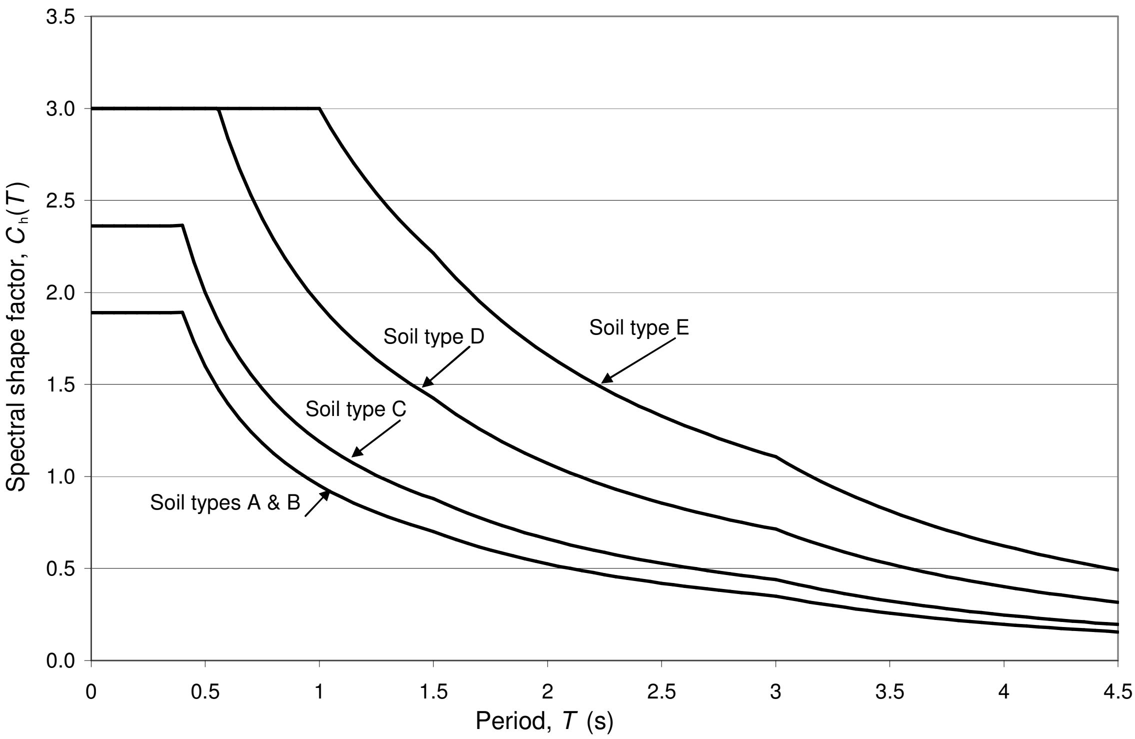


FIGURE 3.1 SPECTRAL SHAPE FACTOR, $C_h(T)$ – GENERAL

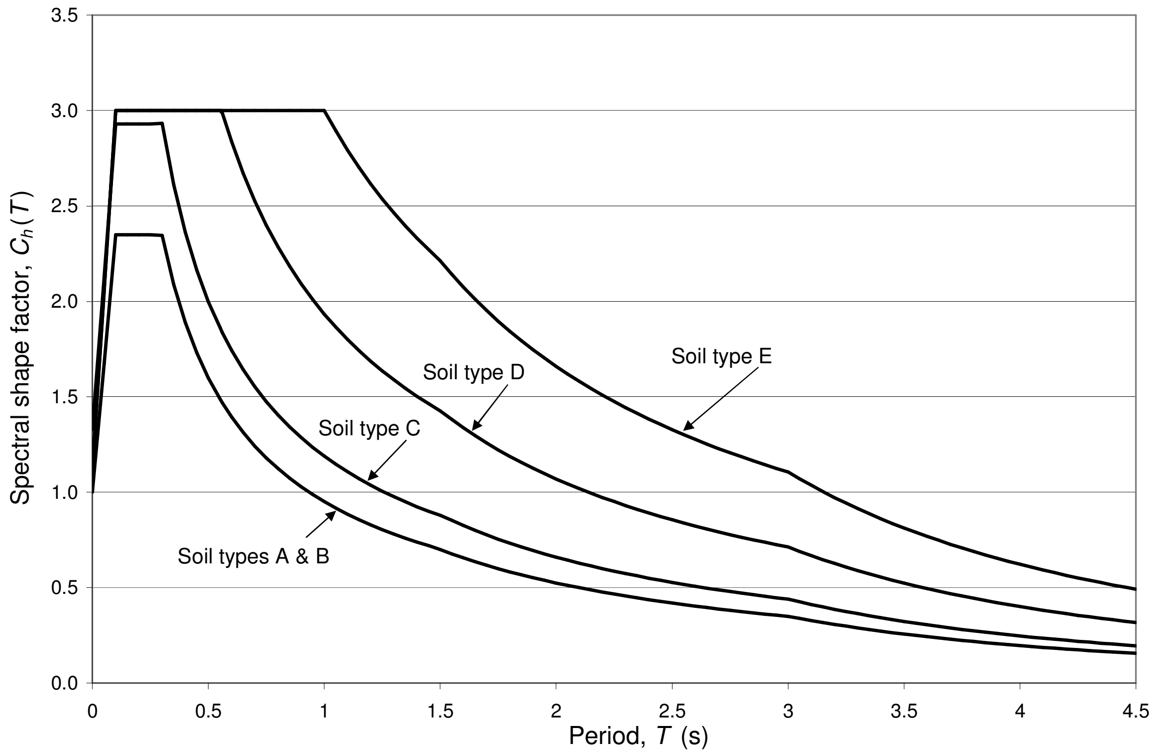


FIGURE 3.2 SPECTRAL SHAPE FACTOR, $C_h(T)$ FOR MODAL ANALYSIS, NUMERICAL INTEGRATION TIME HISTORY ANALYSIS, VERTICAL LOADING AND PARTS

3.1.3.2 *Class A – Strong rock*

Class A is defined as strong to extremely-strong rock with:

(a) Unconfined compressive strength greater than 50 MPa; and

(b) An average shear-wave velocity over the top 30 m greater than 1500 m/s; and

(c) Not underlain by materials having a compressive strength less than 18 MPa or a shear-wave velocity less than 600 m/s.

3.1.3.3 *Class B – Rock*

Class B is defined as rock with:

(a) A compressive strength between 1 and 50 MPa; and

(b) An average shear-wave velocity over the top 30 m greater than 360 m/s; and

(c) Not underlain by materials having a compressive strength less than 0.8 MPa or a shear-wave velocity less than 300 m/s.

A surface layer of no more than 3 m depth of highly-weathered or completely-weathered rock or soil (a material with a compressive strength less than 1 MPa) may be present.

3.1.3.4 *Class C – Shallow soil sites*

Class C is defined as sites where:

(a) They are not class A , class B or class E sites; and

(b) The low amplitude natural period is less than or equal to 0.6 s; or

(c) Depths of soil do not exceed those listed in Table 3.2.

The low amplitude natural period may be estimated from four times the shear-wave travel time from the surface to rock, be estimated from Nakamura ratios or from recorded earthquake motions, or be evaluated in accordance with Clause 3.1.3.7 for sites with layered subsoil, according to the hierarchy of methods given in Clause 3.1.3.1.

3.1.3.5 *Class D – Deep or soft soil sites*

Class D is defined as sites:

(a) That are not class A , class B or class E sites; and

(b) Where low-amplitude natural period is greater than 0.6 s; or

(c) With depths of soils exceeding those listed in Table 3.2; or

(d) Underlain by less than 10 m of soils with an undrained shear-strength less than 12.5 kPa or soils with SPT N-values less than 6.

The low amplitude natural period may be determined in accordance with Clause 3.1.3.4.

3.1.3.6 *Class E – Very soft soil sites*

Class E is defined as sites with:

(a) More than 10 m of very soft soils with undrained shear strength less than 12.5 kPa; or

(b) More than 10 m of soils with SPT N-values less than 6; or

(c) More than 10 m depth of soils with shear-wave velocities of 150 m/s or less; or

(d) More than 10 m combined depth of soils with properties as described in (a), (b) and (c) above.

TABLE 3.2

MAXIMUM DEPTH LIMITS FOR SITE SUBSOIL CLASS C

Soil type and description		**Maximum depth of soil** (m)
Cohesive soil	**Representative undrained shear strengths** (kPa)	
Very soft	< 12.5	0
Soft	12.5 – 25	20
Firm	25 – 50	25
Stiff	50 – 100	40
Very stiff or hard	100 – 200	60
Cohesionless soil	**Representative SPT N values**	
Very loose	< 6	0
Loose dry	6 – 10	40
Medium dense	10 – 30	45
Dense	30 – 50	55
Very dense	> 50	60
Gravels	> 30	100

3.1.3.7 *Evaluation of periods for layered sites*

For sites consisting of layers of several types of material, the low-amplitude natural period of the site may be estimated by summing the contributions to the natural period of each layer. The contribution of each layer may be estimated by multiplying 0.6 s by the ratio of the layer's thickness to that for its soil type in Table 3.2. In evaluating site periods, material above rock shall be included in the summation.

3.1.4 Hazard factor

The hazard factor, *Z*, shall be taken from Table 3.3 or interpolated from Figures 3.3 or 3.4 but shall not be taken as less than 0.13. For locations listed in Table 3.4, the hazard factor *Z* shall be taken from the Table rather than interpolated from Figures 3.1 or 3.2. Figures 3.3 or 3.4 are applicable only to onshore areas and not to offshore areas. The offshore contours are provided only for the purposes of interpolation to the shoreline.

TABLE 3.3

Z-VALUES AND SHORTEST MAJOR FAULT DISTANCES *D* FOR NEW ZEALAND LOCATIONS (North to South)

#	Location	*Z*	*D*(km)[1]
1	Kaitaia	0.13	-
2	Paihia/Russell	0.13	-
3	Kaikohe	0.13	-
4	Whangarei	0.13	-
5	Dargaville	0.13	-
6	Warkworth	0.13	-
7	Auckland	0.13	-
8	Manakau City	0.13	-
9	Waiuku	0.13	-
10	Pukekohe	0.13	-
11	Thames	0.16	-
12	Paeroa	0.18	-
13	Waihi	0.18	-
14	Huntly	0.15	-
15	Ngaruawahia	0.15	-
16	Morrinsville	0.18	-
17	Te Aroha	0.18	-
18	Tauranga	0.20	-
19	Mount Maunganui	0.20	-
20	Hamilton	0.16	-
21	Cambridge	0.18	-
22	Te Awamutu	0.17	-
23	Matamata	0.19	-
24	Te Puke	0.22	-
25	Putaruru	0.21	-
26	Tokoroa	0.21	-
27	Otorohanga	0.17	-
28	Te Kuiti	0.18	-
29	Mangakino	0.21	-
30	Rotorua	0.24	-
31	Kawerau	0.29	-
32	Whakatane	0.30	-
33	Opotiki	0.30	-
34	Ruatoria	0.33	-
35	Murupara	0.30	-
36	Taupo	0.28	-
37	Taumarunui	0.21	-
38	Turangi	0.27	-
39	Gisborne	0.36	-
40	Wairoa	0.37	-
41	Waitara	0.18	-
42	New Plymouth	0.18	-
43	Inglewood	0.18	-
44	Stratford	0.18	-
45	Opunake	0.18	-
46	Hawera	0.18	-
47	Patea	0.19	-

#	Location	*Z*	*D*(km)[1]
48	Raetihi	0.26	-
49	Ohakune	0.27	-
50	Waiouru	0.29	-
51	Napier	0.38	-
52	Hastings	0.39	-
53	Wanganui	0.25	-
54	Waipawa	0.41	-
55	Waipukurau	0.41	-
56	Taihape	0.33	-
57	Marton	0.30	-
58	Bulls	0.31	-
59	Feilding	0.37	-
60	Palmerston North	0.38	8 – 16
61	Dannevirke	0.42	10
62	Woodville	0.41	≤ 2
63	Pahiatua	0.42	8
64	Foxton/Foxton Beach	0.36	-
65	Levin	0.40	-
66	Otaki	0.40	-
67	Waikanae	0.40	15 – 20
68	Paraparaumu	0.40	14 – 20
69	Masterton	0.42	6 – 10
70	Porirua	0.40	8 – 12
71	Wellington CBD (north of Basin Reserve)	0.40	≤ 2
72	Wellington	0.40	0 – 8
73	Hutt Valley–south of Taita Gorge	0.40	0 – 4
74	Upper Hutt	0.42	≤ 2
75	Eastbourne–Point Howard	0.40	4 - 8
76	Wainuiomata	0.40	5 – 8
77	Takaka	0.23	-
78	Motueka	0.26	-
79	Nelson	0.27	-
80	Picton	0.30	16
81	Blenheim	0.33	0 – 5
82	St Arnaud	0.36	≤ 2
83	Westport	0.30	-
84	Reefton	0.37	-
85	Murchison	0.34	-
86	Springs Junction	0.45	3
87	Hanmer Springs	0.55	2 – 6
88	Seddon	0.40	6
89	Ward	0.40	4
90	Cheviot	0.40	-

TABLE 3.3

Z-VALUES AND SHORTEST MAJOR FAULT DISTANCES *D* FOR NEW ZEALAND LOCATIONS (North to South) (Continued)

#	Location	*Z*	*D*(km)[1]
91	Greymouth	0.37	-
92	Kaikoura	0.42	12
93	Harihari	0.46	4
94	Hokitika	0.45	-
95	Fox Glacier	0.44	≤ 2
96	Franz Josef	0.44	≤ 2
97	Otira	0.60	3
98	Arthurs Pass	0.60	12
99	Rangiora	0.33	-
100	Darfield	0.30	-
101	Akaroa	0.16	-
102	Christchurch	0.22	-
103	Geraldine	0.19	-
104	Ashburton	0.20	-
105	Fairlie	0.24	-
106	Temuka	0.17	-
107	Timaru	0.15	-
108	Mt Cook	0.38	-
109	Twizel	0.27	-
110	Waimate	0.14	-

#	Location	*Z*	*D*(km)[1]
111	Cromwell	0.24	-
112	Wanaka	0.30	-
113	Arrowtown	0.30	-
114	Alexandra	0.21	-
115	Queenstown	0.32	-
116	Milford Sound	0.54	-
117	Palmerston	0.13	-
118	Oamaru	0.13	-
119	Dunedin	0.13	-
120	Mosgiel	0.13	-
121	Riverton	0.20	-
122	Te Anau	0.36	-
123	Gore	0.18	-
124	Winton	0.20	-
125	Balclutha	0.13	-
126	Mataura	0.17	-
127	Bluff	0.15	-
128	Invercargill	0.17	-
129	Oban	0.14	-

NOTE:

1 The near-fault factor $N(T,D)$ shall be considered for locations with the distance D to the nearest major fault specified.

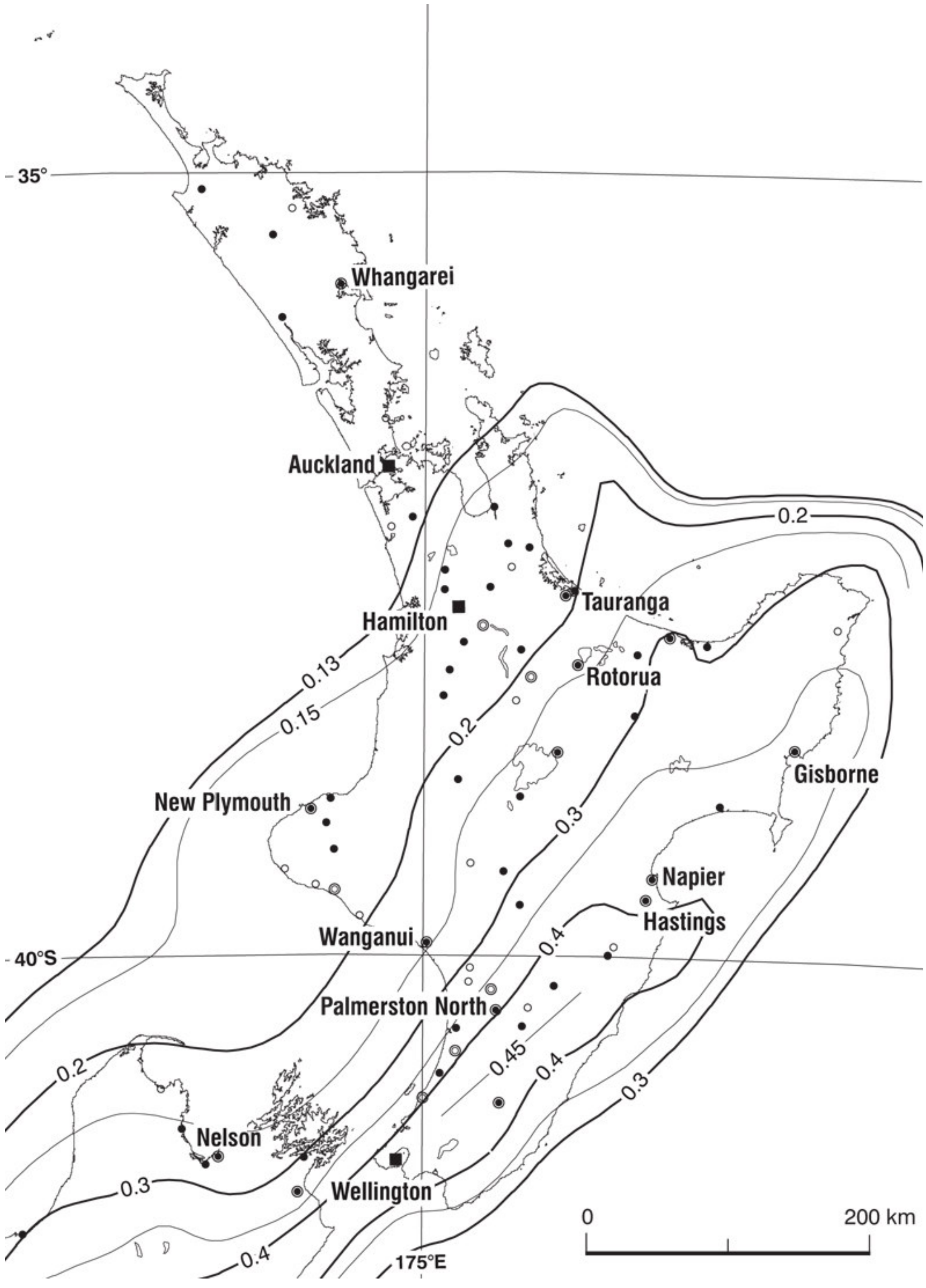
TABLE 3.4
ALPHABETICAL LIST OF LOCATIONS

Location	#
Akaroa	101
Alexandra	114
Arrowtown	113
Arthurs Pass	98
Ashburton	104
Auckland	7
Balclutha	125
Blenheim	81
Bluff	127
Bulls	58
Cambridge	21
Cheviot	90
Christchurch	102
Cromwell	111
Dannevirke	61
Darfield	100
Dargaville	5
Dunedin	119
Eastbourne–Point Howard	75
Fairlie	105
Feilding	59
Fox Glacier	95
Foxton/Foxton Beach	64
Franz Josef	96
Geraldine	103
Gisborne	39
Gore	123
Greymouth	91
Hamilton	20
Hanmer Springs	87
Harihari	93
Hastings	52
Hawera	46
Hokitika	94
Huntly	14
Hutt Valley–south of Taita Gorge	73
Inglewood	43
Invercargill	128
Kaikohe	3
Kaikoura	92
Kaitaia	1
Kawerau	31
Levin	65
Manakau City	8

Location	#
Mangakino	29
Marton	57
Masterton	69
Matamata	23
Mataura	126
Milford Sound	116
Morrinsville	16
Mosgiel	120
Motueka	78
Mount Maunganui	19
Mt Cook	108
Murchison	85
Murupara	35
Napier	51
Nelson	79
New Plymouth	42
Ngaruawahia	15
Oamaru	118
Oban	129
Ohakune	49
Opotiki	33
Opunake	45
Otaki	66
Otira	97
Otorohanga	27
Paeroa	12
Pahiatua	63
Paihia/Russell	2
Palmerston	117
Palmerston North	60
Paraparaumu	68
Patea	47
Picton	80
Porirua	70
Pukekohe	10
Putaruru	25
Queenstown	115
Raetihi	48
Rangiora	99
Reefton	84
Riverton	121
Rotorua	30
Ruatoria	34
Seddon	88
Springs Junction	86

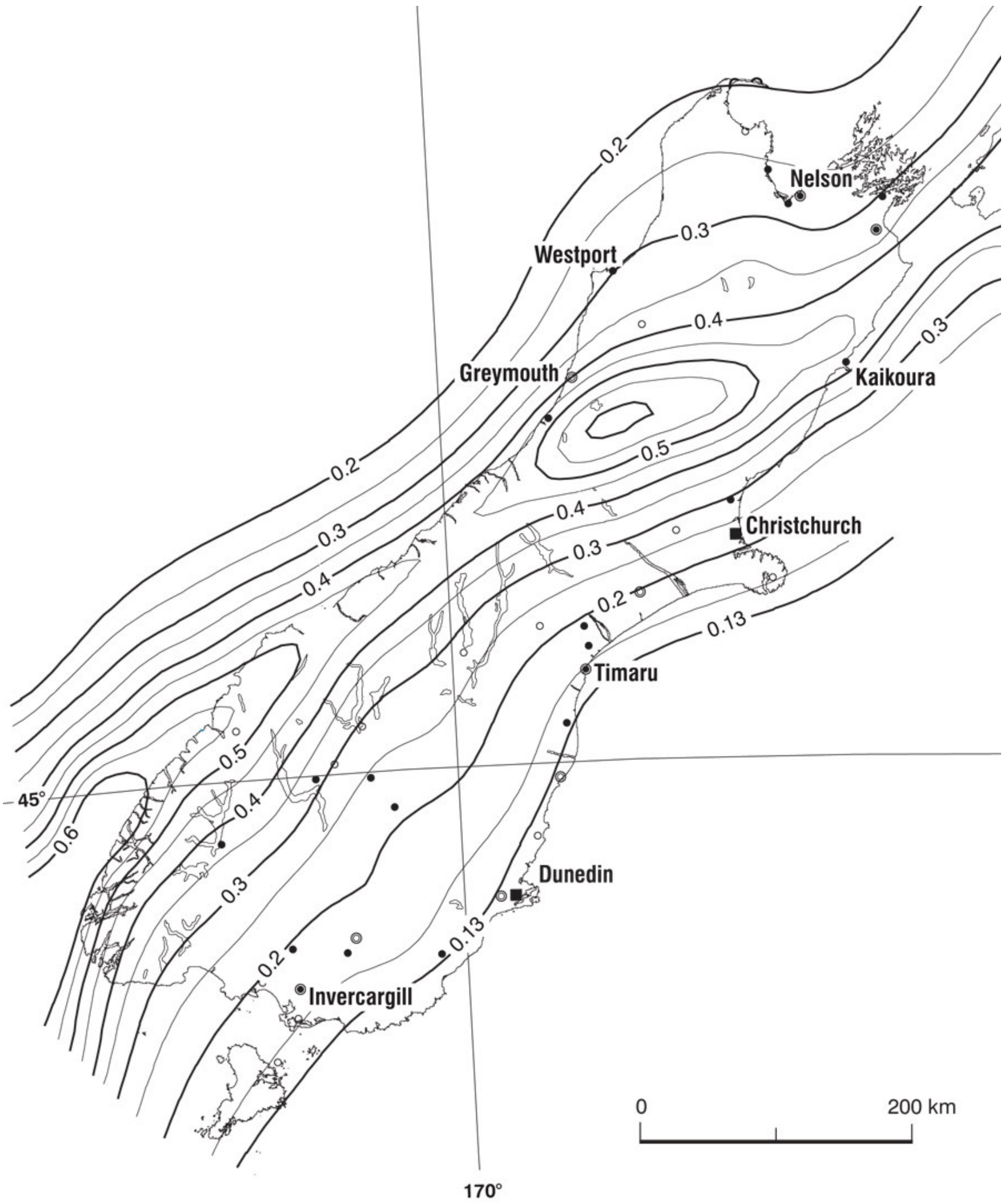
Location	#
St Arnaud	82
Stratford	44
Taihape	56
Takaka	77
Taumarunui	37
Taupo	36
Tauranga	18
Te Anau	122
Te Aroha	17
Te Awamutu	22
Te Kuiti	28
Te Puke	24
Temuka	106
Thames	11
Timaru	107
Tokoroa	26
Turangi	38
Twizel	109
Upper Hutt	74
Waihi	13
Waikanae	67
Waimate	110
Wainuiomata	76
Waiouru	50
Waipawa	54
Waipukurau	55
Wairoa	40
Waitara	41
Waiuku	9
Wanaka	112
Wanganui	53
Ward	89
Warkworth	6
Wellington	72
Wellington CBD (north of Basin Reserve)	71
Westport	83
Whakatane	32
Whangarei	4
Winton	124
Woodville	62

NOTE: Numbers refer to the corresponding entry in Table 3.3.



NOTE: Circles and squares correspond to towns and cities.

FIGURE 3.3 HAZARD FACTOR, *Z*, FOR THE NORTH ISLAND



NOTE: Circles and squares correspond to towns and cities.

FIGURE 3.4 HAZARD FACTOR, *Z*, FOR THE SOUTH ISLAND

3.1.5 Return period factor

The return period factor, R_s for the serviceability limit state or R_u for the ultimate limit state, shall be obtained from Table 3.5 for the return period or probability of occurrence appropriate for the limit state under consideration as prescribed in Table 3.3 of AS/NZS 1170.0.

TABLE 3.5

RETURN PERIOD FACTOR

Required annual probability of exceedance	R_s or R_u
1/2500	1.8
1/2000	1.7
1/1000	1.3
1/500	1.0
1/250	0.75
1/100	0.5
1/50	0.35
1/25	0.25
1/20	0.20

NOTE: Shaded rows correspond to annual probabilities of exceedance related to a 50 year design working life in Table 3.3 of AS/NZS 1170.0.

3.1.6 Near-fault factor

The near-fault factor, $N(T,D)$, shall be determined from Equations 3.1(2) and 3.1(3) for locations at shortest distance, D, of less than 20 km from the nearest major fault listed in Table 3.6. The locations of these faults are shown in Figure 3.5.

3.1.6.1 *Annual probability of exceedance ≥ $^1/_{250}$*

$$N(T,D) = 1.0 \qquad \ldots 3.1(2)$$

3.1.6.2 *Annual probability of exceedance < $^1/_{250}$*

$$\begin{aligned} N(T,D) &= N_{max}(T) && D \leq 2 \text{ km} \\ &= 1 + (N_{max}(T) - 1)\frac{20-D}{18} && 2 \text{ km} < D \leq 20 \text{ km} \\ &= 1.0 && D > 20 \text{ km} \qquad \ldots 3.1(3) \end{aligned}$$

where

D = the shortest distance (in kilometres) from the site to the nearest fault listed in Table 3.6

$N_{max}(T)$ = the maximum near-fault factor and is linearly interpolated for period T from Table 3.7.

TABLE 3.6

MAJOR FAULTS REQUIRING NEAR-FAULT FACTORS > 1.0

Major faults requiring near-fault factors > 1.0
Alpine
Awatere
Clarence
Hope
Kakapo
Kekerengu
Kelly
Mohaka
Wairarapa
Wairau
Wellington

The near-fault factor shall be taken as 1.0 for locations at distances of 20 km or greater from the major faults that are listed in Table 3.6.

Distances, *D,* shall be taken as those given in Table 3.3 where they are listed for a locality. Where Table 3.3 gives a range of distances for a locality, *D* may be taken by default as the shortest of the listed distances or may be determined as the shortest distance from the site to the nearest fault plane listed in Table 3.6 using detailed geological information. For those locations listed in Table 3.3 without a distance *D* specified, the near-fault factor shall be taken as 1.0. For unlisted locations, the shortest distance *D* to the fault plane of any of the faults listed in Table 3.6 shall be determined using detailed geological information to evaluate the near-fault factor.

TABLE 3.7

MAXIMUM NEAR-FAULT FACTORS $N_{max}(T)$

Period T s	$N_{max}(T)$
≤ 1.5	1.0
2	1.12
3	1.36
4	1.60
≥ 5	1.72

3.2 SITE HAZARD SPECTRA FOR VERTICAL LOADING

The elastic site hazard spectrum for vertical loading, $C_v(T)$, for a given return period shall be given by Equation 3.2(1):

$$C_v(T) = 0.7C(T) \qquad \text{. . . 3.2(1)}$$

where

$C(T)$ = elastic site hazard spectrum for horizontal loading determined from Clause 3.1.1 for the modal or time history method of analysis.

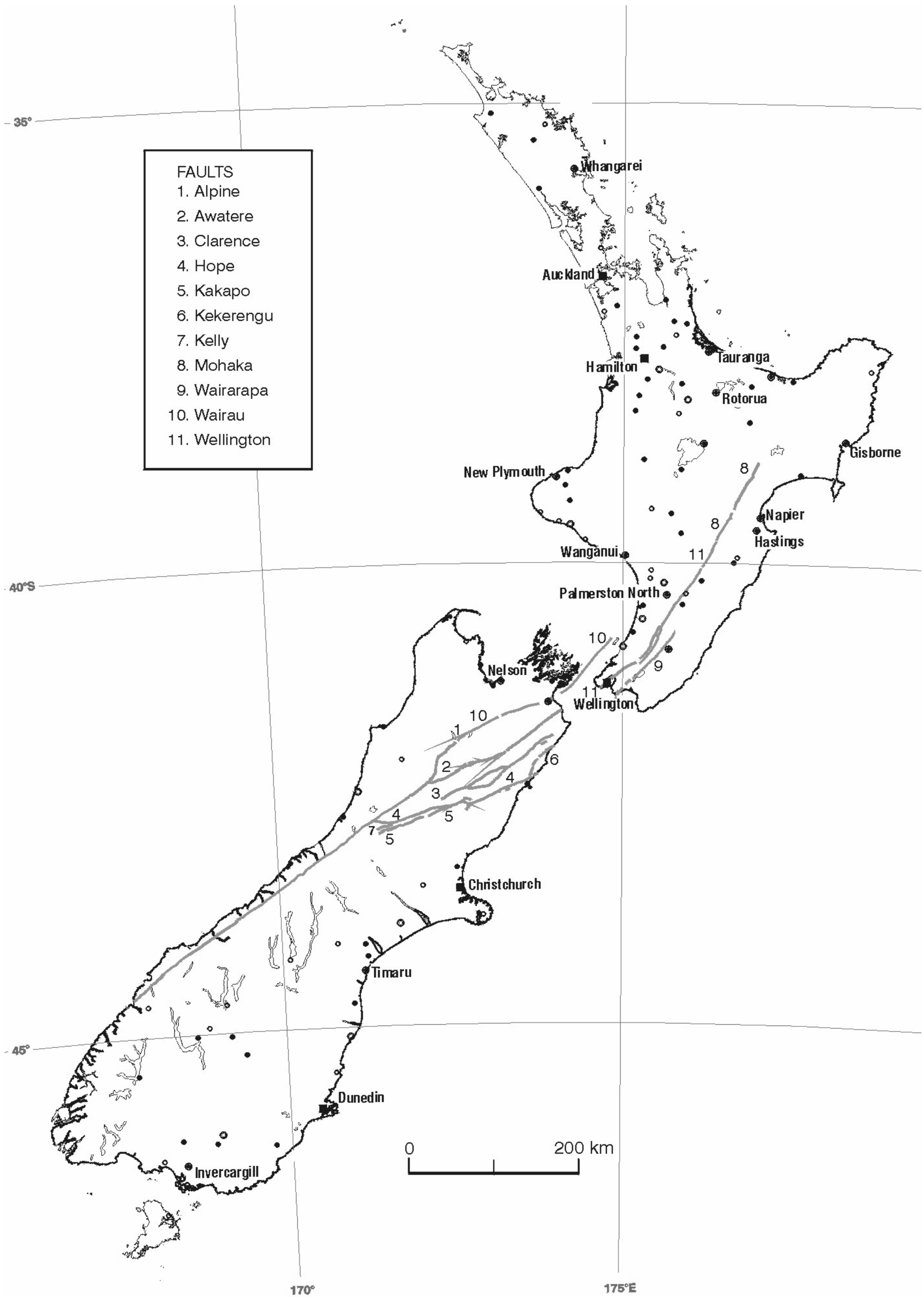


FIGURE 3.5 MAJOR FAULTS REQUIRING NEAR-FAULT FACTORS > 1.0

SECTION 4 STRUCTURAL CHARACTERISTICS

4.1 PERIOD OF VIBRATION

4.1.1 General

The periods of vibration, T_i, shall be established from properly substantiated data or computation, or both, using material and section properties appropriate to the limit state under consideration.

4.1.2 Period determination for the equivalent static method

Where the equivalent static method of analysis is used, the largest translational period in the direction under consideration, T_1, may be calculated from Equation 4.1(1) in Clause 4.1.2.1, or from an equivalent method.

4.1.2.1 *Rayleigh method*

$$T_1 = 2\pi\sqrt{\frac{\sum_{i=1}^{n}\left(W_i d_i^{\,2}\right)}{g\sum_{i=1}^{n}\left(F_i d_i\right)}} \qquad \ldots 4.1(1)$$

where

d_i	=	the horizontal displacement of the centre of mass at level i, ignoring the effects of torsion
F_i	=	the displacing force acting at level i
g	=	acceleration due to gravity
i	=	the level under consideration of structure
n	=	number of levels in a structure
W_i	=	the seismic weight at level i

4.2 SEISMIC WEIGHT AND SEISMIC MASS

The seismic weight at each level shall be given by:

$$W_i = G_i + \Sigma \Psi_E Q_i \qquad \ldots 4.2(1)$$

where

G_i and $\Psi_E Q_i$ are summed between the mid-heights of adjacent storeys

G_i	=	the permanent action (self-weight or 'dead' action) at level i
Ψ_E	=	0.6 is the earthquake imposed action (live load) combination factor for storage applications
Ψ_E	=	0.3 is the earthquake imposed action (live load) combination factor for all other applications
Q_i	=	the imposed action for each occupancy class on level i, (refer AS/NZS 1170.1)

Q_i for roofs shall include an allowance of 1.0 kPa for ice on roofs where required by AS/NZS 1170.3.

The seismic mass at each level, m_i, shall be taken as W_i/g.

4.3 STRUCTURAL DUCTILITY FACTOR

4.3.1 Ultimate limit state

4.3.1.1 *Assignment of the structural ductility factor*

The assignment of the structural ductility factor, μ, shall be chosen to be consistent with the capability of the associated detailing from the appropriate material Standard in accordance with Clause 2.3.2.

4.3.1.2 *Mixed systems*

For mixed systems comprising different structural forms of seismic-resisting systems for a given direction of loading, the design seismic action on each system shall be determined by a rational analysis. This analysis shall take into account the relative stiffness, plastic mechanism development, and material strain capacities in the potential inelastic zones associated with each system.

4.3.2 Serviceability limit state

The structural ductility factor, μ, for the serviceability limit state SLS1 shall be $1.0 \leq \mu \leq 1.25$ and for SLS2 shall be within the limits $1.0. \leq \mu \leq 2.0$.

4.4 STRUCTURAL PERFORMANCE FACTOR, S_p

4.4.1 For stability

When considering lateral stability of a whole structure against sliding or toppling, the structural performance factor, S_p, shall be taken as 1.0.

4.4.2 For ultimate limit state

Unless otherwise defined by the appropriate material Standard, the structural performance factor, S_p, for the ultimate limit state shall be taken as 0.7 except where $1.0 < \mu < 2.0$ then S_p shall be defined by:

$$S_p = 1.3 - 0.3\mu$$

4.4.3 Systems with ductile capabilities but designed as nominally ductile

Structural systems that are designed for $\mu < 2.0$ but which have strain capability greater than that corresponding to μ may, for the ultimate limit state, use the structural performance factor, S_p, appropriate for the provided level of ductility capacity.

4.4.4 For serviceability limit state

The structural performance factor, S_p, for the serviceability limit state shall be taken as 0.7 unless otherwise defined by the appropriate material Standard.

4.5 STRUCTURAL IRREGULARITY

A structure shall be considered as irregular if it has any of the features listed in Clauses 4.5.1 and 4.5.2. One storey penthouses or roofs with a weight less than 10% of the level below shall not be considered when applying these criteria.

4.5.1 Vertical irregularity

4.5.1.1 *Weight (mass) irregularity*

Weight irregularity shall be considered to exist where the weight, W_i, of any storey is more than 150% of the weight of an adjacent storey. A roof that is lighter than the floor below need not be considered.

4.5.1.2 *Vertical stiffness irregularity*

Vertical stiffness irregularity shall be considered to exist when the lateral stiffness of the primary structure in a storey is less than 70% of the stiffness of any adjacent storey, or less than 80% of the average stiffness of the three storeys above or below.

4.5.1.3 *Discontinuity in capacity – weak storey*

A weak storey is one in which the storey shear strength is less than 90% that in the storey above. The storey shear strength is the total strength of all vertical seismic-resisting elements of the primary structure sharing the storey shear for the direction under consideration.

4.5.1.4 *Vertical geometric irregularity*

Vertical geometric irregularity shall be considered to exist where the sum of the horizontal dimensions of the vertical elements of the primary structure in the direction under consideration in any storey is more than 130% of that in an adjacent storey.

4.5.2 Plan irregularity

4.5.2.1 *Horizontal offsets of columns in moment-resisting frames*

Horizontal plan irregularity shall be considered to exist where, in the direction under consideration, in-plane or out-of-plane offsets of columns at any floor level are present where either:

(a) The average of the absolute values of the tangent of the offset angle:

$$\frac{\sum^{N_c}\left|\frac{a_j}{b_j}\right|}{N_c} > 0.1 \qquad \text{. . . 4.5(1)}$$

where

a_j = the horizontal offset at column j

b_j = the vertical distance between the base of the upper column and the top of the lower column j

N_c = the total number of columns at the level under consideration

or:

(b) For any single column j, the tangent of the offset angle:

$$\frac{a_j}{b_j} > 0.4 \qquad \text{. . . 4.5(2)}$$

4.5.2.2 *Out-of-plane offsets of lateral force resisting walls*

Horizontal plan irregularity shall be considered to exist in lateral force resisting walls where out-of-plane offsets occur that conform to Equations 4.5(1) or 4.5(2).

4.5.2.3 *Torsional sensitivity*

Horizontal plan irregularity resulting from torsional sensitivity shall be considered to exist when the ratio γ exceeds 1.4 when calculated as follows.

The ratio γ_i for each level i according to the following equation is determined independently for each orthogonal direction:

$$\gamma_i = \frac{d_{max}}{d_{av}} \qquad \text{. . . 4.5(3)}$$

where

d_{av} = the average of the displacements at the extreme points of the structure at level i produced by the actions above this level

d_{max} = the maximum storey displacement at the extreme points of the structure at level i in the direction of the earthquake induced by the equivalent static actions acting at distances ±0.10 times the plan dimension of the structure, b, from the centres of mass at each floor

γ = the maximum of all values of γ_i in both orthogonal directions

SECTION 5 DESIGN EARTHQUAKE ACTIONS

5.1 GENERAL

5.1.1

The earthquake actions for the ultimate and serviceability limit states shall be evaluated in accordance with Clause 5.2 or Clause 5.5.

5.1.2

The elastic site hazard spectrum, $C(T)$, the near fault factor, $N(T,D)$, the return period factor for the serviceability state, R_s, and the return period factor for the ultimate state, R_u, shall be determined from Section 3. Note the limitation of Clause 3.1.1 that applies in regions of high seismicity.

The structural Performance Factor, S_p, the largest translational period in the direction being considered, T_1 and the ductility μ, shall be determined from Section 4.

5.2 HORIZONTAL DESIGN ACTION COEFFICIENTS AND DESIGN SPECTRA

5.2.1 Equivalent static method – Horizontal design action coefficient

5.2.1.1 *Ultimate limit state*

For the ultimate limit state, the horizontal design action coefficient, $C_d(T_1)$, shall be as given by Equation 5.2(1):

$$C_d(T_1) = \frac{C(T_1)S_p}{k_\mu} \qquad \text{. . . 5.2(1)}$$

$$\geq (Z/20 + 0.02)R_u \text{ but not less than } 0.03R_u \qquad \text{. . . 5.2(2)}$$

where

$C(T_1)$ = the ordinate of the elastic site hazard spectrum determined from Clause 3.1.1

S_p = the structural performance factor determined by Clause 4.4

Z = the hazard factor determined from Clause 3.1.4 taking account of the limitation on the value of ZR_u given by Clause 3.1.1

For soil classes A, B, C and D

$$k_\mu = \mu \qquad \text{for } T_1 \geq 0.7 \text{ s}$$

$$= \frac{(\mu - 1)T_1}{0.7} + 1 \qquad \text{for } T_1 < 0.7 \text{ s}$$

For soil class E

$$k_\mu = \mu \qquad \text{for } T_1 \geq 1\text{s or } \mu < 1.5$$

$$= (\mu - 1.5)T_1 + 1.5 \qquad \text{for } T_1 < 1\text{s and } \mu \geq 1.5$$

provided that for the purposes of calculating k_μ, T_1 shall not be taken less than 0.4 s.

5.2.1.2 *Serviceability limit state*

For the serviceability limit state, the horizontal design action coefficient, $C_d(T_1)$, shall be as given by Equation 5.2(1) where:

$C(T_1)$ = the ordinate of the elastic site spectrum for the largest translational period of vibration determined from Clause 3.1.1 using R_s

T_1 = the largest translational period of vibration

S_p = 0.7

μ = determined in accordance with Clause 4.3.2

5.2.2 Modal response spectrum method

5.2.2.1 *Ultimate limit state design response spectrum*

For the ultimate limit state, the horizontal design response spectrum, $C_d(T)$, shall be as given by Equation 5.2(3):

$$C_d(T) = \frac{C(T)S_p}{k_\mu} \qquad \ldots 5.2(3)$$

where

$C(T)$, k_μ and S_p are defined in Clause 5.2.1.1 using R_u.

5.2.2.2 *Ultimate limit state design – scaling of actions and displacements*

Seismic design actions and displacements shall be scaled by the appropriate factor, k, given in (a) or (b) below:

(a) Structures that are not classified as irregular under Clause 4.5:

(i) Where the base shear is equal to or greater than 80% of the base shear corresponding to the equivalent static analysis:

$k = 1.0$

(ii) Where the base shear is less than 80% of the base shear corresponding to the equivalent static analysis:

$k = 0.8\ V_e / V$

(b) Structures that are classified as irregular under Clause 4.5:

(i) Where the base shear is equal to or greater than 100% of the base corresponding to the equivalent static analysis:

$k = 1.0$

(ii) Where the base shear is less than corresponding to the equivalent static analysis:

$k = V_e / V$

where V_e = the base shear found from the equivalent static method

V = the base shear found from the modal response spectrum method

5.2.2.3 *Serviceability limit state*

For the serviceability limit state, the horizontal design response spectrum, $C_d(T)$, shall be as given by Equation 5.2(4):

$$C_d(T) = C(T)S_p \quad \text{. . . 5.2(4)}$$

where $C(T)$ is defined in Clause 3.1.1 using R_s and $N(T,D)$ as specified in Clause 3.1.6 and $S_p = 0.7$.

5.3 APPLICATION OF DESIGN ACTIONS

5.3.1 Direction of actions

For all structures, different directions of application of the specified actions shall be considered in order to determine the most unfavourable effect in any structural member.

5.3.1.1 *Ductile structures*

For ductile structures including structures of limited ductility:

(a) With seismic-resisting systems located along two perpendicular directions, the specified actions may be assumed to act separately along each of these two horizontal directions; or

(b) With seismic resisting systems not located along two perpendicular directions, the specified actions shall be applied separately in sufficient directions to produce the most unfavourable effect in any structural member.

5.3.1.2 *Nominally ductile and brittle structures*

For nominally ductile and brittle structures an action set comprising 100% of the specified earthquake actions in one direction plus 30% of the specified earthquake actions in an orthogonal direction to this shall be applied as follows:

(a) For seismic-resisting systems located along two perpendicular directions, the action set shall be applied separately in each perpendicular direction (100% on the first axis with 30% on the second axis, and then 30% on the first axis and 100% on the second axis); or

(b) For seismic resisting systems not located along two perpendicular directions the action set shall be applied in sufficient directions so as to produce the most unfavourable effect in any structural member.

5.3.2 Accidental eccentricity

For each required direction of earthquake loading, allowance shall be made for accidental eccentricity of the earthquake actions. The eccentricity shall be applied in the same direction at all levels. The accidental eccentricity shall be measured from the nominal centre of mass and shall be determined as follows:

(a) For actions applied in a direction parallel to the principal orthogonal axes of the structure, the eccentricity shall be taken as not less than ±0.1 times the plan dimension, *b*, of the structure at right angles to the direction of loading.

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

5.4 VERTICAL DESIGN ACTIONS

5.4.1 For the structure as a whole

When required by Clause 6.4.1 the vertical design action coefficient, C_{vd}, shall be as follows:

$$C_{vd} = C_v(T_v)S_p \qquad \text{. . . 5.4(1)}$$

where

$C_v(T_v)$ = elastic site hazard spectrum for vertical loading determined from Clause 3.2 using R_s or R_u as appropriate

T_v = the vertical period of the structure to be taken as 0

S_p = the structural performance factor for the structure as given by Clause 4.4

5.4.2 For elements

When required by Clause 8.4.2, the vertical design action coefficient, C_{vd}, shall be as follows:

$$C_{vd} = C_v(T_v) \qquad \text{. . . 5.4(2)}$$

where

$C_v(T_v)$ = elastic site hazard spectrum for vertical loading determined from Clause 3.2 using R_s or R_u as appropriate

T_v = the vertical period of the element under consideration

5.5 GROUND MOTION RECORDS FOR TIME HISTORY ANALYSES

5.5.1 Selection of acceleration ground motion records

Earthquake ground motion records used for time history analysis shall consist of at least the two horizontal components. The vertical component of the record may also be necessary when considering the response of structures or parts that are sensitive to vertical accelerations such as for horizontal cantilevers or some items of equipment.

The ground motion records shall be selected from actual records that have a seismological signature (i.e. magnitude, source characteristic (including fault mechanism) and source-to-site distance) the same as (or reasonably consistent with) the signature of the events that significantly contributed to the target design spectra of the site over the period range of interest. The ground motion is to have been recorded by an instrument located at a site, the soil conditions of which are the same as (or reasonably consistent with) the soil conditions at the site.

When the site is near a major fault, i.e. when $N(T,D) > 1.0$ in Clause 3.1.6, then one record in three in each family selected shall have a forward directivity component, while the remainder of the family shall be of near-neutral or backwards directivity. A record has strong-forward directivity when half or more of the rupture propagation is towards the site, i.e. the epicentre is at a distance of half the rupture length or greater from the site.

The ground motion records used for time history analysis shall consist of a family of not less than three records. Where three appropriate ground motion records are not available, simulated ground motion records may be used to make up the family.

Each record shall be scaled by a record scale factor, k_1, so as to match the target spectra over the period range of interest. The target spectra is the design spectrum appropriate for the site and limit state of the structure under consideration. Each record within the family of records is then to be scaled by the family scale factor, k_2 which is applied to ensure that the energy content of at least one record in the family exceeds that of the design spectrum over the target period range.

As most structures do not have the same fundamental period in different directions, the period range of interest will differ in different directions. It should be expected that both the record scale factor, k_1 and the family scale factor, k_2 will be different for different directions. It is therefore also likely that the principal component (defined in Clause 5.5.2(c)) of any record may also switch according to the period of the structure.

5.5.2 Scaling ground motion records

The record scale factor, k_1 shall be determined as follows:

(a) Compute the target spectrum, SA_{target}, for the site given by:

$$SA_{\text{target}} = \left(\frac{1+S_{\text{p}}}{2}\right)C(T) \qquad \ldots 5.5(1)$$

where

$C(T)$ = elastic site hazard spectrum

S_{p} = structural performance factor given by Clause 4.4.2

(b) Calculate the 5% damped spectrum, $SA_{\text{component}}$, of each component of each ground motion record within the family of records being considered.

(c) Determine the principal component, $SA_{\text{principal}}$, and associated record scale factor for each direction in which the records are to be applied by:

(i) Determining the structure orientation relative to the direction selected.

(ii) Determine the largest translational period, T_1, of the response mode in the direction of interest.

(iii) Calculate the period range of interest, T_{range}, as being between $T_{\min}$ and $T_{\max}$ where $T_{\min} = 0.4\, T_1$ and $T_{\max} = 1.3T_1$ and where T_1 is the largest translational period in the direction being considered but not less than 0.4 sec.

(iv) Select records that have a seismological signature (i.e. magnitude, source characteristic (including fault mechanism) and source-to-site distance) the same as (or reasonably consistent with) the signature of the site.

(v) Determine the record scale factor, k_1, for each of the horizontal ground motion components where k_1 = scale value which minimizes in a least mean square sense the function $\log(k_1 SA_{\text{component}}/SA_{\text{target}})$ over the period range of interest.

In each case the periods used to determine k_1 are to be selected so that each period is within 10% of the preceding one, except that an increment not greater than 1 second may be used for periods greater than 5 seconds.

(vi) Verify that the amplitude of the selected record is sufficiently similar by confirming that $0.33 < k_1 < 3.0$. Reject records that do not satisfy this criteria.

(vii) Verify that the record selected is of reasonable fit to the target spectra. This can be demonstrated by it satisfying the requirement that D_1, being the root mean square difference between the logs of the scaled primary component and the target spectra over the period range of interest, is less than log (1.5). Reject records that are not of reasonable fit.

(viii) Nominate the principal component as being the record component with the smaller k_1 value and assign this value of k_1 as the record scale factor for this target period T. The other component of the record is considered to be the secondary component.

(d) Determine the record family scale factor, k_2, which is required to ensure that for every period in the period range of interest, the principal component of at least one record spectrum scaled by its record scale factor k_1, exceeds the target spectrum.

The record family scale factor k_2, is the maximum value of the ratio $SA_{\text{target}}/max(SA_{\text{principal}})$ but at least 1.0 over the period range of interest for the direction under consideration and $max(SA_{\text{principal}})$ is the maximum principal component of each record within the family at each period considered.

(e) If k_2 is in the range 1.0 to 1.3, then the principal and secondary components selected may be confirmed.

If $k_2 > 1.3$ then:

(i) Continue using the principal components as selected; or

(ii) Select a different record as one of the family so as to better cover the target spectrum and reassess k_2; or

(iii) If the record scale factors of the components are within 20% of each other at period T, swap the principal and secondary component and reassess k_2.

(f) Confirm the principal and secondary components of each record as being those selected from (e) above or amend this selection by consideration of the scaled secondary components so as to minimize the product of k_1k_2.

(g) Repeat the steps in (c) to (f) above for other directions of interest noting that in each case the orthogonal direction to each initial selection will need to be considered.

5.6 CAPACITY DESIGN

5.6.1 General

Capacity design shall be applied to structures of limited ductility, ductile structures and to other structures where required by the appropriate material Standard as set out in Clauses 5.6.2, 5.6.3 and Appendix C, together with additional requirements on capacity design as set out in the appropriate material Standard.

5.6.2 Structures of limited ductility

For structures of limited ductility that satisfy all of the following:

(a) They are not classified as irregular under Clause 4.5;

(b) A height which is less than 15 m;

(c) Any additional requirements in the appropriate material Standard;

the nominal capacity design requirements for the design of structures of limited ductility in the appropriate material Standard may be used in lieu of the requirements in Clause 5.6.3. Structures that do not satisfy these requirements shall be designed to satisfy the requirements of Clause 5.6.3.

5.6.3 Capacity design requirements for ductile structures

5.6.3.1 *Potential inelastic zones*

Ductile failure modes for the proposed structure shall be identified for each potential direction of seismic actions. The location of all potential inelastic zones shall be identified and proportioned so that the design strength exceeds the design actions at these locations. The form of the potential inelastic zone, whether unidirectional or reversing, shall be identified.

5.6.3.2 *Deformation of potential inelastic zones*

Determine the level of detailing required to sustain the material strain levels in the critical potential inelastic zones when the displacements defined in Clause 7.2 are applied to the structure. Detail the potential inelastic zones to be capable of sustaining these deformations as specified in the appropriate material Standard.

In unidirectional inelastic zones, allow for the amplification of material strain levels beyond that found for reversing inelastic zones.

5.6.3.3 *Overstrength actions in potential inelastic zones*

Determine the maximum likely strength of each potential inelastic zone, as designed and detailed, as specified in the appropriate material Standard. Where an appropriate material Standard does not exist, use the upper characteristic strengths and allow for strain hardening levels appropriate to the level of material strain required in the potential inelastic zones.

5.6.3.4 *Design outside inelastic zones*

Where members sustain actions due to overstrength moments and shears from two axes the bi-axial effects from both axes shall be considered. The full dynamic magnification coefficient shall be assumed to act on one axis while on the other axis the dynamic magnification factor shall be not less than 1.0.

In certain cases, material Standards may specify a nominal distribution of actions in a member in place of dynamic magnification factors.

Design actions shall, where required, be either:

(a) Amplified by dynamic magnification factors; or

(b) Distributed into the element, as specified in the appropriate material Standard,

to prevent premature failure modes from developing due to higher mode effects.

SECTION 6 STRUCTURAL ANALYSIS

6.1 GENERAL

6.1.1 Methods of analysis

A structural analysis to determine the action effects shall be carried out in accordance with one of the following providing the limitations of Clause 6.1.3 are complied with:

(a) A method based on equivalent static forces as outlined in Clause 6.2; or

(b) The modal response spectrum method as outlined in Clause 6.3;

(c) The numerical time history method as outlined Clause 6.4.

6.1.2 P-delta analysis

P-delta effects shall be considered in accordance with Clause 6.5 in analyses of design actions and deflections for the ultimate limit state.

6.1.3 Limitations on the use of methods of analysis

6.1.3.1 *Equivalent static method*

The equivalent static method of analysis shall be used only when at least one of the following criteria is satisfied:

(a) The height between the base and the top of the structure is less than 10 m; or

(b) The largest translational period calculated as specified in Clause 4.1.2 is less than 0.4 s; or

(c) The structure is not classified as irregular under Clause 4.5 and the largest translational period is less than 2.0 seconds.

6.1.3.2 *Modal response spectrum method*

The modal response spectrum method may be used on all structures that fall within the scope of this Standard provided that three-dimensional analyses shall be used when the structure is classified as torsionally sensitive under Clause 4.5.2.3.

6.1.3.3 *Numerical integration time history analyses*

Numerical integration time history analyses may be used on all structures that fall within the scope of this Standard to verify that specific response parameters are within the limits of acceptability assumed during design. Three-dimensional time history analyses shall be used where the structure is classified as torsionally sensitive under Clause 4.5.2.3.

6.1.4 Diaphragm response

6.1.4.1 *Requirement for modelling*

For structures over 15 m in height where the structure is classified as irregular under provisions of Clause 4.5, diaphragms shall be modelled in a three-dimensional modal response spectrum or three-dimensional numerical integration time history analysis. Where diaphragms are not rigid compared to the vertical elements of the vertical action resisting system, the model should include representation of the diaphragm's flexibility.

Elastic diaphragms shall be used in structures and modelled as such.

Inelastic deformations associated with in-plane diaphragm actions resulting from earthquake induced forces shall only be permitted to occur when justified by rational analysis which has been substantiated by experimental data.

6.1.4.2 *Actions for design of diaphragms*

Actions to be used in the design of these diaphragms shall be the sum of the actions determined from the two-dimensional or three-dimensional analysis, plus the actions derived from considering how the inertia of the diaphragm elements are distributed to the horizontal load resisting members of the structure. Actions within the diaphragms shall account for higher mode effects and the influence of overstrength actions in accordance with Clause 5.6.3.3, generated from within the structure as a whole.

6.2 EQUIVALENT STATIC METHOD

6.2.1 Equivalent static forces

6.2.1.1 *General*

The set of equivalent static forces in the direction being considered that are specified in this Clause shall be assumed to act simultaneously at each level of the structure.

6.2.1.2 *Horizontal seismic shear*

The horizontal seismic shear, V, acting at the base of the structure in the direction being considered shall be calculated from:

$$V = C_d(T_1)W_t \qquad \text{. . . 6.2(1)}$$

where

$C_d(T_1)$ = the horizontal design action coefficient as given in Clause 5.2.1.1 for the ultimate limit state and Clause 5.2.1.2 for the serviceability limit state

W_t = the seismic weight of the structure defined in Clause 4.2

6.2.1.3 *Equivalent static horizontal force at each level*

The equivalent static horizontal force, F_i at each level, i, shall be obtained from Equation 6.2(2)

$$F_i = F_t + 0.92V \frac{W_i h_i}{\sum_{i=1}^{n}(W_i h_i)} \qquad \text{. . . 6.2(2)}$$

where $F_t = 0.08V$ at the top level and zero elsewhere.

6.2.2 Points of application of equivalent static forces

The equivalent static design forces shall be applied through points eccentric to the nominal centre of mass at each level as specified in Clause 5.3.2.

6.2.3 Scaling of deflections

The magnitudes of the deflections, including the inter-storey deflections, may be reduced by multiplying by the deflection scale factor, k_d given below:

(a) For determining the period of the structure, by the method set out in Clause 4.1.2.1, the scale deflection factor used for the lateral displacements, d_i, shall be taken as 1.0.

(b) For structures that have a weak storey or a flexible storey, as defined in Clause 4.5, the scale deflection factor shall be taken as 1.0.

(c) For other structures the deflection scale factor is given by Table 6.1.

TABLE 6.1

DEFLECTION SCALE FACTORS

No. of storeys	Deflection scale factor, k_d
1	1.0
2	0.97
3	0.94
4	0.91
5	0.88
6 or more	0.85

6.3 MODAL RESPONSE SPECTRUM METHOD

6.3.1 General

A dynamic analysis of a structure by the response spectrum method shall use the peak response of the modes specified in Clause 6.3.3. Peak modal responses shall be calculated using the ordinates of the appropriate response spectrum given in Clause 6.3.2. The design action effects shall be the maximum modal action effects combined in accordance with Clause 6.3.4.

6.3.2 Design response spectrum

The design response spectrum used for the modal response spectrum method shall be calculated in accordance with Clause 5.2.2.1 for the ultimate limit state and Clause 5.2.2.3 for the serviceability limit state.

6.3.3 Number of modes

6.3.3.1 *Two-dimensional analyses*

Sufficient modes shall be included in the analysis to ensure that at least 90% of the total mass of the structure is participating in the direction under consideration.

6.3.3.2 *Three-dimensional analyses*

Sufficient modes shall be included in the analysis to ensure that at least 90% of the total mass of the structure is participating in each of two orthogonal directions.

In structures that are modelled so that modes are considered that are not those of the horizontal load resisting systems, then all modes not part of the horizontal load resisting systems shall be ignored.

6.3.4 Combination of modal action effects

6.3.4.1 *Two-dimensional analyses*

For two-dimensional analyses, the combination of modal action effects (e.g. storey shear, moment, drift, displacements and inter-storey displacements) shall be carried out by either taking the square root of the sum of the squares (SRSS) of the contribution from each mode, or the complete quadratic combination (CQC) technique or any other generally accepted combination method.

6.3.4.2 *Three-dimensional analyses*

For three-dimensional analyses the combination of modal action effects shall be carried out using the complete quadratic combination (CQC) technique or any other generally accepted combination method.

6.3.4.3 *Closely spaced modes*

If the SRSS combination method is used, the modal action effects from any modes with frequencies within 15% shall first be combined by direct summation ignoring any signs.

6.3.5 Torsion

6.3.5.1 *General*

Where a structure is not classified as plan irregular under the provisions of Clause 4.5.2 and a two-dimensional modal response spectrum analysis is used for translational effects, an analysis for torsional effects may be conducted by the static method of Clause 6.3.5.2.

In all other cases torsional effects shall be included in a three-dimensional analysis method using the provisions of Clause 6.3.5.3.

6.3.5.2 *Static analysis for torsional effects*

For a static analysis for torsional effects, the applied torsion at each level shall use either the actions calculated by the equivalent static method or the combined storey earthquake actions found in a two-dimensional modal response spectrum analysis for translation. The eccentricity used shall be as required in Clause 5.3.2. Torsional effects shall be combined with the translational effects by direct summation, with signs chosen to produce the most adverse combined effects in the resisting members.

6.3.5.3 *Three-dimensional analyses*

(a) Except as provided in (b) below, for each direction of loading the position and distribution of the mass shall be adjusted to account for the eccentricity specified in Clause 5.3.2. The sign of the eccentricity shall be that producing the largest design actions in the resisting members.

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

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

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

6.4 NUMERICAL INTEGRATION TIME HISTORY METHOD

6.4.1 General

Where the numerical integration time history method is used, the structure shall be subjected to a family of not less than three ground motion records, each of which has been scaled to match the design level earthquake for the limit state and location of the structure.

Where only horizontal excitation effects are to be assessed, then the two horizontal components of appropriately scaled ground-motion record shall be applied to the structure simultaneously in orthogonal directions. Where the response parameter under consideration is significantly influenced by vertical excitation, then the vertical component shall also be applied in addition to, but simultaneously with, the two horizontal components.

6.4.2 Structural modelling

The sectional properties assigned to members for time history analyses shall be those that result in the most adverse response of the parameter under consideration with the constraint that they remain within a range appropriate for the limit state under consideration. Unless otherwise justified, section parameters (i.e. the strength, post-elastic degradation models, and member stiffness which are to be applied and the viscous damping assigned) are to be determined by reference to the appropriate material Standards. Variations that result in a difference in response of less than 10% are to be considered insignificant and need not be the subject of further investigation.

6.4.3 Application of time history analyses

The response parameter of interest shall be assessed by subjecting the structure to ground motion represented by each member of the family of ground motion records scaled in accordance with Clause 5.5.2.

Application of each ground motion record shall be as follows:

(a) Each component of the ground motion record is to be scaled by the record scale factor, k_1, and the family scale factor, k_2, applied in the time domain (i.e. with the record ordinate being multiplied by the product k_1k_2 the values of k_1 and k_2 being as determined by Clause 5.5.2)

where

k_1 = the record scale factor for the principal component of the record at period T_1

k_2 = the family scale factor

T_1 = the largest translational period of the structure in the direction of application of the principal component of the ground motion record

(b) The principal and secondary components, scaled as indicated above, will generally be applied together and at orthogonal directions one to the other.

The direction of application of the principal component of the ground motion record is to be such as to produce the most adverse response of the parameter under consideration.

6.4.4 Direction of application

At least two analyses of the structure shall be carried out for each earthquake. In the first of these, the principal component of the earthquake, (scaled by k_1k_2) must be directed along the direction of translation of the first translational mode together with the secondary component of the same record also scaled by k_1k_2. In the second analysis the direction of the principal component of the earthquake is to be applied in a direction orthogonal to the first analysis. Both the principal component and the scaling factors will usually be different for each analysis because T_1 is different for both situations.

Other analyses having different directions of application of the appropriately scaled earthquake components shall also be considered in order to determine the most unfavourable effect in any structural member.

6.4.5 Analysis time step

The analysis time step:

(a) Shall not be greater than the step at which the records are digitised.

(b) Shall be less than or equal to –

(i) $T_1/100$

(ii) T_n and

(iii) 0.01 s

where

T_1 = is the largest translational period of the first mode (judged by largest mass contribution) in the direction of principal component of the earthquake, and

T_n = is the period of the highest mode in the same direction required to achieve the 90% mass as described in the modal response spectrum method.

(c) Should be sufficiently small to ensure convergence to an accurate solution.

6.4.6 Viscous damping

Viscous damping of 5% for all modes whose period is less than the analysis time step included in the analysis is to be used unless a different value is recommended by the appropriate material Standard. If Rayleigh damping is used, there shall be no more than 5% of critical damping in the two first translational modes, and no more than 40% damping in the mode with period T_n.

6.4.7 Assessment of response parameter

The most critical value of any response parameter (e.g. stress, strain, rotation, displacement) across the family of records shall be used to determine acceptability.

6.4.8 Determination of design horizontal deflections

Calculation of design horizontal deflections for the serviceability limit state shall take into account any departures from linear elastic behaviour.

The design horizontal deflections shall be taken as the maxima of the appropriate deflections obtained for each of the required ground motions.

6.4.9 Determination of inter-storey deflection

The design inter-storey deflection between adjacent levels shall be taken as the maximum of the inter-storey deflections obtained for each of the required ground motions.

6.4.10 Design values

The strength requirements of the potentially inelastic members may be taken as the maximum values obtained from elastic time history analyses, using earthquake records scaled in accordance with Clause 5.5.2 to match the elastic response spectra divided by k_μ, but shall not be taken as less than necessary to satisfy the requirements of the serviceability and ultimate limit state deformation and displacement limits. The value to use for k_μ is that specified in Clause 5.2.1.1.

Inelastic demands placed on the members and capacity actions shall be obtained from inelastic time history analyses, in which the inelastic properties of the members are modelled, using earthquake records scaled in accordance with Clause 5.5.2 and the P-delta actions shall be included in the analysis. The inter-storey displacements shall not exceed the limits given in Clause 7.5.1 and inelastic deformation demands shall not exceed the limits given in the appropriate material Standard.

6.5 P-DELTA EFFECTS

6.5.1 General

When required by Clause 6.5.2, P-delta effects shall be included in the analysis of action effects and deflections in accordance with Clause 6.5.4.

P-delta actions shall be assessed as specified in Clauses 6.5.2 to 6.5.4.2 where either the equivalent static or the modal response spectrum analysis methods are used. When the numerical integration time history method of analysis is used, P-delta effects shall be incorporated into the analysis for the ultimate limit state.

6.5.2 Ultimate limit state

An analysis for P-delta effects for the ultimate limit state is not required where any one of the following applies:

(a) The largest translational period is less than 0.40 seconds; or

(b) The height of the structure measured from the base is less than 15 m and the largest translational period is less than 0.6 seconds; or

(c) The maximum value of the stability coefficient, θ, found for any storey in a structure as given by Equation 6.5(1) is less than 0.1:

$$\theta = \frac{W_i \, \delta_{ui}}{V_i \left(h_i - h_{i-1}\right)} \qquad \ldots 6.5(1)$$

where

h_i = the height of level i above the base of the structure

$(h_i - h_{i-1})$ = the inter-storey height for storey i

δ_{ui} = the inter-storey displacement for storey i for the ultimate limit state as specified in Clause 7.3.1

V_i = the storey shear strength V_i may be conservatively taken as the design storey seismic shear force; and

W_i = the seismic weight resisted by the storey i being considered

For structures of two or more storeys, where capacity design is used to specifically exclude column sway mechanisms, and vertical stiffness irregularities and weak storeys as defined in Clause 4.5 do not exist, Equation 6.5(1) shall be applied to all storeys between the base and mid-height. For all other structures the Equation shall be applied to all storeys.

When the stability coefficient calculated from Equation 6.5(1) exceeds 0.3 the configuration of the structure is not acceptable.

6.5.3 Serviceability limit state

Assessment of P-delta effects is not required for the serviceability limit state.

6.5.4 Analysis for P-delta effects

Where the equivalent static or modal response spectrum analysis method is used, and an analysis for P-delta actions is required, it shall be made using either method A given in Clause 6.5.4.1 or method B given in Clause 6.5.4.2.

6.5.4.1 *Approximate method for P-delta effects – method A*

The structural action effects from Clauses 6.2 or 6.3 shall be multiplied by the ratio:

$$\frac{k_p W_t + V}{V} \qquad \ldots 6.5(2)$$

where

k_p is given by the expression

$$k_p = (0.015 + 0.0075(\mu - 1)) \qquad \ldots 6.5(3)$$

with the limits of $0.015 < k_p < 0.03$

V = the seismic base shear

W_t = the seismic weight of the structure

6.5.4.2 *P-delta analysis – method B*

Step 1

Analyse the structure using either the equivalent static method as defined in Clauses 5.2.1 and 6.2, or the modal response spectrum method as defined in Clauses 5.2.2 and 6.3, neglecting P-delta effects. From the results of the analysis find the maximum of the horizontal displacements of the centre of seismic weight at each floor level in the direction being considered.

Where the diaphragms are stiff a single ratio of the seismic weight may be assumed for each level. With flexible diaphragms the seismic weight shall be distributed to two or more centres for each level in such a way that the resultant centre of seismic weight is maintained.

Step 2

Scale the horizontal displacements of the centres of seismic weight found in step 1 as required in Clause 7.2.1.1 to give the predicted horizontal displacements allowing for inelastic deformation.

Step 3

By assuming that the seismic weight at each level is concentrated at its centre, calculate the actions induced by these weights being displaced through the displacements found in step 2. Find the "additional displacements" due to these actions. (A simple method of carrying out this analysis is given in the commentary.)

Step 4

Calculate the value of β, which makes an allowance for the ductility and incorporates the factor K, which makes an allowance for the period and the foundation subsoil type on P-delta actions, as follows:

$$\beta = {}^{2\mu K}\!/_{3.5} \text{ for } \mu \le 3.5 \qquad \ldots\ 6.5(4)$$

$$= 2.0\ K \text{ for } \mu > 3.5 \qquad \ldots\ 6.5(5)$$

For site subsoil classes A, B and C;

$$K = 1.0 \text{ for } T_1 < 2.0 \qquad \ldots\ 6.5(6)$$

$$= \frac{(6.0 - T_1)}{4} \text{ for } 2.0 \le T_1 \le 4.0 \qquad \ldots\ 6.5(7)$$

$$= 0.5 \text{ for } T > 4 \qquad \ldots\ 6.5(8)$$

For site subsoil classes D and E;

$$K = 1.0 \text{ for } T_1 < 2.5 \qquad \ldots\ 6.5(9)$$

$$= \frac{(6.5 - T_1)}{4} \text{ for } 2.5 \le T_1 \le 4.5 \qquad \ldots\ 6.5(10)$$

$$= 0.5 \text{ for } T_1 > 4.5 \qquad \ldots\ 6.5(11)$$

In no case shall the value of β be taken as less than 1.0.

Step 5

Multiply the additional structural action effects found in step 3 by β and add these to the corresponding actions found in the equivalent static or modal response spectrum analysis method carried out for step 1. The resultant actions are the design actions.

Step 6

Multiply the further lateral displacements found in step 3 by β and add these to the corresponding values found by the equivalent static or modal response spectrum method of analysis. The resultant displacements are scaled as required in Clause 7.2.1.1 to give the resultant deflection profile for the structure including P-delta actions.

6.6 ROCKING STRUCTURES AND STRUCTURAL ELEMENTS

Where energy dissipation is through rocking of structures or structural sub-assemblies (which include, but are not restricted to, walls, frames and foundations), the actions on the structures and parts being supported by the structures shall be determined by special study. Such studies are outside the scope of this Standard (see Clause 1.4).

SECTION 7 EARTHQUAKE INDUCED DEFLECTIONS

7.1 GENERAL

7.1.1 Calculation of deflections

Design horizontal deflections at each level of the structure shall be calculated in accordance with Clause 7.2 using one of the methods of analysis given in Section 6 and shall not exceed the limits specified in Clause 7.4.

7.1.2 Actions considered for calculating deflections

Calculation of design horizontal deflections shall include the effects of both translation and torsion, and the effect of foundation deformations. P-delta effects shall be considered where required by Clause 6.5.2.

7.1.3 Determination of inter-storey deflections

Design inter-storey deflection shall be calculated in accordance with Clause 7.3 for the ultimate limit state and shall not exceed the appropriate limits specified in Clause 7.5.

7.1.4 Properties to be used when determining deflections

For each limit state, deflections shall be calculated from the action determined from Section 5 based on the stiffness properties of components or members, as designed and detailed.

7.2 DETERMINATION OF DESIGN HORIZONTAL DEFLECTIONS

7.2.1 Ultimate limit state

7.2.1.1 *Using equivalent static or modal response spectrum methods*

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

(a) Elastic deflections found from either the equivalent static method, Clause 6.2.3, or from the modal response spectrum method, Clause 6.3.4, multiplied by a scale factor equal to the structural ductility factor, μ.

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

  (i) The elastic deflection profile shall be determined using either the equivalent static method or the modal response spectrum.

  (ii) The sidesway mechanism deflection profiles shall be constructed by considering all potential sidesway mechanisms except those that are specifically suppressed through the application of capacity design procedures. The deflection profile for each sidesway mechanism shall be consistent with obtaining a deflection at the level of the uppermost principal seismic weight equal to the corresponding value found in (a), for this level.

7.2.1.2 *Increase in displacements due to P-delta actions*

Where method A given in Clause 6.5.4.1 is used to allow for P-delta actions, the deflections found from Clause 7.2.1.1 shall be multiplied by the factor given by Equation 6.5(2).

Where method B given in Clause 6.5.4.2 is used to allow for P-delta actions, the resultant deflections, which include the P-delta effects, are given by Clause 6.5.4.2.

7.2.1.3 *Using numerical integration time history analysis*

The ultimate limit state horizontal deflections shall be taken as the maxima of the appropriate deflections obtained for each of the required ground motions.

7.2.2 Serviceability limit state

Calculation of design horizontal deflections for the serviceability limit state shall be based on the linear elastic response of each member, unless some additional but limited inelastic displacement is considered acceptable and is nominated as such within the appropriate material Standard. If so, account shall be taken of the inelastic displacement in the calculation.

7.3 DETERMINATION OF DESIGN INTER-STOREY DEFLECTION

7.3.1 Ultimate limit state

7.3.1.1 *Equivalent static method and modal response spectrum method*

Where horizontal deflections have been derived in accordance with Clause 7.2.1.1, the design inter-storey deflection between adjacent levels shall be the maximum value found from the deflection profile multiplied by the drift modification factor, k_{dm}, determined from Table 7.1.

When computing P-Delta effects, as required by Clause 6.5, the inter-storey deflection between adjacent levels shall be the unmodified maximum value found from the deflection profile.

TABLE 7.1

DRIFT MODIFICATION FACTOR

Structure height	Drift modification factor, k_{dm}
$h < 15$ m	1.2
$15 \leq h \leq 30$ m	$1.2 + 0.02(h - 15)$
$h > 30$ m	1.5

7.3.1.2 *Numerical integration time history method*

Where the horizontal deflections have been computed in accordance with Clause 7.2.1.3, incorporating inelastic member response, the design inter-storey deflection between levels shall be taken as the maximum inter-storey deflection obtained for each required ground motion record that do not include forward directivity and 0.67 of that maximum for records that do include forward directivity motions.

7.3.2 Serviceability limit state

Design inter-storey deflection for the serviceability limit state shall be taken as the difference in the design horizontal deflections between adjacent levels calculated in accordance with Clause 7.2.2 or those determined from the combined modal inter-storey deflections.

7.4 HORIZONTAL DEFLECTION LIMITS

7.4.1 Ultimate limit state

7.4.1.1 *Adjacent to boundaries*

The design horizontal deflection of any point on the perimeter of a structure shall not exceed the distance from that point on the structure to the boundaries of adjacent sites, except for street frontages.

7.4.1.2 *Adjacent to structures on the same site, or existing structures on adjacent sites*

At any point above the ground, the design horizontal deflection of the structure shall be such that, when combined with the design horizontal deflection of any adjacent structure at the same height, contact does not occur.

7.5 INTER-STOREY DEFLECTION LIMITS

7.5.1 Ultimate limit state

The ultimate limit state inter-storey deflection determined in accordance with Clause 7.3.1 shall not exceed 2.5% of the corresponding storey height or such lesser limit as may be prescribed in the appropriate material Standard.

7.5.2 Serviceability limit state

For the serviceability limit state, the inter-storey deflection shall be limited so as not to adversely affect the required performance of other structure components in accordance with Clause 2.1.4(b). The design horizontal deflections shall not be greater than any separation provided to avoid contact between adjacent parts of the structure, or between the structure and its parts and shall be limited so as not to impair their function nor that of other structure components.

SECTION 8 REQUIREMENTS FOR PARTS AND COMPONENTS

8.1 GENERAL

8.1.1 Scope

Where required by Section 2, all parts of structures, including permanent, non-structural components and their connections, and permanent services and equipment supported by structures, shall be designed for the earthquake actions specified in this Section.

For category P.1, P.2 and P.3 parts the scope is limited to parts that weigh more than 10 kg and are able to fall more than 3 m onto a publicly accessible area.

Where the mass of the part is in excess of 20% of the combined mass of the part and the primary structure and its lowest translational period is greater than 0.2 seconds, a special study shall be carried out to determine the dynamic characteristics of the part.

8.1.2 Classification of parts

Parts shall be classified into the categories shown in Table 8.1.

TABLE 8.1

CLASSIFICATION OF PARTS

Category	Criteria	Part risk factor R_p	Structure limit state[1]
P.1	Part representing a hazard to life outside the structure [2]	1.0	ULS
P.2	Part representing a hazard to a crowd of greater than 100 people within the structure [2]	1.0	ULS
P.3	Part representing a hazard to individual life within the structure [2]	0.9	ULS
P.4	Part necessary for the continuing function of the evacuation and life safety systems within the structure	1.0	ULS
P.5	Part required for operational continuity of the structure [3]	1.0	SLS2
P.6	Part for which the consequential damage caused by its failure are disproportionately great	2.0	SLS1
P.7	All other parts	1.0	SLS1

NOTES:

1 Refer to Section 2 for the return period of exceedance appropriate for this limit state.

2 To be considered in this category, the part must weigh more than 10 kg, and be able to fall more than 3 metres onto a publicly accessible area.

3 Only parts essential to the operational continuity of structures with importance level 4 will be classified as P.5. Non-essential parts and parts within structures of other importance levels will be otherwise classified.

8.2 DESIGN RESPONSE COEFFICIENT FOR PARTS

When the part is supported directly on the ground floor it shall be designed as a separate structure with design actions derived in accordance with Section 5 using the structural characteristics determined in Section 4.

In cases when the part is supported at level i of a structure, the design response coefficient for parts, $C_p(T_p)$ is the horizontal acceleration coefficient derived for the level of structure that provides support for the part. It shall be determined from Equation 8.2(1):

$$C_p(T_p) = C(0)\, C_{Hi}\, C_i(T_p) \quad \ldots 8.2(1)$$

where

$C(0)$ = the site hazard coefficient for $T = 0$ determined from Clause 3.1, using the values for the modal response spectrum method and numerical integration time history methods

C_{Hi} = the floor height coefficient for level i, determined from Clause 8.3

T_p = the period of the part

$C_i(T_p)$ = the part spectral shape factor at level i, determined from Clause 8.4

8.3 FLOOR HEIGHT COEFFICIENT , C_{Hi}

The floor acceleration coefficient at level i, C_{Hi}, shall be calculated from Equations 8.3 (1), 8.3(2) or 8.3(3) as appropriate for the elevation of the support height of the part. For elevations that satisfy the height limitations of both Equations 8.3(1) and 8.3(2) the lesser value of C_{Hi} shall be used.

$$C_{Hi} = \left(1 + \frac{h_i}{6}\right) \quad \text{for all } h_i < 12 \text{ m} \quad \ldots 8.3(1)$$

$$C_{Hi} = \left(1 + 10\frac{h_i}{h_n}\right) \quad \text{for } h_i < 0.2h_n \quad \ldots 8.3(2)$$

$$C_{Hi} = 3.0 \quad \text{for } h_i \geq 0.2h_n \quad \ldots 8.3(3)$$

where

h_i = height of the attachment of the part

h_n = height from the base of the structure to the uppermost seismic weight or mass.

C_{Hi} for levels below ground floor level shall be taken as the same as at ground floor level.

8.4 PART SPECTRAL SHAPE COEFFICIENT

The part spectral shape coefficient, $C_i(T_p)$, is the ordinate of a tri-linear function depicting the shape of the horizontal acceleration of the part with the period of that part, T_p. The ordinates of the part spectral shape factor are given in Equations 8.4(1), 8.4(2) and 8.4(3).

$$C_i(T_p) = 2.0 \quad \text{for } T_p \leq 0.75 \text{ s} \quad \ldots 8.4(1)$$

$$= 0.5 \quad \text{for } T_p \geq 1.5 \text{ s} \quad \ldots 8.4(2)$$

$$= 2(1.75 - T_p) \quad \text{for } 0.75 < T_p < 1.5 \text{ s} \quad \ldots 8.4(3)$$

8.5 DESIGN ACTIONS ON PARTS

8.5.1 Horizontal design actions

The horizontal design earthquake actions on a part, F_{ph}, shall be determined from Equation 8.5(1):

$$F_{ph} = C_p(T_p)\, C_{ph}\, R_p\, W_p \leq 3.6\, W_p \qquad \ldots 8.5(1)$$

where

$C_p(T_p)$ = the horizontal design coefficient of the part, determined from Clause 8.2

C_{ph} = the part horizontal response factor determined from Clause 8.6

R_p = the part risk factor as given by Table 8.1

W_p = the weight of the part

8.5.2 Vertical design actions

Parts that are sensitive to vertical acceleration amplification shall be designed for vertical earthquake actions. Unless determined by a special study, the vertical earthquake actions on a part, F_{pv}, shall be calculated using Equation 8.5(2):

$$F_{pv} = C_{pv}\, C_{vd}\, R_p\, W_p \leq 2.5\, W_p \qquad \ldots 8.5(2)$$

where

C_{pv} = parts vertical response factor determined from Clause 8.6

C_{vd} = the vertical design action coefficient determined from Clause 5.4 for the period of the system supporting the part

R_p = the part risk factor as given by Table 8.1

W_p = the weight of the part

8.5.3 Deflection induced actions

Where the part is connected to the primary structure on more than one level, the part shall be designed to sustain the actions resulting from the relative deflections that occur for the limit state being considered.

These deflections shall be calculated in accordance with Clause 7.3 at the limit state being considered.

8.6 PART RESPONSE FACTOR C_{ph}

The part horizontal factor, C_{ph}, shall be as provided in Table 8.2 with the ductility of the part $\mu_p = 1.0$ unless the level of floor acceleration is such as to bring about yielding of the part.

The part vertical response factor, C_{pv}, shall be determined according to Table 8.2 with $\mu_p = 1.0$ unless otherwise determined by special study.

For serviceability limit states $\mu_p = 1.0$.

TABLE 8.2

PART RESPONSE FACTOR, C_{ph} and C_{pv}

Ductility of the part μ_p	C_{ph} and C_{pv}
1.0	1.0
1.25	0.85
2.0	0.55
3.0 or greater	0.45

8.7 CONNECTIONS

8.7.1

Non-ductile connections for parts shall be designed for seismic actions corresponding to a ductility factor of the part of $\mu_p = 1.25$. Non-ductile connections include, but are not limited to, expansion anchors, shallow chemical anchors or shallow (non-ductile) cast-in-place anchors in tension and not engaged with the main reinforcement.

8.7.2

Other connections may be designed for a greater value of μ_p where the specific detailing can be verified to sustain not less than 90% of their design action effects at a displacement greater than twice their yield displacement under reversed cyclic loading.

APPENDIX A

DEFINITIONS

(Normative)

For the purpose of this Standard the definitions below apply:

Appropriate material Standard

A limit state format material Standard or Standards for the particular materials under consideration that has been developed for use with this Standard and, where required (i.e. μ is taken greater than 1.00) incorporates detailing provisions to achieve the required inelastic strain demands under earthquake actions, and also procedures for carrying out capacity design. Such Standards shall comply with Appendix D.

Base

The level at which earthquake motions are considered to be imparted to the structure, or the level at which the structure as a dynamic vibrator is supported, or the level at which primary ground coupling takes place.

Brittle structure

See structural systems.

Building part

A member that is either attached to, and supported by, the structure but is not part of the structural system or an element of the structural system that can be loaded by an earthquake in a direction not usually considered in the design of that element.

Capacity design

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

Capacity design principles

Appropriate material Standard design and detailing provisions that enable zones where post-elastic response is acceptable to be identified and detailed in a manner that ensures these zones are capable of accepting the inelastic demands placed upon them. All other zones are designed to ensure that all other undesirable inelastic response mechanisms are suppressed and detailed in a manner that the ultimate limit state horizontal deformations that they are expected to be subjected to, can be sustained without significant (e.g. > 20%) loss of load carrying capacity after four complete cycles of loading.

Connection

A mechanical means that provides a load path for actions, between structural members, between non-structural elements and between structural and non-structural elements.

Designer

The suitably qualified person who is responsible for the adequacy of all those aspects of the design that affect structural performance.

Design spectrum

A spectrum used for analysis and design of a structure.

Diaphragm

A horizontal or near horizontal system that acts to transmit horizontal actions to the vertical elements of the lateral action resisting elements.

Flexible diaphragm

A diaphragm that is sufficiently flexible that the maximum lateral deformation is more than twice the average inter-storey deflection at that level.

Rigid diaphragm

A diaphragm that is sufficiently rigid that the maximum lateral deflection is less than twice the average inter-storey deflection at that level.

Ductile structure

See structural systems.

Ductility

Member ductility

The ability of a member to maintain a capacity to carry certain loads, while exhibiting plastic deformations and dissipating energy when it is subjected to cyclic inelastic displacements during an earthquake.

Structural ductility

The ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake, accounting for development of plastic mechanisms and the ability of structural members to accommodate the consequential demands for plasticity.

See also Ductile structure

Earthquake actions

Inertia-induced actions arising from the structure response to earthquake.

Element

A physically distinguishable assembly of members that act together in resisting lateral actions, such as moment resisting frames, structural (shear) walls, and diaphragms.

Flexible diaphragm

See diaphragm

Inter-storey height

The overall height between two successive structural levels of a structure.

Limit state

Serviceability limit state

This condition is reached when the structure undergoes damage that limits its intended use through deformation, vibratory response, degradation or other physical aspects under an earthquake of low intensity.

Ultimate limit state

This condition is reached when the structure loses structural integrity, becomes unstable or loses equilibrium under the design seismic action but does still retain a small residual load bearing capacity that prevents local or global collapse.

Limited ductility provisions

Appropriate material Standard detailing provisions that will result in the development of potential inelastic zones within members where post-elastic behaviour is expected and other zones or members where the development of inelastic behaviour is suppressed. Potential inelastic zones shall be capable of sustaining the level of inelastic demand placed upon them when subjected to earthquake ground motions consistent with the structural ductility nominated. Zones where inelastic behaviour is suppressed are to be designed to a higher level of action effect so as to ensure suppression of that behaviour. These zones are to be detailed in a manner such that the ultimate limit state horizontal deformations are expected to be subjected to can be sustained without significant (e.g. > 20%) loss of vertical load carrying capacity.

Member

A physically distinguishable part of a structure, such as a wall, beam, column, slab, connection.

Nominally ductile structure

See structural systems.

P-delta effect

Refers to the structural actions induced as a consequence of the gravity loads being displaced horizontally due to horizontal actions.

Part

An element that is not intended to participate in the overall resistance of the structure to horizontal displacement under earthquake conditions, for the direction being considered.

Partition

A permanent or relocatable internal dividing wall between floor spaces.

Primary structure

The structural system provided to carry the inertial action effects generated in the structure by earthquake actions to the ground.

Principal axis

One of two orthogonal directions of consideration at least one of which is aligned parallel to the principal horizontal action resisting system of a structure.

Rigid diaphragm

See diaphragm

Secondary elements

Elements of the secondary structural system that are not part of the main energy dissipating structure.

Secondary members

Members that are not considered to be part of the earthquake resisting system and whose strength and stiffness against seismic actions is neglected. They are not required to comply with all the requirements of NZS 1170.5, but are designed and detailed to maintain support of gravity loads when subjected to the displacements caused by the seismic design condition.

Secondary structure

The structural system provided to carry actions other than the earthquake actions generated in the structure.

Special study

A procedure for justifying departure from some or all of the requirements of this Standard.

NOTE: Special studies are outside the scope of this Standard.

Storey

The part of a structural system between the logical consecutive horizontal divisions.

Strength

Design strength

The nominal strength multiplied by the strength reduction factor as given in the appropriate material Standard.

Nominal strength

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

Overstrength

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

Probable strength

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

Structural ductility factor

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

Structural performance factor

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

Structural systems

Brittle structure

A structure or its structural components that are not capable of inelastic deformation.

Ductile structure

A structure designed and detailed in accordance with this Standard and the appropriate material Standard so that a structural ductility factor that is greater than 1.25 and less than or equal to 6.0 is appropriate in assessing the ultimate limit state seismic actions.

Nominally ductile structure

A structure designed and detailed in accordance with this Standard and the appropriate material Standard so that a structural ductility factor that is greater than 1.0 and less than or equal to 1.25 is appropriate in assessing the ultimate limit state seismic actions.

Top (of a structure)

The level of the uppermost principal seismic weight.

Weak storey

A weak storey is one that has a shear strength of less than 90% that of the storey above.

APPENDIX B
NOTATION
(Normative)

Unless stated otherwise, the notation used in this Standard shall have the following meanings.

$C(0)$	=	the site hazard coefficient for T = 0 determined from Clause 3.1, using the values for the modal response spectrum method and numerical integration time history methods
$C(T)$	=	elastic site hazard spectrum for horizontal loading
$C(T_1)$	=	the ordinate of the elastic site spectrum for the lowest translational period of vibration
$C_d(T)$	=	horizontal design response spectrum
$C_d(T_1)$	=	horizontal design action coefficient
$C_h(T)$	=	the spectral shape factor
C_{Hi}	=	floor acceleration coefficient at level i
$C_h(0)$	=	values referred to in Table 3.1
$C_i(T)$	=	the floor spectral shape factor
$C_p(T)$	=	the elastic site design spectrum determined in Clause 3.1
$C_p(T_p)$	=	spectral ordinate of the part elastic site hazard spectra for the period of the part T_p
C_{ph}	=	the part horizontal response factor
C_{pv}	=	the part vertical response factor
$C_v(T)$	=	elastic site hazard spectrum for vertical loading
$C_v(T_v)$	=	elastic site hazard spectrum for vertical loading determined from Clause 3.2 using R_s or R_u as appropriate
C_{vd}	=	the vertical design action coefficient for the period of the system supporting the part
D	=	the shortest distance (in kilometres) from the site to the nearest fault, km
D_1	=	the root mean square difference between the logs of the scaled primary component and the target spectra over the period range of interest
d_i	=	the horizontal displacement of the centre of mass at level i, m
d_{av}	=	is the average of the displacements at the extreme points of the structure at level i produced by the actions above this level
d_{max}	=	is the maximum storey displacement at the extreme points of the structure at level i in the direction of the earthquake induced by the equivalent static forces acting at distances ±0.10 times the plan dimension of the structure from the centres of mass at each floor
E_s	=	earthquake actions for serviceability limit state, N, kN m
E_u	=	earthquake actions for ultimate limit state, N, kN m
$f_{A,i}$, $f_{B,i}$	=	the distance between points of inflection, m

F_i	=	the displacing force at level *i,* N
F_{ph}	=	the horizontal design earthquake actions on a part
F_{pv}	=	the vertical earthquake actions on a part
F_t	=	component of equivalent static horizontal force at top level, N
g	=	acceleration due to gravity (usually taken as 9.81 m/s^2)
G_i	=	the permanent action (self-weight or 'dead' action) at level *i,* N, N/m or kPa
h_i	=	the height of level *i* above the base of the structure, mm
$(h_i - h_{i-1})$	=	the inter-storey height for storey *i*
h_n	=	height from the base of the structure to the uppermost seismic weight or mass, mm
h'	=	the height between the centre of the plastic hinge at the base of the wall and the top of the wall
i	=	the level of the structure under consideration
K	=	factor used in determining P-delta effects that makes an allowance for the period and the foundation subsoil type
k	=	seismic design actions and displacements
k_1	=	record scale factor
k_2	=	family scale factor
k_3	=	inter-storey deflection scale factor
k_d	=	deflection scale factor
k_p	=	factor used in determining P-delta effects
k_s	=	storey shear strength factor for the column sway mode
k_μ	=	the inelastic spectrum scaling factor
L	=	span of beams between column centrelines, mm
L'	=	distance between the centres of the plastic hinge zones
m_i	=	seismic mass at each level, N
$max(SA_{principal})$	=	the maximum principal component of each record within the family at each period considered
M_a, M_b	=	the bending moments sustained at the critical sections of the beam, kN m
$M_{A,i}$, $M_{B,i}$	=	the overstrength moments applied to the columns through the plastic hinge zones in the beams, kN m
n	=	number of levels in a structure
$N_{max}(T)$	=	the maximum near-fault factor that is linearly interpolated for period T from Table 3.7
$N(T,D)$	=	the near-fault factor
$N_{max}(T,D)$	=	the maximum near-fault factor
Q_i	=	the imposed action for each occupancy class on level *i,* N, N/m or kPa
R	=	the return period factor, R_s or R_u, for the appropriate limit state
R_p	=	the part risk factor

R_s	=	return period factor for the serviceability limit state
R_u	=	return period factor for the ultimate limit state
$SA_{component}$	=	spectral acceleration of each component of the 5% damped spectrum of the ground motion record
$SA_{principal}$	=	the principal component of the spectral acceleration of the ground motion record
SA_{target}	=	the spectral acceleration of the target 5% damped design spectrum
S_p	=	the structural performance factor
T_1	=	largest translational period of the structure in the direction being considered, s
T_i	=	the period of the i_{th} mode, s
T_n	=	the period of the highest mode in the direction being considered, s
T_p	=	the vertical period of the element or part under consideration, s
T_{min}	=	the minimum period of interest, s
T_{max}	=	the maximum period of interest, s
T_{range}	=	the period range of interest, s
T_v	=	the vertical period of the structure, s
u_i	=	the horizontal deflection of the centre of mass at level *i,* m
V	=	horizontal seismic base shear found from the modal response method, N
V_c	=	column shear, N
$V_{c,i}$	=	is the column shear sustained when plastic hinges form in the column and sustain their overstrength moments, N
V_g	=	the vertical loading shear, N
V_e	=	the base shear found from the equivalent static method, N
V_i	=	the storey shear strength, N
W_i	=	the seismic weight at level *i,* N
W_p	=	the weight of the part, N
W_t	=	the seismic weight of the structure, N
Z	=	the hazard factor
α	=	the plastic hinge rotation
β	=	factor used in determining P-delta effects that makes an allowance for the ductility demand
δ	=	the inter-storey drift, m
δ_e	=	elastic deformation, m
δ_p	=	plastic deformation, m
δ_t	=	the lateral displacement at the top of a wall, m
δ_{ui}	=	the inter-storey displacement for storey *i* for the ultimate limit state as specified in Clause 7.3.1, m
θ_p	=	plastic hinge rotation
γ	=	the maximum of all values of γ_i in both orthogonal directions

γ_i	=	torsional sensitivity = d_{max}/d_{av}
μ	=	structural ductility factor
μ_p	=	ductility of the part
θ	=	stability coefficient
Ψ_E	=	the earthquake imposed action combination factor for storage and other applications

APPENDIX C
DESIGNING FOR ULTIMATE LIMIT STATE
(Informative)

C1 INTRODUCTION

This Appendix gives guidance on:

1 Designing for higher mode effects in frames and structural walls,

2 The different forms of plastic hinge that may form,

3 Assessing material strains in plastic hinge zones,

4 Stiffness values for use in seismic analysis.

C2 DESIGNING FOR HIGHER MODE EFFECTS

C2.1 Background

This Section gives guidance on the "design strengths" required with capacity design for structural members and regions of structural members that are outside potential inelastic zones (plastic hinges).

With the formation of inelastic zones in structures their dynamic characteristics change in ways that cannot be predicted from elastic based analyses such as equivalent static, modal response spectrum or elastic time history methods. Structural actions, that arise due to this change in structural behaviour, are referred to as "higher mode effects". The resultant bending moments and shears may be distributed in patterns that are very different from those found in the elastic based methods of analysis. Due to the dependence of higher mode actions on the dynamic behaviour these values cannot be predicted from push over analyses, even though inelastic behaviour is modelled. The extent to which higher mode effects, or actions arise, increases with the ductility and the number of potential modes or storeys. Hence recommendations to cover these effects may be expected to vary with the structural ductility factor and the number of levels or fundamental period of the buildings.

Numerous time history analyses have been made to assess how actions, which have been obtained from elastic based methods of analysis, should be modified to allow for higher mode effects. The recommendations have been based on two different approaches for finding strengths necessary to cover structural actions associated with "higher mode effects". In the first approach, which has traditionally been used for the design of reinforced concrete columns, the analytical moments are scaled to correspond to overstrength values in the beams, and then these values are multiplied by a dynamic magnification factor. In the second approach, which has been used for structural concrete walls, a design action such as bending moment at the base of a wall, is scaled to correspond to its overstrength value. This value in then applied to a standard distribution to define the design bending moments, or other structural action, over the height of the wall. For structures that are not pure moment resisting frames or structural walls some mix of these two approaches may be appropriate.

Material characteristics can have a significant influence on the value of the dynamic magnification factor that is used. For example plastic hinging in reinforced concrete columns in ductile moment resisting frames is undesirable for two reasons.

1 Plastic hinging has an adverse influence on the performance of laps in reinforcing bars, and hence it is important to keep laps away from these regions. Where potential plastic hinge zones may form in columns, laps or splices are confined to the mid-height regions of storeys. This involves additional complications in construction.

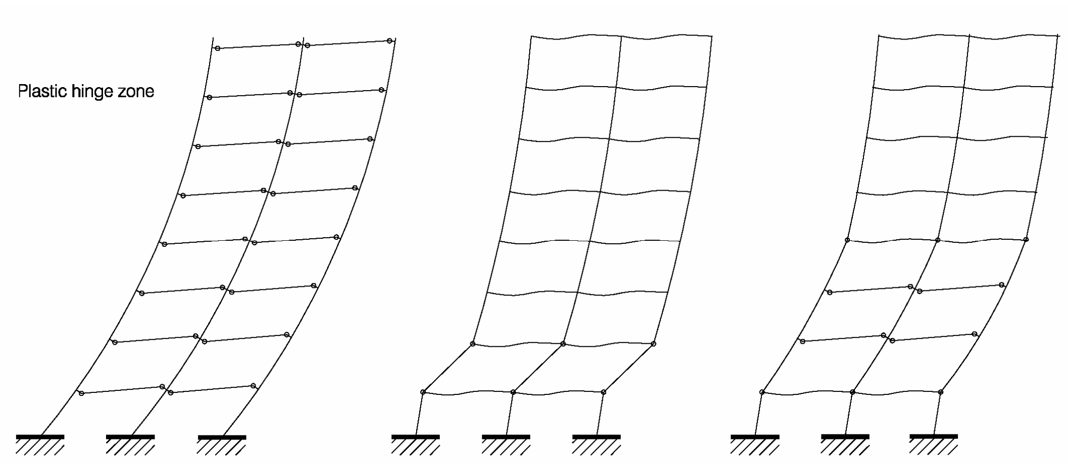
2 Where potential plastic hinging may form in columns extensive confinement reinforcement is required to prevent premature failure. In addition spalling cover concrete from high columns can impose a hazard to life.

For these two reasons structural concrete columns are often designed to have a high level of protection against yielding above the base level. However, structural steel columns used in a similar structural form can sustain some yielding and still perform adequately, provided that there is sufficient strength to prevent the premature formation of a column sway mechanism.

C2.2 Moment resisting frames

C2.2.1 *Background*

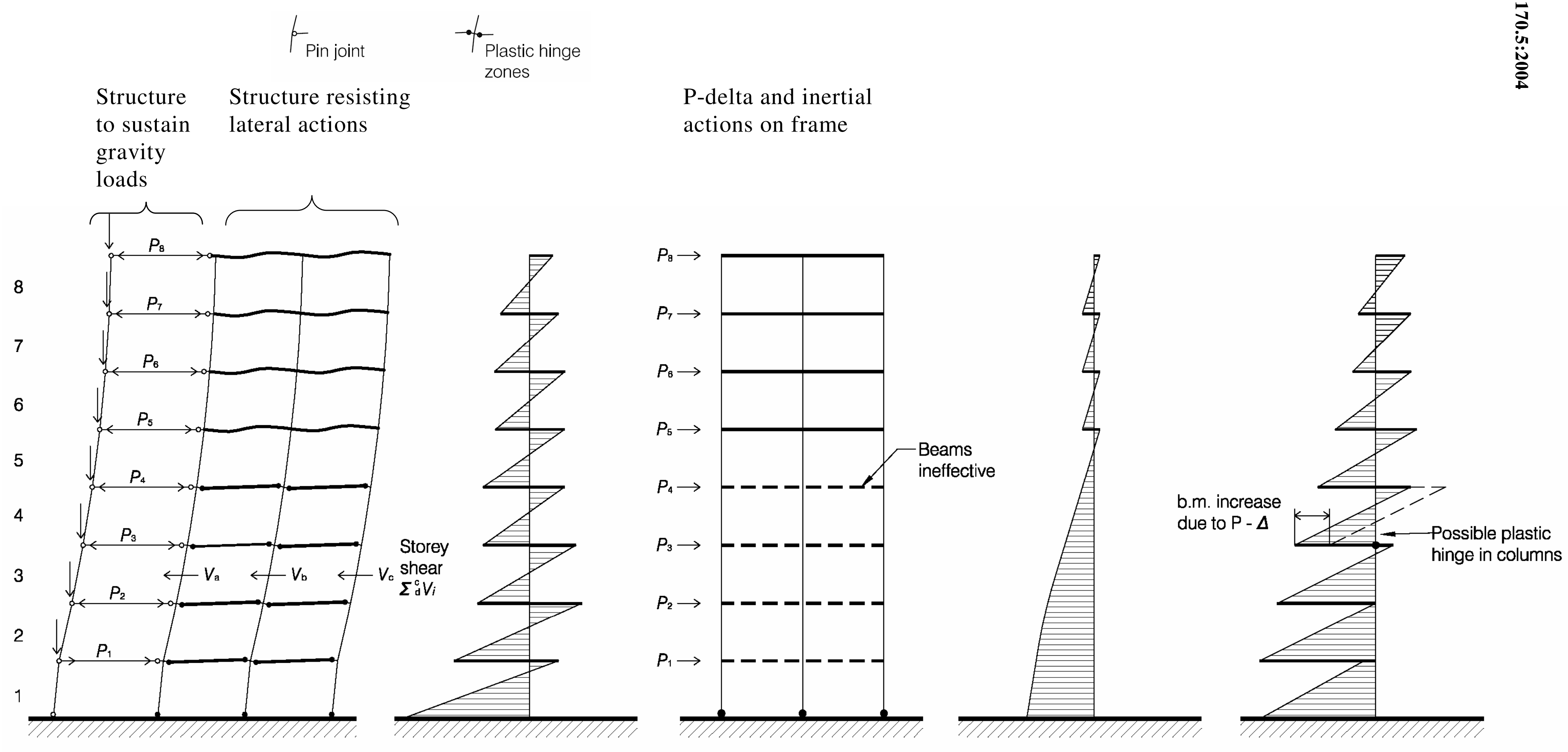
The different sway modes that may develop in moment resisting frame structures are illustrated in Figure C1. Generally the column sway mode has limited ductility and structures deforming in this way are very susceptible to collapse due to P-delta actions. To avoid this potential non-ductile failure mechanism, or the premature formation of a mixed beam-column sway mode, the columns require strength in excess of that indicated by elastic analysis. This may be achieved by ensuring that the storey shear strength corresponding to a column sway mode is greater than the corresponding strength in a beam sway mode. It should be noted that this is a minimum requirement to ensure that the structure will behave in a ductile manner in a major earthquake. In addition to this, with some construction materials there will be additional requirements, as were indicated for reinforced concrete in the previous section.



(a) Beam sway mode (b) Column sway mode (c) Mixed beam and column sway mode

FIGURE C1 SWAY MODES FOR MOMENT RESISTING FRAMES

The actions arising in a multi-storey ductile moment resisting frame are illustrated in Figure C2. During a major earthquake plastic hinges tend to form in the beams in several levels simultaneously, as illustrated in the figure. Over the time interval that these have formed, the only resistance to additional lateral displacement due to P-delta actions and inertia actions on the structure arises from the elastic response of the columns so long as they remain elastic. These columns act as springs that tend to reduce the residual displacement of the frame after each phase when the beam plastic hinges have formed. Without this spring action the displacement could progressively increase and P-delta actions could accumulate during the earthquake, endangering collapse of the structure if the earthquake motion is of long duration.



(a) Basic structure

(b) Bending moments in a column due to moments sustained by beams

(c) Effective structure to resist P-delta & inertia actions when beam plastic hinges form in levels 1 – 4

(d) Bending moments induced in a column by P-delta and internal actions when beam plastic hinges form levels 1 to 4

(e) Resultant column bending moments

FIGURE C2 AMPLIFICATION OF COLUMN BENDING MOMENTS IN DUCTILE FRAMES DUE TO P-DELTA HINGES

Figure C2 (a) shows a portion of a multi-storey frame at a stage where plastic hinges have formed in the beams over several levels, in this case on levels 1, 2, 3 and 4. The actions arising from P-delta actions can be visualised by assuming the gravity loads act on a line of columns that are pinned at each level. This is a valid way of assessing P-delta actions due to seismic response in any structure (see commentary C6.5.4). With the lateral displacement of the gravity load resisting columns lateral actions are transmitted to the frame due to the P-delta actions. Figure C2(b) shows the moments induced in a column by the beams. However, as the frame is displaced laterally P-delta actions act on the frame and induce shear in the columns. Added to this are further small inertia actions associated with the motion of the floors. These lateral actions induce shear in the columns over several storeys and hence additional bending moments are induced, as illustrated in Figures C2(c) and C2(d). The resultant bending moments in the columns are shown in Figure C2(e). These bending moments can increase until a mixed beam column sway mode develops. This involves the formation of plastic hinges in columns separated by several storeys, as illustrated in Figure C1(c). This action can lead to premature failure unless the column strengths are appreciably greater than those indicated in an elastic based analysis.

C2.2.2 *Recommendations for columns in multi-storey ductile moment resisting frames*

In the previous Clause it was noted that structures should be proportioned to avoid the formation of a column sway mode in a major earthquake. It is suggested that this objective can be achieved by designing columns in ductile moment resisting frames to satisfy two criteria. The first of these is to ensure that the design storey shear strength based on the column sway mechanism is equal to, or exceeds 1.2 times the design storey shear. The second criterion involves designing the structure so that the storey shear strength in a column sway mode is greater than the corresponding shear strength in a beam sway mode. Provided the margin between these storey shear strengths is sufficient premature formation of a mixed beam column sway mode should be prevented. On this basis, the recommendation is that ductile moment resisting frame structures should be proportioned so that the nominal storey shear strength based on a column sway mode is equal to or greater than k_s times the corresponding overstrength storey shear strength based on the beam sway mode. A difficulty arises in assessing the storey strength based on the beam sway mode. This might be based on a push over analysis or on an analysis in which the positions of points of inflection are assumed. Allowance also needs to be made for the influence of bi-axial actions in any column that is part of two frames in different planes. Suggested values for k_s are given in Table C1. In this table the first axis refers to the axis containing the moment resisting frame, while the second axis refers to the case where the column also forms part of a second frame. In this case bi-axial actions are induced, with the over-strength moments being amplified by the appropriate k_s, values from each axis.

TABLE C1

VALUES OF k_s

Type of column	k_s First axis	k_s Second axis
Top storey	1.1	1.0
All other storeys	1.3	1.0

Figure C3 illustrates how this approach can be applied to ductile moment resisting frames in which the potential plastic hinge zones, except those at the base of the columns, are designed to form in the beams. As indicated in Figure C3(b) the nominal column sway storey shear strength is found by the shear that can be resisted in each of the columns in the storey. The individual column shears are found from the nominal flexural strengths of the column being considered. The nominal bending moment strengths are used to identify the position of the points of inflection in the columns. To assess the beam sway shear strength the beams are assumed to sustain their overstrengths. With these strengths either of the following methods may be used.

1 A push over analysis is made with the loading pattern being based on either equivalent static actions or actions corresponding to the first translational mode in the direction being considered. In this case the storey beam sway shear strength is obtained direct from the analysis.

2 Points of inflection are assumed to develop in each column at the same level as was assumed for the column sway shear strength (see Figure C3(b)). The storey shear strength is taken as equal to the sum of the beam overstrength moments that act at the column centre-lines divided by the distance between the points of inflection in the columns in the storeys adjacent to the level being considered, see Figure C3(b) (dimension f_2). It should be noted that this is an approximate value, hence the higher coefficient that is used with this approach compared with the push over method.

This approach can be adapted to deal with the case where the design solution assumes that potential plastic hinges can occur in some of the columns instead of beams. This situation is illustrated in Figure C4, where it is assumed that the column C can form plastic hinges in the upper and lower levels of the storey. The nominal storey shear strength based in the column sway mode is found as before. To assess the beam sway storey shear strength the overstrength shear that can be sustained from column C is found. To this is added the shear corresponding to the sum of the beam bending moments applied to the columns divided by the distance between the points of inflection in adjacent storeys. This distance is found from the nominal column bending moments as illustrated in Figure C3(b). With reference to the illustration the beam mode storey shear strength is given by:

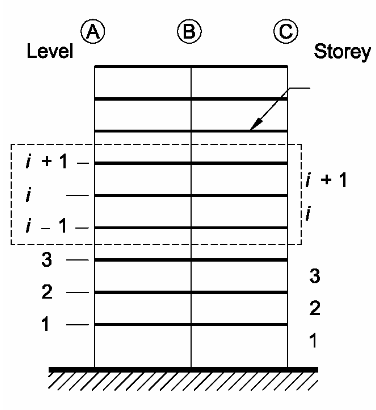
$$\frac{M_{\mathrm{A,i}}}{f_{\mathrm{A.i}}} + \frac{\mathrm{M}_{\mathrm{B,i}}}{f_{\mathrm{B,i}}} + V_{\mathrm{c,i}} \qquad \text{. . . C2.2(1)}$$

where $f_{\mathrm{A,i}}$ and $f_{\mathrm{B,i}}$ are the distance between points of inflection, $M_{\mathrm{A,i}}$ and $M_{\mathrm{B,i}}$ are the overstrength moments applied to the columns through the plastic hinge zones in the beams and $V_{\mathrm{c,i}}$ is the column shear sustained when plastic hinges form in the column and sustain their overstrength moments.

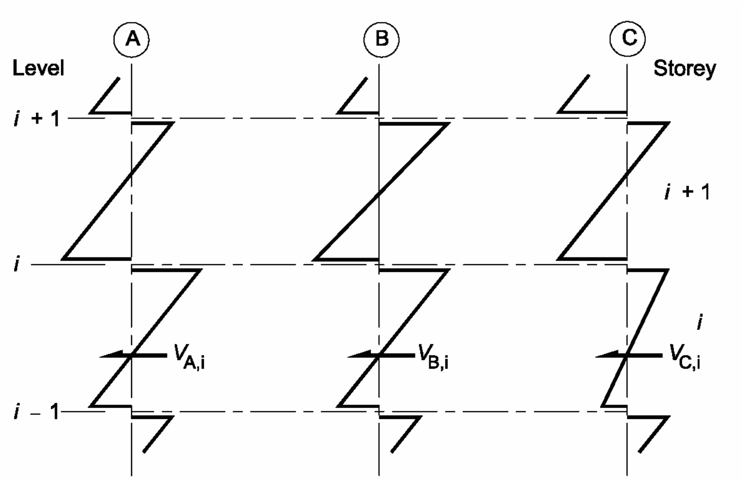
C2.3 Cantilever structural walls

The ductile failure mechanism for cantilever walls is generally based on the formation of a plastic hinge at its base. Above this level the wall is intended to remain essentially elastic, though in practice a limited amount of flexural yielding is acceptable. The formation of the plastic hinge at the base reduces the structural actions, bending moments and shears, associated with first mode type behaviour. However, as the wall above this level remains essentially elastic, higher mode responses that do not induce appreciable moments at the base of the wall, are not suppressed (NZS 3101:1995 Ref. 1). As a result both bending moment and shear force envelopes over the height of the walls are very different from those deduced from elastic based analyses where ductile behaviour at the base of the wall is assumed to reduce all modes equally.

A number of different distributions have been made for design envelopes for bending moments and shears in uniform walls. The recommended design envelope for bending moments in NZS 3101 is shown in Figure C5. This distribution limits yielding above the plastic hinge zone at the base to acceptable limits for sections without special confinement reinforcement. The corresponding distributions for shear forces are also given in the commentary to NZS 3101:1995 (Ref. 2).



(a) Elevation of frame



Column shears found assuming nominal strength
Column sway storey shear = $V_{A,i} + V_{B,i} + V_{C,i}$

(b) Column sway storey shear strength

FIGURE C3 ASSESSMENT OF COLUMN STOREY SHEAR STRENGTHS

Figure C3 illustrates how the column sway storey shear strength can be assessed. It is assumed that the columns can sustain their nominal flexural strengths at the top and bottom levels of the storey being considered. There is one limit on this method. This occurs where the equivalent static or first mode response analysis indicates that a point of contra-flexure does not develop in the storey being considered. In this situation it may be necessary to limit the moment which is assumed to act at one of the critical sections in the column, to ensure the plastic hinge deformation associated with the moment redistribution does not cause the material strains to exceed permissible values.

The beam sway storey shear strength may be assessed by considering the actions induced in the columns, when over-strength bending moments act in the plastic hinge zones and by scaling from an equivalent static or first mode analysis. The process is illustrated in Figure C4. Over-strength bending moments are assumed to develop simultaneously in the plastic hinge zones associated with the beam sway mode at the level immediately above the storey being considered. This is illustrated in Figure C4 (b) for the case where all the plastic hinges are located in the beams, and in Figure C4(c) for the case where some of the plastic hinges are located in the columns. A scale factor, *R*, which is equal to the sum of the over-strength moments acting on the columns at the intersection of beams and columns divided by the corresponding moments form the equivalent static or fist mode analysis, is found. With reference to Figure C4, this ratio is given by:

$$R = \frac{\sum M_{\mathrm{o,1}}}{\sum M_{\mathrm{E,I}}}$$

The beam sway storey shear strength is assessed by summing the shear resisted in each column in the storey. Points of inflection may be assumed to occur at the mid-height of each storey, that is a distance of $f_{\mathrm{i+1}}$ above level i and f_{i} below level i. The storey shear strength is given by:

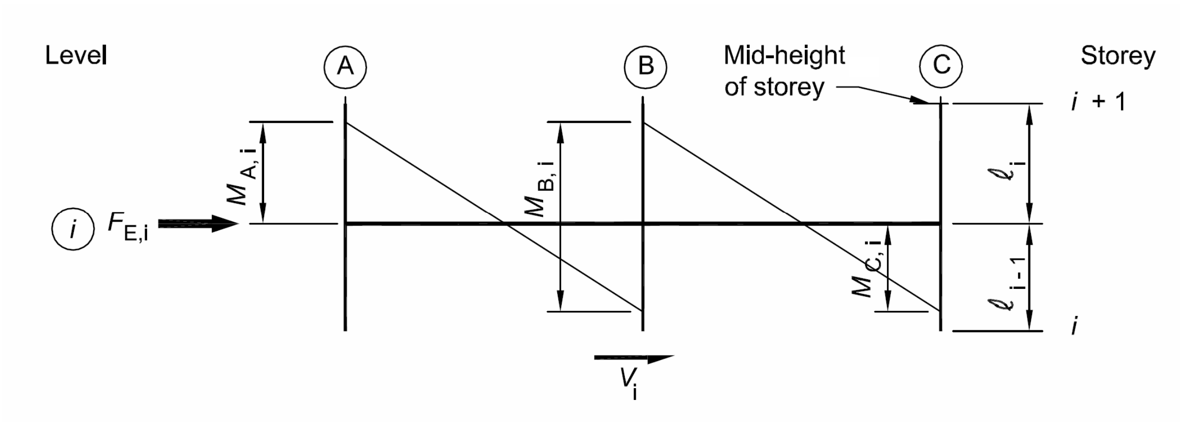
$$V_{\mathrm{bi}} = \sum \frac{M_{\mathrm{o,i}}}{f_{\mathrm{i+1}} + f_{\mathrm{i}}} + RF_{\mathrm{Ei}} \frac{f_{\mathrm{i+1}}}{f_{\mathrm{i+1}} + f_{\mathrm{i}}}$$

where F_{Ei} is equal to the lateral force at level i found from the equivalent static or first mode analysis, see Figure C4 (a). For the top storey the value of f_{i} is taken as zero.

The required column sway storey shear strength is found by multiplying the beam sway storey shear strength, V_{bi}, by k_{s}, and distributing this shear into the columns in the storey. In carrying out this process it is important to ensure that the resultant bending moments in the columns are such that the locations of the potential beam sway plastic hinge zones are maintained.

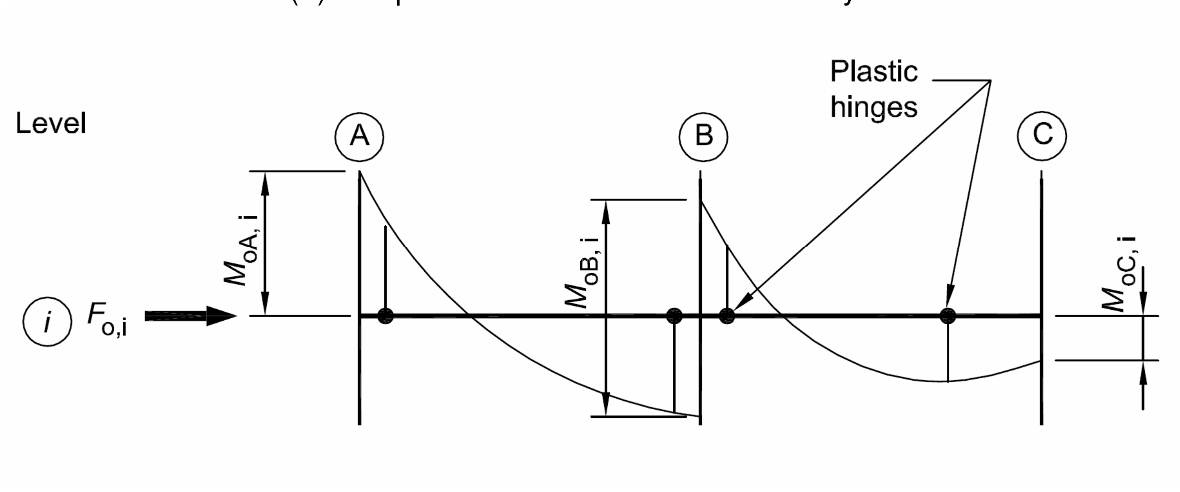
Other structures

The behaviour of other structural forms, such as structural walls or wall frames, is strongly dependent upon the material that is used. Guidance on appropriate dynamic magnification factors or distribution of design actions is given in the appropriate material Standards.



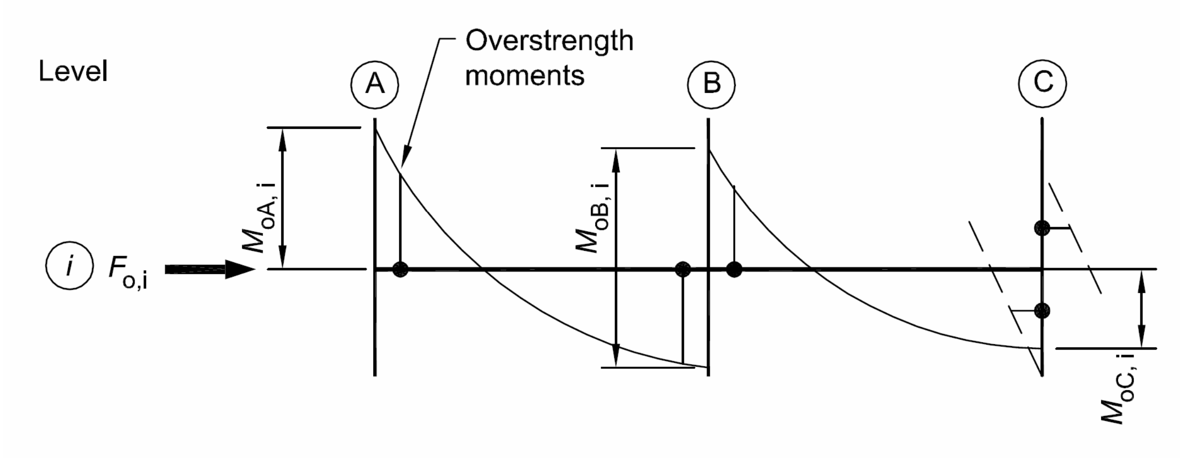
$M_{EI} = M_{AI} + M_{Bi} + M_{Ci}$

(a) Equivalent static or first mode analysis



$M_{oi} = M_{OA} + M_{OB} + M_{osi}$

(b) Plastic hinges in beams



(c) Some plastic hinges located in columns

FIGURE C4 BEAM SWAY STOREY SHEAR STRENGTH

C3 ASSESSING MATERIAL STRAIN DEMANDS IN PLASTIC HINGE ZONES

C3.1 General

The distribution of strain within a plastic hinge zone is complex and it cannot be easily determined. It depends on the material, the magnitude of shear and axial load that act, the way in which the potential plastic hinge is attached to an adjacent column, foundation pad or foundation beam, and the form of plastic hinge (unidirectional or reversing). In practice a designer is required to assess the likely order of material strain sustained by an inelastic zone in the event of a design level earthquake. This material strain is then used as an index to establish the appropriate level of detailing. For example, for a plastic hinge this index is curvature.

C3.2 General ductile moment resisting frames

The inter-storey displacement in a level is composed of two components, namely an inter-storey displacement due to the elastic response of the structure and a displacement due to the plastic deformation in the plastic hinge zones. From knowledge of the inter-storey drift in the ultimate limit-state and the drift corresponding to a ductility of one, the plastic hinge rotations can be assessed. However, in determining the critical rotation for design purposes two different forms of plastic hinge need to be recognized, namely unidirectional plastic hinges and reversing plastic hinges [3]. As indicated in the appropriate material Standards the peak rotation that can be sustained by each of these plastic hinge forms is different.

C3.3 Criteria for determining plastic hinge type

Reversing plastic hinges are illustrated in Figure C5 (a). These are sustained if either:

(a) The member strengths are varied along the member in such a way as to prevent positive moment plastic hinges forming away from the column faces, see Figure C6; or

(b) The shear induced by the bending moments acting at the two ends of the member exceeds the shear induced by the vertical loading. This case is illustrated in Figure C5(a) where the vertical loading shear, V_g, is less than:

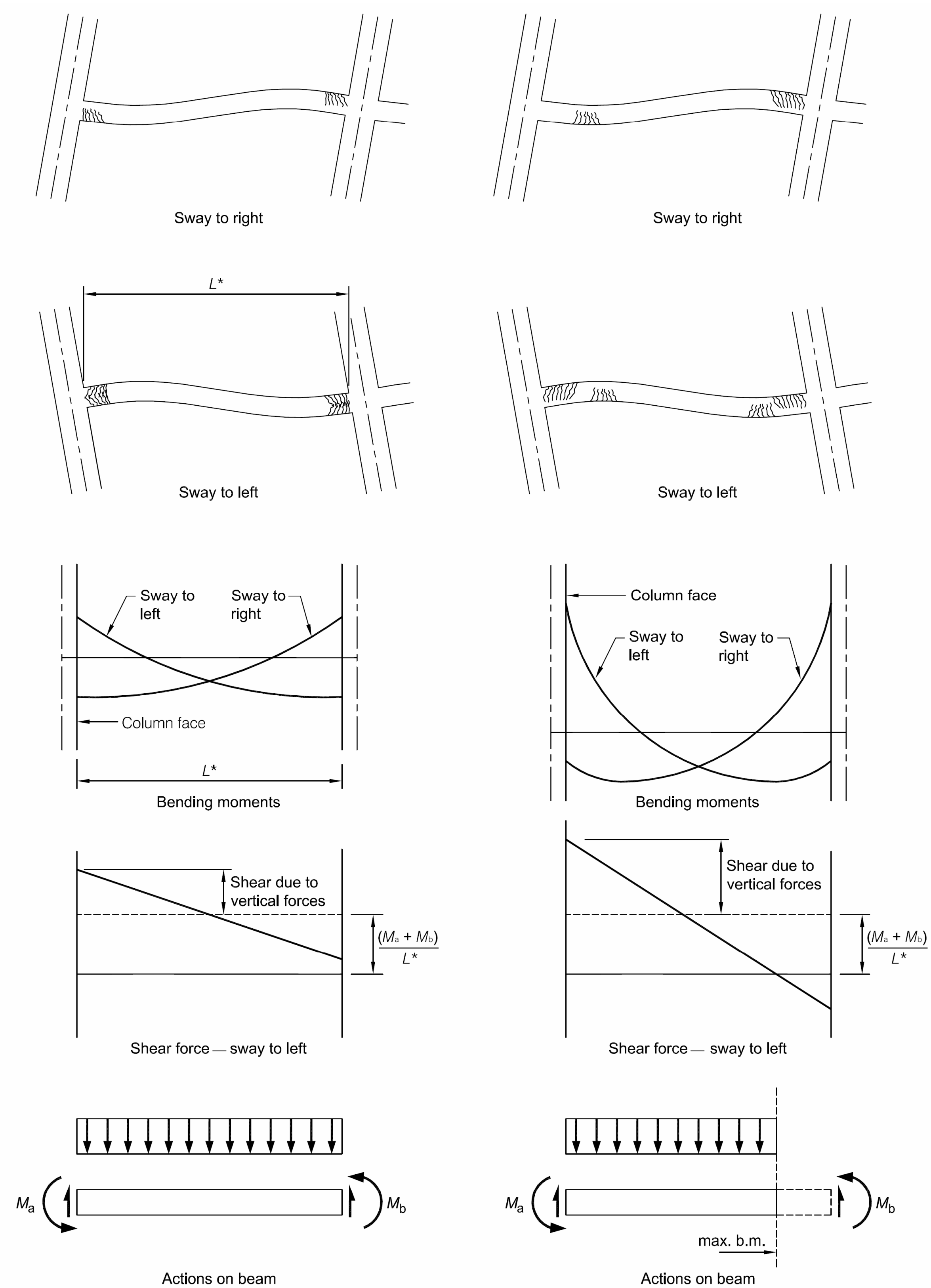
$$V_g < \frac{M_a + M_b}{L^*} \qquad \text{. . . C3.3(1)}$$

and M_a and M_b are the bending moments sustained at the critical sections of the beam, L^* is the distance between these sections (generally the column faces). If neither of these two criteria are satisfied a unidirectional plastic hinge can form in a severe earthquake, see Figure C5(b).

The theoretical criterion that establishes which type of plastic hinge may form, is given by Equation C3.3(1) for cases where additional strength has not been added to confine the location of positive moment plastic hinges (see Figure C6). The curvature limits for the two forms of plastic hinge should be given in the appropriate material Standard. For the cases where the gravity load shear, V_g, is close to the limiting value given by Equation C3.3(1) some interpolation between the limits given by the appropriate material Standard may be used to define the permissible curvature. In such cases inelastic zones may form as a mix of reversing and unidirectional plastic hinges. The form of plastic hinge that may develop in a particular situation can not be rigorously identified for loading cases close to the criterion given by Equation C3.3(1). This uncertainty occurs due to differing rates in which different plastic hinges strain-harden, the uncertainty in live loading and shears arising from vertical ground motion.

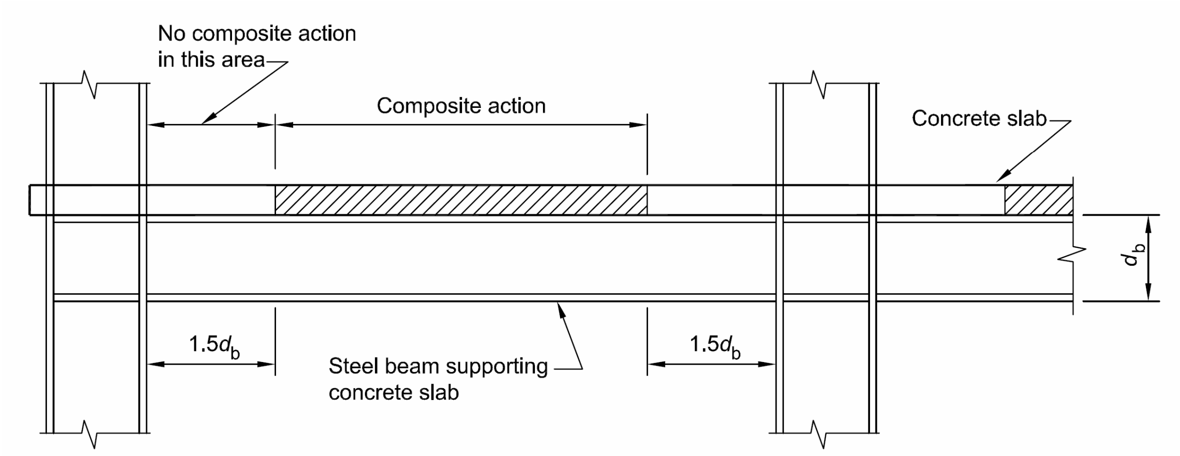
C3.4 Reversing plastic hinge zones

As illustrated in Figure C5 (a), with this form of plastic hinge, plastic hinge sway to the right generates a positive moment rotation close to the left-hand column and a negative moment rotation occurs close to the right hand column. A reversal in the direction of rotation results in rotations in the opposite direction. Consequently the maximum rotation that is sustained coincides with the maximum inter-storey displacement.

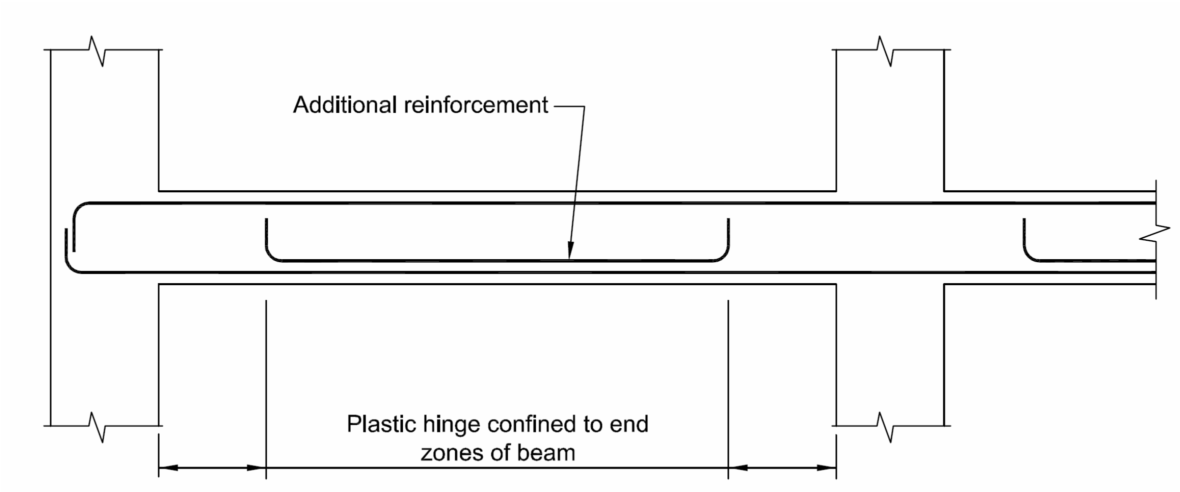


(a) Reversing plastic hinges (b) Undirectional plastic hinges

FIGURE C5 FORMATION OF REVERSING AND UNIDIRECTIONAL PLASTIC HINGES



(a) Structural steel beams supporting concrete slab



(b) Reinforced Concrete

FIGURE C6 STRUCTURAL DETAILS TO PREVENT FORMATION OF UNIDIRECTIONAL PLASTIC HINGES

Generally for ductile moment resisting frames the inter-storey drift due to elastic deformation of the columns is small. In Figure C7 the peak lateral displacement sustained in part of a ductile moment resisting frame is broken down into two components, namely that which can be sustained by elastic deformation, δ_e, and the value δ_p, which is associated with the plastic hinge rotations of θ_p in the beams. The plastic hinge rotation is related to the inter-storey drift by the expression:

$$\theta_p = \frac{\delta L}{h_i L'} \qquad \text{. . . C3.4(1)}$$

where h_i is the inter-storey height, L is the span of the beams between the column centre-lines and L' is the distance between the centres of the plastic hinge zones.

In practice the plastic hinge rotation can be conservatively estimated neglecting the elastic deformation of the structure. With this assumption:

$$\theta_p = \frac{\delta L}{h_i L'} \qquad \text{. . . C3.4(2)}$$

where δ is the inter-storey drift of the storey. Only in a few cases will it be necessary to use the more accurate expression and only critical inelastic zones need to be considered.

From the plastic hinge rotation the required curvature, which is one form of material strain, can be assessed as θ_p divided by the plastic hinge length. The effective length of the plastic hinge is given in the appropriate material Standard (it is generally of the order of half the beam or member depth).

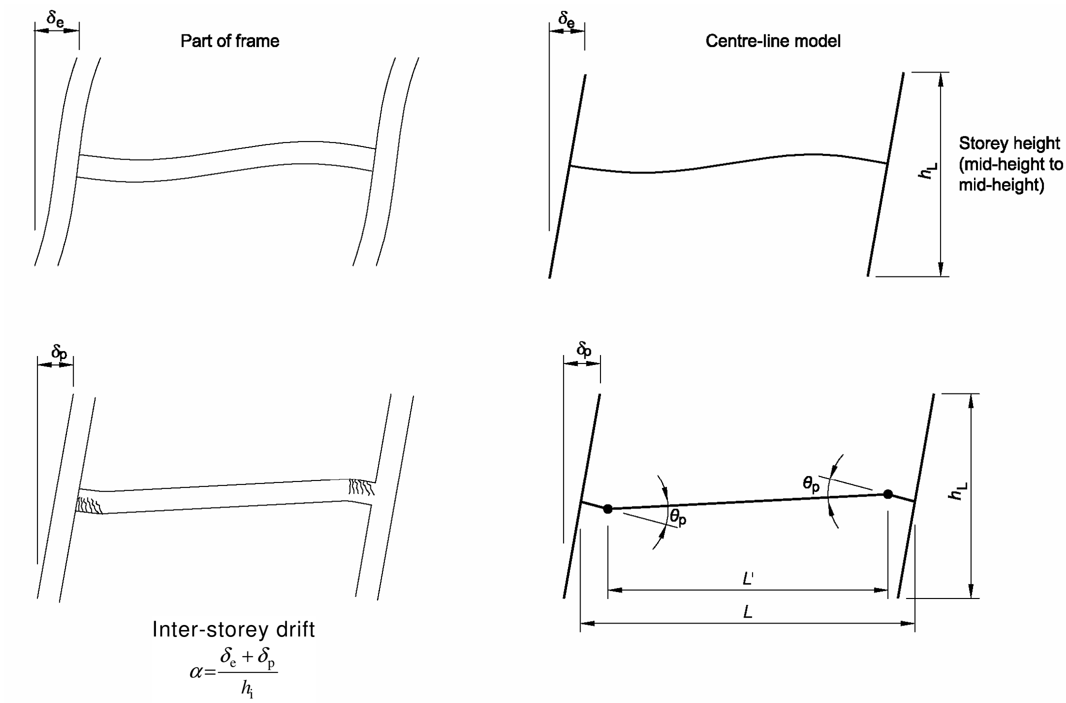


FIGURE C7 ELASTIC AND INELASTIC DEFORMATION IN PART OF A DUCTILE MOMENT RESISTING FRAME WITH A REVERSING PLASTIC HINGE

C3.5 Unidirectional plastic hinges

With unidirectional plastic hinges the gravity load shear exceeds the flexural induced shear due to the bending moments that can be sustained at the two ends of the member. The situation is illustrated in Figure C5(b). When sway occurs to the left-hand side a negative plastic hinge rotation occurs close to the left-hand column face and a positive moment plastic hinge forms in the right-hand side of the span. When the direction of displacement is reversed a negative moment rotation occurs in a plastic hinge close to the face of the column on the right-hand side and a new positive moment plastic hinge forms in the left-hand side of the span. However, the inelastic rotations sustained in the initial displacement to the left-hand remain. With each inelastic displacement of the structure during the earthquake the plastic hinge rotations increase (Fenwick et. al. Ref. 3) as illustrated in Figure C8(b). Hence the maximum rotation that is sustained is considerably greater than the equivalent rotation sustained by the reversing plastic hinge and it cannot be calculated by the approach outlined for reversing plastic hinge zones.

Analyses have indicated that the rotation imposed on a unidirectional plastic hinge is appreciably greater than that required for a reversing plastic hinge. However, it has also been found that the rotation that a unidirectional plastic hinge can sustain is appreciably greater than that of a reversing plastic hinge (Fenwick et. al. Ref. 3).

A series of time history analyses of structures forming unidirectional and reversing plastic hinges has been reported (Fenwick et. al. Ref. 3). It was found that with a displacement ductility of 6 the rotation imposed on a unidirectional plastic hinge varied from 2.5 to 4.4 times the corresponding rotation imposed on a reversing plastic hinge located on the column centreline. This ratio was found to decrease with a reduction in the displacement ductility.

On the basis of these analyses it is proposed that the equivalent rotation demand on a unidirectional plastic hinge can be assessed for design purposes as set out below and illustrated in Figure C8.

Determine the plastic hinge rotations that are required assuming that reversing plastic hinges in the beam are located on the column centrelines, that is angle α in Figure C8 (b). Multiply this value by the appropriate factor given below, depending on the magnitude of the structural ductility factor, μ,

$$1.0 + 0.63\,(\mu - 1) \qquad \text{for } \mu \le 2.0 \qquad \text{. . . C3.5(1)}$$

and

$$1.63\sqrt{(\mu - 1)} \qquad \text{for } 2.0 < \mu < 6.0 \qquad \text{. . . C3.5(2)}$$

Calculating the critical curvature for unidirectional plastic hinges follows the same approach as used for reversing plastic hinges. As the positive moment plastic hinges develop in a zone of low shear away from the column faces the inelastic curvature spreads over a longer length than is the case for negative moment plastic hinges.

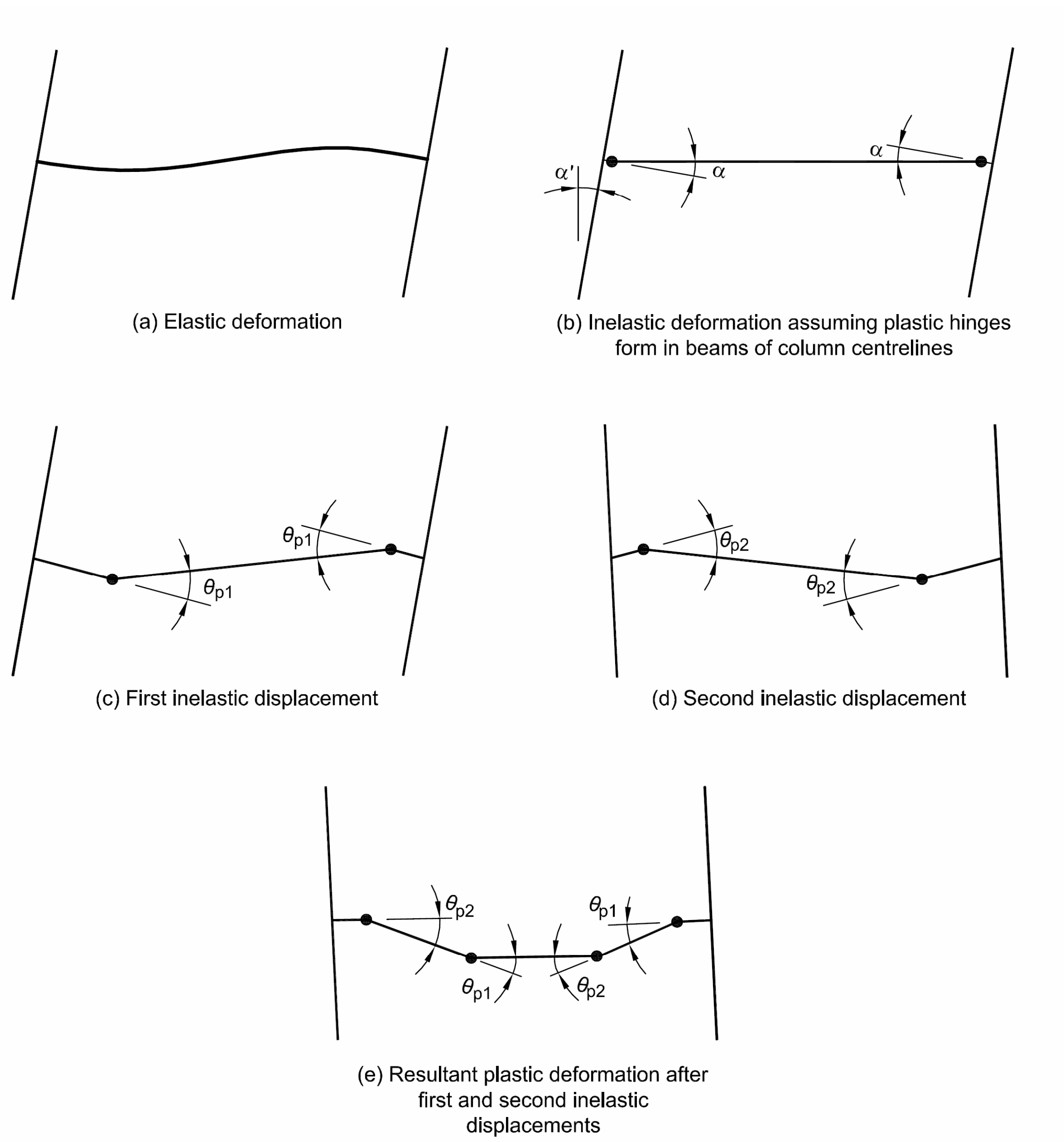


FIGURE C8 PLASTIC HINGE ROTATIONS IN BEAM WITH UNIDIRECTIONAL PLASTIC HINGES

C3.6 Structural walls

The rotation that a potential plastic hinge is required to be capable of sustaining may be assessed from the ultimate limit state deflection at the top of the wall. It is given by the expression:

$$\theta_p = \frac{(\mu - 1)}{\mu} \frac{\delta_t}{h'} \qquad \text{. . . C3.6(1)}$$

where δ_t is the lateral displacement at the top of the wall and h' is the height between the centre of the plastic hinge at the base of the wall and the top of the wall.

The curvature to be sustained by the plastic hinge at the base of the wall can be assessed by dividing by the effective plastic hinge length.

C4 STIFFNESS VALUES FOR SEISMIC ANALYSES

The stiffness to be used in analyses for seismic actions in the ultimate limit state, in which a structural ductility factor of greater than 1.0 is used, should be based on the member stiffness determined from the load and deflection that is sustained by the member when either:

(a) The material sustains first yield; or

(b) The material sustains significant inelastic deformation.

In either case, in assessing stiffness it is to be assumed that the member has been cycled to plus and minus the displacement corresponding to the critical deflection given above.

For serviceability limit state analyses the stiffness is to be based on several cycles of loading to plus and minus the level of anticipated deformation for that load level in the serviceability limit state (NZS 3101:1995, Ref. 1, Fenwick et al., Ref. 3).

REFERENCES

1 NZS 3101:1995, 'Concrete Structures Standard', Standards New Zealand.

2 NZS 3101:1995, 'Commentary on the Design of Concrete Structures', Standards New Zealand.

3 FENWICK, R. C., DELY, R. and DAVIDSON, B. J., 'Ductility Demand for Uni-directional and Reversing Plastic Hinges in Ductile Moment Resisting Frames', *Bulletin of NZ National Society for Earthquake Engineering*, Mar. 1999, Vol. 32, No. 1, pp. 1-12.

APPENDIX D

REQUIREMENTS FOR MATERIAL DESIGN STANDARDS

(Informative)

D1 GENERAL

This Appendix identifies the linkages that are required between the material design Standards and this Standard for earthquake design.

The overall objectives of this Standard are described in Clause D2.

D2 DESIGN OBJECTIVES

The objectives that this Standard has been written to satisfy are listed in Clause C2.1 of the Commentary to this Standard. In developing a material Standard, or in a special study, all of the objectives shall be considered.

D3 STRUCTURAL DUCTILITY FACTOR

Application of the provisions of this Standard requires that a structural ductility factor be assigned to the building under consideration. In some cases different structural ductility factors may apply for different directions depending on the structural configuration and construction materials used. It is expected that the structural ductility factor will be specified in the appropriate material design Standard.

It is the responsibility of the material standards to assign structural ductility factors that are consistent with the expected inelastic performance of the materials and structural configurations being defined (refer also to Clause 2.3.2). There is an expectation that the levels of inelastic material strain implied by a particular choice of structural ductility factor will be reliably achievable at the ultimate limit state load levels defined by this Standard.

Material standards shall consider the potential adverse effects of Localized high ductility demands where inelastic deformation is concentrated in relatively few locations. Even without this concentration additional ductility demand may be required to meet the collapse risk expectations set out in Clause C2.1. In these circumstances it is expected that the material standards may either reduce the structural ductility factor that can be used or alternatively apply other restrictions such as are discussed in Clause C2.1.

D4 MATERIAL STRAIN LIMITS

It is an expectation of this standard that material design Standards will define material strain limits for the specified detailing that will be appropriate for the ultimate limit state. It is also expected that the material Standards will provide the material strain capacity of the specified detailing to enable verification of the collapse risk where this might be attempted.

Material strain limits may be defined in several different ways. Limiting curvatures and plastic hinge lengths may be defined or limiting plastic hinge rotations defined. The difference between limiting values for both unidirectional and reversing plastic hinges shall be defined. Limiting shear strain and axial strain values shall be defined.

D5 INTER-STOREY DISPLACEMENT LIMITS

Further restrictions on inter-storey displacements for the ultimate limit state are expected to be specified in the material design Standards when the inter-storey displacement limits specified in this Standard are inappropriate for the structural configuration, material detailing or part under consideration. The inter-storey displacement limits for the ultimate limit state shall reflect the need for the material and structural configuration to meet the collapse risk expectations set out in Clause C2.1.

D6 INFORMATION TO BE PROVIDED BY MATERIAL DESIGN STANDARDS

The use of this Standard depends on design information being sourced from the relevant material Standard. This design information is to be consistent with objectives of this Standard and provide the following:

For the ultimate limit-state

1 Calculation of nominal strengths,

2 Strength reduction factors,

3 Maximum permissible structural ductility factors,

4 Minimum permissible structural performance factors,

5 Maximum permissible material strains including with axial strains, shear strains, section curvatures or plastic rotations and plastic hinge lengths, for both unidirectional and reversing inelastic zones, as appropriate for respective materials,

6 Maximum inter-storey deflection limits,

7 Detailing requirements related to material strain levels,

8 Section properties to be used in seismic analyses,

9 Calculation of overstrength actions,

10 Definition of regular and irregular buildings,

11 Dynamic magnification factors where required for columns, walls and combined wall frame structures.

For the serviceability limit-state

1 Calculation of strength for seismic serviceability requirements,

2 Structural performance factor,

3 Section properties to be used for serviceability seismic analyses,

4 Maximum permissible strain and deformation limits,

5 Detailing requirements for serviceability.

NOTES

NOTES

Approved by the Standards Council on 21 December 2004 to be a New Zealand Standard pursuant to the provisions of section 10 of the Standards Act 1988.

First published: 22 December 2004

The following SNZ references relate to this Standard:

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Typeset and printed by: Standards New Zealand