



## The Australia/New Zealand Earthquake Loadings Standard, AS/NZS 1170.4

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**ABSTRACT:** The development of the joint Australian/New Zealand earthquake loadings standard has been underway for the past 5 years as part of the development of the joint Structural design actions (Loadings) standard, AS/NZS 1170. Comments within this paper are based on committee draft 8 (December 2002) which, although expected to still be subject to further change, is considered to be technically approaching the final standard. The paper is structured to follow the layout of the new standard. Important changes are emphasised along with discussion of the background upon which the committee decisions were concluded.

### 1 SECTION 1 SCOPE AND GENERAL

Section 1 provides the link between Part 4 Earthquake Actions and the other parts of the standard defining structural design actions. While the types of buildings covered by the overall standard are defined in General requirements of Part 0, additional building types considered beyond the scope of the earthquake provisions such as bridges and dams are listed in this section.

Links are provided to the earthquake actions, ultimate limit state,  $E_u$  and serviceability limit state,  $E_s$  prescribed in Part 0. References between the building importance levels (in section 3 of Part 0 for New Zealand and the Building Code of Australia) and to the nominal imposed loads (Part 1) are provided.

### 2 SECTION 2 VERIFICATION PROCEDURES

Section 2 outlines the principles encapsulated in the standard and the associated performance levels expected during earthquake attack. The application of these principles is described in detail in the following sections of the standard.

The general overriding principle of sound earthquake engineering is to provide a reliable load path through which the lateral forces generated during the earthquake can be transferred to the ground. This is stated in Section 2 with the commentary related to this clause outlining the performance objectives associated with earthquake actions, namely

- Performance Level 1: Buildings can sustain more frequent earthquakes of moderate intensity shaking without damage that requires repair. This is considered as a serviceability limit state condition II (SLS II)

- Performance Level 2: Buildings can withstand major earthquake shaking with a reasonable margin against structural collapse, loss of support of heavy elevated parts or components, failure of systems necessary for building evacuation (ultimate limit state – ULS) or, in the case of Importance Level IV buildings, without loss of operational continuity (SLS I).
- Performance Level 3: Buildings can survive the most severe earthquakes with a small margin against collapse – this condition is not usually specifically considered during design. Buildings at this level are on the point of collapse with the reserve capacity of the structural systems has been consumed and the residual capacities of individual members having been exceeded to the point that instability is imminent.

In attaining Performance Level 2, well conditioned buildings, designed and detailed in accordance with dependable capacities prescribed in recognised structural material standards, will generally have sufficient reserve capacity to attain Performance Level 3 without further consideration. In low seismic areas, however, where the difference between the seismic shaking implied in objectives 2 and 3 can be large, there is likely to be insufficient reserve to meet objective 3 if objective 2 is only just met. To avoid the necessity to design for objective 3 in these circumstances the design actions have been enhanced by the introduction of a lower limit for the hazard. The implication of the lower limit is that all structures, no matter where they are located, should have a reasonable expectation of surviving a magnitude 6.5 earthquake with an epicenter within 20km of the site. In accordance with these performance objectives, variations in annual probabilities of exceedence of the design event are provided through the Return Period Factor, R, as indicated in Table 1. The introduction of the lower limit referred to above will result in a decrease in annual probability of exceedence for the ULS in areas of low seismicity. This is intentional.

**Table 1 Design annual probability of exceedence variance with building importance level**

Annual Prob. of exceedence	Return Period Factor R	Building Importance Levels			
		I Temporary structures	II Normal buildings	III Important buildings incl schools	IV Critical post disaster bldgs
1/2500	1.8				ULS
1/1000	1.3			ULS	
1/500	1.0		ULS		SLS I
1/100	0.5	ULS			
1/25	0.25		SLS II	SLS II	SLS II

Section 2 prescribes two verification procedures which satisfy the earthquake design provisions.

- Verification Procedure I (VPI) is a simplified means of compliance for Australia only. It is not permitted for use in New Zealand. VPI prescribes nominal lateral load levels (1.5%) of the seismic mass (G+0.3Q) applied at each level of the building. The action effect is to be considered in combinations with the gravity load and long-term live load to ensure adequate member strength. Joints and connections are required to have a capacity sufficient to resist 5% of the lateral load imposed, thereby creating a member/joint hierarchy. Face loaded walls within VPI are required to sustain 10% of their mass applied as a uniform load across their area. Other criteria (e.g. building torsion, lateral deflection, etc) are not required to be specifically considered for this method. Special exclusions are also provided for domestic building, thereby retaining the status quo of avoiding earthquake design of houses in Australia. It is the intention that application of VPI ensures that these buildings are well tied together and have recognizable load paths to reliably transfer seismic loads from any part of the structure to the ground. This procedure is intended to be more qualitative than quantitative.

- Verification Procedure II requires the determination of lateral forces, their distribution over the height of the building and between lateral load resisting systems, and the determination by analysis of the action effects that these forces develop within the structural system and all parts attached thereto. The strength requirements are satisfied when adequate levels of strength and ductility are provided according to one of the following three options:
  - ❖ Buildings designed and detailed according to capacity design principles with an establish hierarchy of post-elastic response may be designed for an earthquake base shear determined using a structural ductility factor  $\mu = 6.0$  and a structural performance factor of  $S_p=2/3$ .
  - ❖ Buildings designed for limited ductility are expected to experience a degree of inelastic behavior and are to be detailed in accordance with these actions, including a hierarchy which suppresses the formation of undesirable collapse mechanisms. The failure hierarchy for limited ductile structures will be achieved implicitly through the requirements of the materials standards rather than requiring explicit application of capacity design procedures. The earthquake actions imposed on such buildings are to be determined from base shear determined from a design spectra determined with  $\mu = 3.0$  and  $S_p=2/3$ .
  - ❖ Buildings detailed according to the general design requirements of the material standard are to remain either fully elastic ( $\mu=1.0$  and  $S_p=1.0$ ) or, when detailed in accordance with specific provisions within the appropriate materials standard, they may be considered as 'nominally ductile' ( $\mu=1.25$  and  $S_p=2/3$ )

The material standards are called upon to provide detailing appropriate for the anticipated levels of ductility within regions of plasticity and reserve capacity to suppress the formation of alternative mechanisms. Guidance is provided to the material standards writers (in Appendix C) as to the information expected within those standards to enable the earthquake provisions to be applied and satisfied.

Compliance procedures necessary to satisfy the serviceability limit state earthquake conditions and deformation control measures for both ultimate limit state and serviceability limit state conditions are outlined in this section. Reference is also made to the requirement that building parts be designed in accordance with the procedures outlined in Section 8.

### 3 SECTION 3 SITE HAZARD SPECTRA

Section 3 outlines the procedure for determining the site hazard spectra. McVerry (2003) provides more detail on the technical basis use to derive the site hazard spectra for New Zealand. The elastic site spectrum for the site is given by equation 1

$$C(T) = C_h(T) Z R N(T,D) \quad (1)$$

Where  $C_h(T)$  is the spectral shape factor (see Figure 1)

$Z$  is the hazard factor (see Figure 2 and for comparison Figure 3)

$R$  is the return period factor  $R_s$  or  $R_u$  for the appropriate limit state Table 1

$N(T, D)$  is the near-fault factor (see Table 2 (For Australia  $N(T, D) = 1.0$ ))

Figure 3 is a comparison between the seismicity of New Zealand as reflected the current loading standard (SNZ 1992) and that to be used in the revised standard. It is not included in that standard but is presented here to show how the seismicity studies have progressed. The 2002 map hazard contours have been scaled according to the NZS 4203 spectral values to enable a direct comparison which is not possible from Figure 2 since the basis for the spectral shape factor is different. The scaling is based on the spectral values at 0.5s for intermediate soil sites (NZS 4203 1992) and shallow soil sites in this standard. This results in significant swings across the country (e.g. Tauranga down to 0.5 from 0.8, New Plymouth down to 0.45 from 1.0 and Greymouth up to 1.2 from 0.9 from the NZS 4203 comparisons. Perhaps more significant is the new range of seismic factors which vary from 0.13 (Auckland), 0.16 (Hamilton), 0.20 (Tauranga) to 0.44 (Wellington) to 0.60 (Kaikoura & Otira))

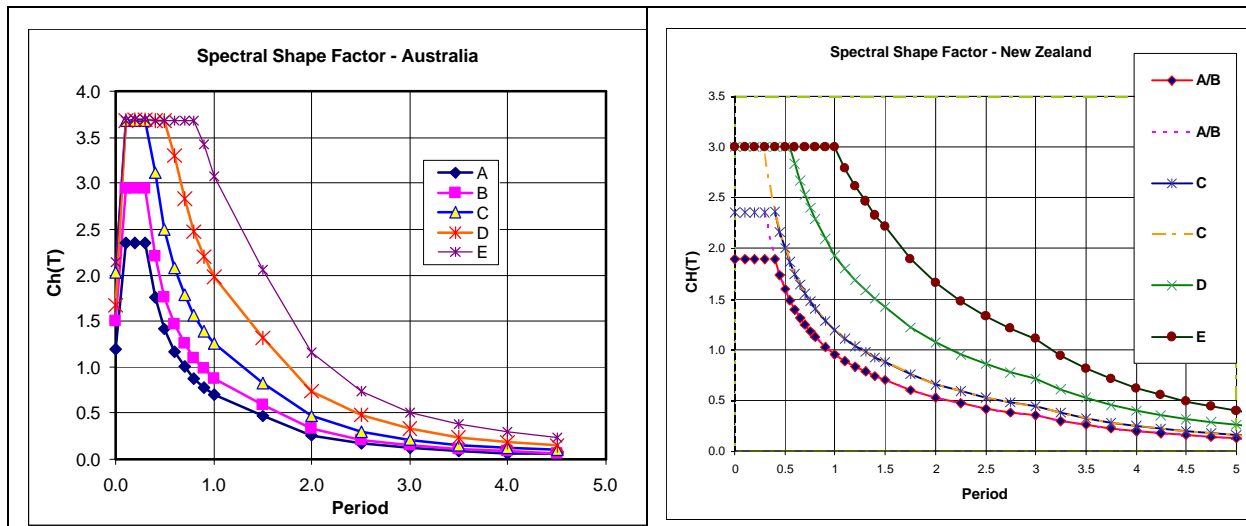


Figure 1 Spectral Shape Factors (Where A to E represent difference site classifications)

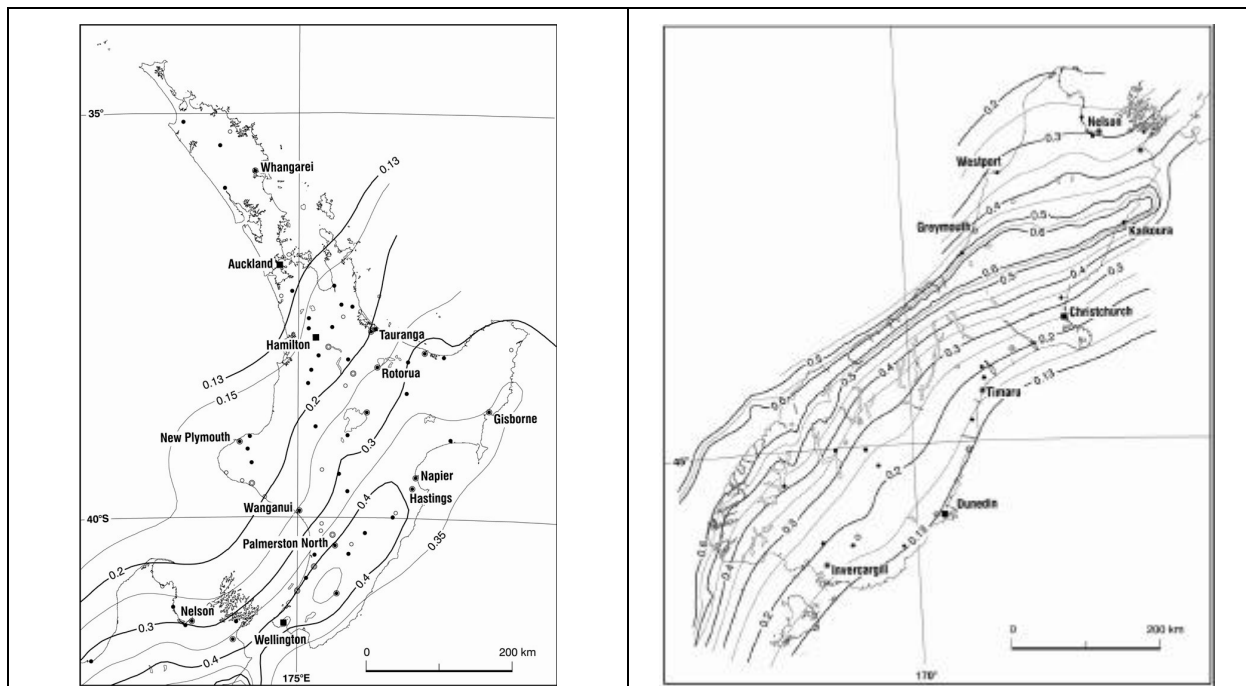


Figure 2 New Zealand Hazard Factors, Z

The near fault factor,  $N(T,D)$ , shown in Table 2, is applicable when the site is within 20 km of a major fault. It applies only to New Zealand and it has the effect of increasing the response spectra for periods greater than 1.5 second.

A list of major faults is provided (see Table 3) with an indicative map showing their general location. Reference to local knowledge or regional maps is expected when more detail is required for fault locations. A list of 127 towns and cities throughout New Zealand is included in the standard providing both the zone factor and maximum  $N(T,D)$  value for each centre where  $T$  is the building period and  $D$  the distance from the fault. This list will take precedence over the mapped values and will trigger users who need to consider near fault effects to look further than the standard.

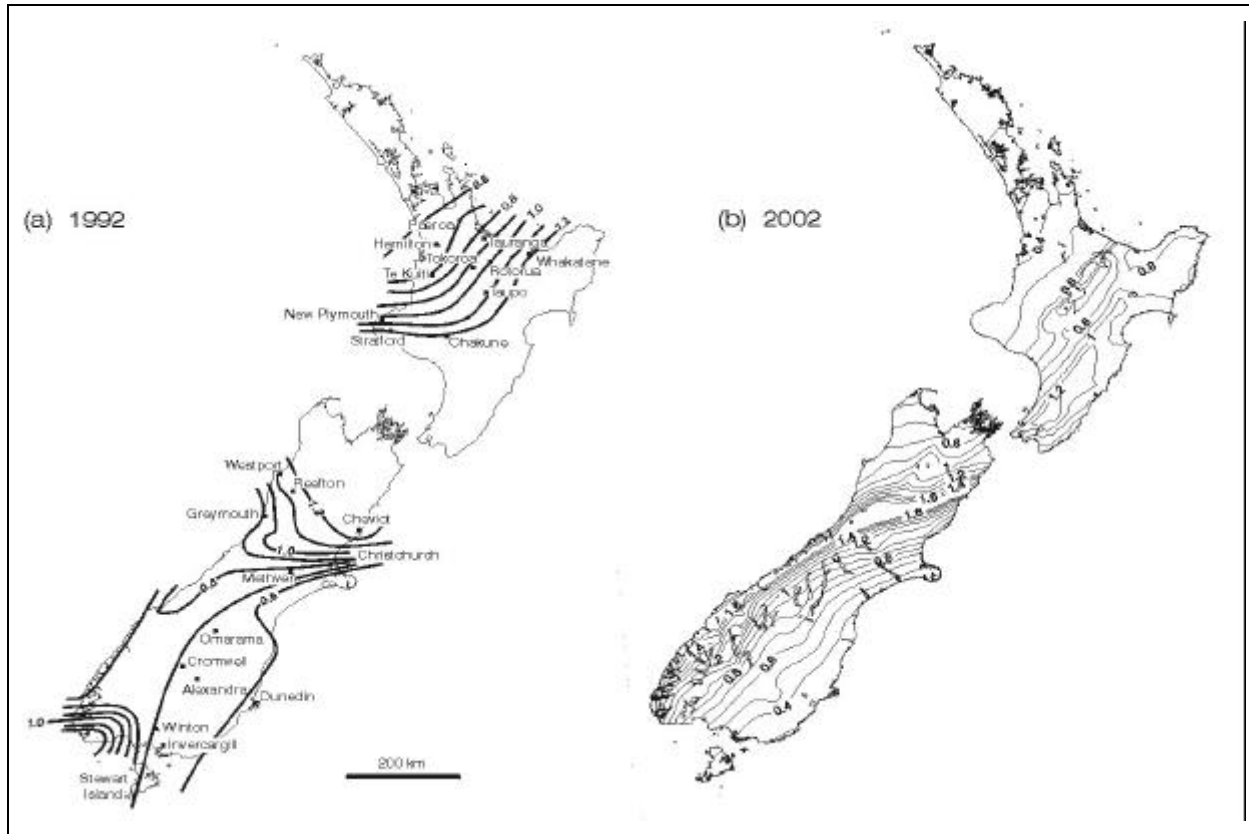


Figure 3 Seismicity Comparison AS 1170.4 and NZS 4203 (pers comm McVerry 2002)

**Table 2 Maximum Mean Fault Factor,  $N_{max}(T)$**

$N(T,D) = N_{max}(T)$ for $D < 2$ km $= 1 + (N_{max}(T) - 1)(20 - D)/18$ for $2 < D < 20$ $= 1$ for $D > 20$	Period T	$N_{max}(T)$
	< 1.5	1
	2	1.12
	3	1.36
	4	1.60
	$\geq 5$	1.72

**Table 3 Major Faults which require  $N(T,D) > 1.0$**

Major faults requiring Near-Fault Factors > 1.0		
Alpine	Kakapo	Wairarapa
Awatere	Kekerengu	Wairau
Clarence	Kelly	Wellington
Hope	Mohaka	

#### 4 SECTION 4 STRUCTURAL CHARACTERISTICS

This section recognises that earthquake actions are strongly influenced by the structural characteristics of the building itself. The entry point for both equivalent-static analysis and modal response analysis is the building period. Both these methods consider the building in its elastic state and assumptions are required to be made to translate this response into that associated with significant building damage (ULS).

The fundamental building response period,  $T$ , is pivotal to the determination of the structural base shear. The assumptions underpinning its determination are, however, imprecise. Permitted methods of determining the building period are the Rayleigh method, an alternative simple height dependent formula or 'an equivalent method' (e.g. assessment of period by elastic computer analysis). With the lower spectral ordinates, wider differential in zone factors between regions of high and low/moderate seismicity, and increased acceptable probability of annual return period serviceability limit state events, serviceability limit state considerations are expected to control member sizes in more buildings than in the past. This is acknowledged with the requirement that the section properties be considered for each limit state when determining the fundamental response period. Thus uncracked concrete sections are expected to be incorporated when considering damage limitation or, in the case of Importance Level IV buildings, the continued occupancy requirements of SLS I.

The determination of seismic weight and seismic mass used to determine the lateral base shear is outlined. Live load reduction factors,  $\psi_e$ , are defined as 0.6 for storage areas and 0.3 otherwise. These factors reduce the nominal live load to an average-arbitrary-point-in-time value consistent with that expected during a rare event such as a design earthquake.

The selection of the structural ductility factor directs users to the structural materials standard appropriate for the structural system being used. Considerably greater emphasis has been placed on the material standards to provide the design and related detailing information necessary for the designer and during construction. This information includes the structural ductility factor and maximum member curvature appropriate for the detailing provisions prescribed for the general design requirements, the limited ductility requirements and the capacity design requirements. A table of structural ductility factors is provided within Section 4 as an interim measure until the material standards are able to provide the detailing requested. These values are conservative estimates reflecting the expected ductility from current detailing practices. Guidance on what the material standards are expected to prescribe is given in informative Appendix C. This appendix highlights key issues that are to be considered when ascertaining the member curvature capacity and hence the structural ductility factor appropriate for levels of detailing prescribed.

The structural performance factor,  $S_p$ , has been retained. This is a somewhat nebulous quantity which recognises the contribution of non-quantifiable factors to the enhanced behaviour of actual buildings over their theoretical counterparts. International standards writers often amalgamate the structural ductility factor with the inverse of the structural performance factor. Within AS/NZS1170 the committee decided to retain these as separate factors. Two values of  $S_p$  are nominated, namely  $S_p=1.0$  for elastically responding systems ( $\mu=1.0$ ) and  $S_p=0.67$  for systems with ductility ( $\mu \geq 1.25$ ). The step function introduced is deliberate and seeks to recognise the proven poor performance of brittle systems compared to systems that exhibit even a small amount of ductility. It is not intended that the higher  $S_p$  apply to structures which have available ductility  $\geq 1.25$  but for other reasons are being designed for elastic ( $\mu < 1.25$ ) load levels.

Structural irregularity is recognised as decreasing the reliability with which designers can predict or model the response of buildings which are either structurally irregular in plan or in elevation. While the structural regularity assessment is determined in this section, restrictions on acceptable analytical methods for irregular buildings are prescribed in section 5.

Diaphragms are acknowledged as being an integral part of the primary structural system. Designers are required to use three dimensional modal or time history analysis methods which include modelling the diaphragm for irregular buildings greater than 15 m high. The diaphragm actions include the load transfer role between lateral force resisting systems and the inertia effects on each diaphragm.

Rocking structures have been reconsidered. While significant benefits are acknowledged from the energy dissipation of such systems, the potential for rogue rocking systems developing unforeseen response characteristics is highlighted in this section. The necessity to consider foundation systems supporting rocking systems as equivalent to  $\mu=2.0$  has been retained as a reasonable compromise.

## 5 DESIGN EARTHQUAKE ACTIONS

The design spectrum for equivalent-static or for modal analysis is derived by scaling the elastic response spectra in accordance with Equations 2 and 3 as follows;

$$C_d = C_h(T)ZR_sS_p \quad \text{for the serviceability limit state} \quad (2)$$

$$C_d = C_h(T)ZR_uS_p/k_m \quad \text{for the ultimate limit state} \quad (3)$$

Where  $k_m = m$  for  $T \geq 0.7$  sec and  $\frac{(\mu - 1)T_1}{0.7} + 1$  for  $T < 0.7$ . For the serviceability limit state, the building is required to remain predominantly elastic.

For the ultimate limit state inelastic response is acceptable. Three inelastic response zones are recognised across the spectral range namely equal energy at short period, equal displacement at medium and long periods and a transition zone approaching PGA for very short periods. The relationship given above approximates this using a linear transition from equal displacement at  $T \geq 0.7$ sec to unity at  $T = 0$ sec. Equal energy is achieved at approximately  $T = 0.3$ sec. The introduction of the lower period cut-offs for site soil classes C, D and E has a significant impact on the value of  $k_m$  for shorter period structures on these ground conditions. The New Zealand design spectra for  $\mu=3$  and  $\mu=6$  are shown in Figure 4.

The modal response spectra is derived by scaling the elastic response spectra by the first mode derive scale factor,  $k_{\mu}$ , determined for the fundamental building period  $T_1$ . The provision that the modal base shear be not less than 80% of that derived from equivalent-static design (i.e. the old  $S_{m2}$  factor) has been retained. This is recognised as being more of a design check that the modal combinations have been correctly combined rather than being a technically justified requirement but was retained because as a cost effective design quality verification measure.

How and where the earthquake design actions are to be applied is prescribed in this section. Two aspects are outlined, namely:

- the actions are to be imposed in an orientation which results in the most unfavourable action effect on the structure (with several exemptions being listed for regular plan buildings and orthogonal concurrency only being required for buildings with  $\mu \leq 1.25$ )
- the point of application of the lateral load be offset by 0.1 times the plan dimension of the building at right angles to the direction of loading to allow for accidental eccentricity

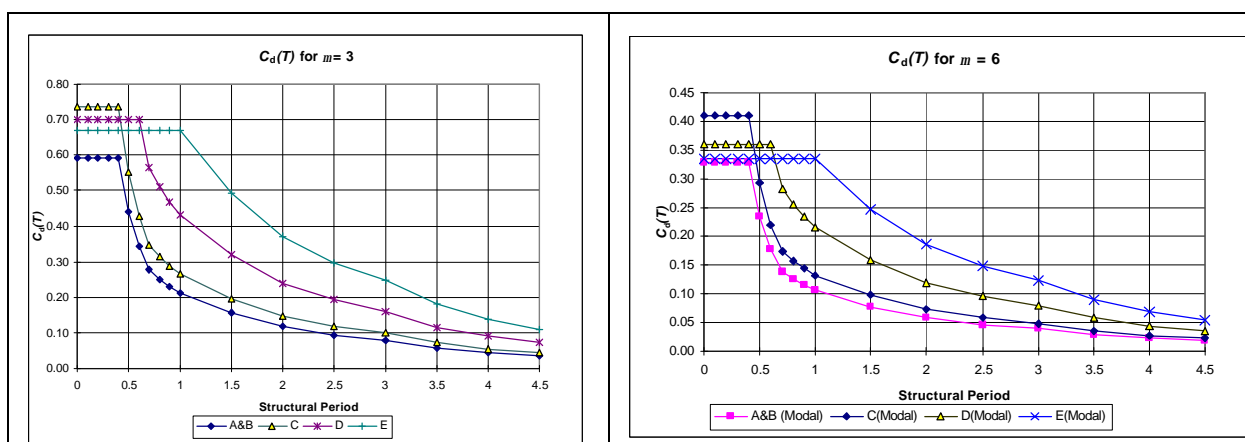


Figure 4 New Zealand earthquake design spectra

Numerical integration time history analysis (or Time-History Analysis as it is referred to within this paper) has been included in the standard. Section 5 requires that each assessment be undertaken using a family of not less than three real earthquake ground motion records. These are to have a seismic signature (i.e. magnitude, slip characteristic and site conditions) similar to that of the site under consideration. The selection of records with different characteristics can be expected for events of

different annual probability of exceedence (and thus for different building importance levels in the same locality). When the site is within a near fault zone one in three records selected are required to have a forward directivity component (i.e. be recorded from a site in advance of the rupture promulgation direction) while two in three are to be either neutral or have a backward directivity component. The record scale factor,  $k_1$ , is used to match the spectral values of the selected record to that of the elastic response spectrum over a target period range of between  $0.2T$  and  $1.5T$  where  $T$  is the fundamental period of the structure. The family scale factor,  $k_2$ , is used to ensure full coverage of the records within the target period range. Time-history analysis is to be carried out only on three-dimensional models, with both the primary and secondary horizontal components applied simultaneously in orthogonal directions.

## 6 SECTION 6 STRUCTURAL ANALYSIS

Three acceptable methods of analysis are included in the standard namely the equivalent-static method, the modal response spectrum method and the Time-history analysis method.

The use of the equivalent-static method is restricted to medium- and low-rise buildings consistent with the first-mode distribution assumption implicit in this method. The distribution of the base shear,  $V$ , between floors,  $F_i$ , is provided by equation 4 where  $F_t = 0.08V$  at the uppermost level and 0 elsewhere.

$$F_i = F_t + 0.92V \frac{W_i h_i}{\sum_{i=1}^n (W_i h_i)} \quad (4)$$

Where  $W_i$  and  $h_i$  are the seismic weight and height respectively of floor  $i$ .

The modal analysis method requires that sufficient modes are considered to ensure that the contributing mass is not less than 90% of the seismic mass of the building. The rules relating to accumulating the contribution from closely-spaced modes have been removed since it is improbable these modal characteristics will extend into the inelastic response range of the building.

The Time-history analysis method modelling requirements highlight the need to ensure that lower-bound results are obtained which are appropriate for the particular parameter under consideration (e.g. member ductility demand, inelastic deflection or drift, floor acceleration). This may require different models for assessing the acceptability of different response parameters in order that the most severe demand expected from all foreseeable events are in fact forthcoming from the analysis.

Assessment of P-delta effects is required for the ultimate limit state unless restricted height, period, structural ductility or stability coefficient limits are satisfied. For serviceability limit state considerations P-delta effects can be ignored. For ultimate limit state considerations, P-delta effects are required to be considered in buildings of fundamental period  $T > 0.6$  sec and those identified as having a potential stability problem. Two methods of assessing P-delta effects are provided. The simplified method assumes the maximum permissible inter-storey drift is attained at each level and is thus conservative. An alternative, more detailed method is also provided.

## 7 DEFORMATION CONTROL

Design deflections and inter-storey displacements are determined from one of the analysis methods prescribed in Section 6 for both serviceability and ultimate limit state conditions.

Deflections derived from either of the elastic methods of analysis (e.g. equivalent-static or modal) are required to be amplified by the ratio  $\mu/S_p$  to obtain the ultimate limit state deflections. It is expected that this will better approximate the actual displacements during earthquake shaking and will provide displacements that will be similar to those predicted using inelastic time-history analysis methods. The elastic deflection profile is scaled in this manner but consideration of all potential soft-storey side-sway mechanisms is required unless the formation of such mechanisms is specifically suppressed through the application of capacity design procedures.

Overall ULS lateral deflection limits relate to separation distances between adjacent buildings or for assessing the boundary clearance requirements. Interstorey deflection limits for the serviceability limit state are required to ensure that earthquake-induced building deformations do not adversely affect the required performance of other building components. For the ultimate limit state consideration the interstorey drift limit has been set at 0.025 of the storey height regardless of the analysis method used, notwithstanding with the increased scaling of the elastic shapes by  $1/S_p$ . The ULS deflections are also required to be sustained without loss of support of elevated elements and parts whose failure is hazardous to those within or around buildings.

## 8 SECTION 9 EARTHQUAKE DESIGN OF BUILDING PARTS

The design of building parts has been substantially revised and is now somewhat aligned to US practices as prescribed by the Natural Earthquake Hazard Reduction Programme (NEHRP) and the International Building Code. The previous correlation between the fundamental period of the building and that of the part and the consequential resonance effect that may result has not been found within studies undertaken at BRANZ. The correlation is stronger between the acceleration experienced on the floors of the building (and thence their inertial forces) and the peak ground acceleration rather than with the fundamental period response of the building.

Parts of buildings are defined as all parts of the structure, including secondary elements and their connections and services and equipment supports. A special study is required when the mass of the part exceeds 20% of the mass of the structures (i.e. when the inertia effect of the part will significantly affect the response of the primary structural system).

Parts categories and associated part risk factors,  $R_p$ , are as indicated in Table 4. The basis for assigning actual parts to the appropriate part category may vary from building to building depending on the function of the part in that particular situation. The aim of the part risk factor is to adjust the response of the part without requiring the reanalysis of the entire structure. It should also be remembered that parts within categories P1 to P4 are required to withstand ULS actions without loss of support and parts within category P5 are required to remain operational under a 1/500 year return period earthquake. Conversely parts within category P6 are those items within ordinary buildings where the onset of damage beyond normal SLS conditions is appropriate (e.g. sprinkler heads and fire separation walls) but the action is typically the 1/25 year return period event.

**Table 4 Classification of Building Parts (Note still subject to change within the final standard)**

Category	Criteria	$R_p$
P.1	Parts representing a hazard to life outside the building . *	0.9
P.2	Parts representing a hazard to a crowd of greater than 100 people within the building. *	0.9
P.3	Parts representing a hazard to individual life within the building. *	0.8
P.4	Parts necessary for the continuing function of life safety systems within the building	0.9
P.5	Parts required for operational continuity of the building.	1.0
P.6	Parts for which the consequential damage caused by its failure are disproportionately great.	1.2
P.7	All other parts.	1.0

(\* indicates that parts must weigh >10 kg and be able to fall >3 m to qualify)

Three design actions have been identified for parts, namely horizontal actions, vertical actions and

deflection-induced actions. Parts and their connections are to have sufficient strength and ductility to resist these actions without ‘failure’. They are also required to have sufficient stiffness to avoid pounding either with the primary structure or other parts. The ‘failure’ criteria for parts may involve the loss of support (i.e. the part itself sliding, toppling or falling) or damage to the part such that it requires repair or loses its contents.

The horizontal forces on a part is given by equation 5.

$$F_{ph} = C(0) C_f C_p R_p W_p \quad (5)$$

Where:  $C(0)$  is the site hazard coefficient with  $T = 0$

$C_f$  is the floor acceleration coefficient,

$C_p$  is the part coefficient,

$R_p$  is the part risk factor as given by table 3 (at SLS,  $R_p = 1.0$  for all cases),

$W_p$  is the weight of the part.

## 9 ACKNOWLEDGEMENTS

The authors wish to acknowledge the huge effort put into the preparation of the standard by all members of the drafting committee. It has been a long process which is now near the end of its course but the final standard will be a document with which we can all be pleased. The contributions of the following are particularly noted Arthur O’Leary (Sinclair Knight Metz, Wellington), Bob Potter (Cement and Concrete Association, Sydney), Charles Clifton (HERA, Auckland), Simon Matthews (Consulting Engineer, Sydney), Kevin McCue (Australia Seismology Centre), Ian Brewer (Standards New Zealand Wellington) and Richard Weller from Standards Australia. .

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