Future earthquake loadings standards

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ABSTRACT: The draft joint Australian/New Zealand earthquake loadings standard, which has recently been out for public comment, is described. This standard is intended to be a verification method within the general framework of the New Zealand Building Code. This paper summarises the principal aspects of the draft standard and the major differences between the proposed provisions and those in the current standard.
Comment is also made as to the future direction of overseas and international loading standards and the implications of such changes for New Zealand, particularly with regard to the building regulations applicable here and their application to international structural standards.

1 INTRODUCTION

The current New Zealand Loading Standard, NZS 4203, (ZNS 1992) was introduced in 1992 and is based upon technical information available during the mid to late 1980's. In accordance with Standards New Zealand's policy of reviewing standards every 5 to 7 years, it was timely that the loading standard be revisited, and if technical advances merited, revised. This is set against a background of a government directive to avoid potential trade barriers, particularly with Australia through our CER agreement which has result in a Memorandum of Understanding between Standards New Zealand and Standards Australia that future standards will be joint NZ/Australian standards. With this background, work began in the mid-90's to develop a framework for the next generation of structural engineering standards. A policy framework was developed by Standards Australia under which a common loading standard and a set of common materials standards could be developed, but which also acknowledged that while some material standards were reasonably closely aligned and could be merged without too much problem, others were markedly different and in these instances a common standard would be difficult. It was accepted that the Loading Standard was the parent document that needed to be developed first to enable the material standards to evolve by reference to the content and nomenclature of the loading standard.

Consistent with the desire to dismantle standards as a potential barrier to free market and trade was the expectation that internationally recognized standards were to form the basis of future standards with departures from those standards only under exceptional circumstances.

Perhaps the biggest oversight during this evolution was the impact that different regulations would have on the process. The loading standards review committee were advised during preliminary discussions with the regulators that the loading standards of Australia, AS 1170 and the that of New Zealand, NZS 4203, would not be accepted as verification methods if they were submitted to the Australian Building Codes Board (ABCB) or the Building Industry Authority (BIA) today. Their test for such documents is the solution provided should be correct, clear and complete and that variations from such a test was unlikely to be acceptable as a verification method.

It is with this background, of attempting to produce a verification method which satisfies both the ABCB and the BIA that the revision of the loading standards of Australia and New Zealand began.

This paper summarises the principal aspects of the draft Australian/New Zealand earthquake loadings standard and the major differences between the proposed provisions and those in the current standard. Comment is also made on the likely future direction of code development in New Zealand.

2 ISSUES FOR A JOINT AUSTRALIA/NEW ZEALAND STANDARD

There are a number of issues for a joint earthquake loadings standard for Australia and New Zealand.

For a standard applying across two countries it is essential that a consensus is reached regarding the design approach and philosophy.

The seismicity varies considerably across both countries; from areas of Australia that arguably have no seismic activity to regions in New Zealand where the seismicity is as high as anywhere in the world. There is a need to ensure that the complexity of the design process required of designers adequately reflects this considerable variation in seismicity so that the design methods for structures in low seismic areas are not too onerous. The joint loadings standard provides a platform for a more unified approach to earthquake design in the lower seismicity regions of Australia and New Zealand. This will enable a simpler approach for structures in some parts of New Zealand, but will require a more conscious and structured seismic design process for parts of Australia.

The standard must address all the soil types likely to be relevant for both countries. In some areas of Australia rock is encountered that has a significantly higher strength and shear wave velocity than any rock found in New Zealand.

The low seismicity generally in Australia means that there is little available historical data on earthquake occurrence. This has required a deterministic approach to evaluate seismic hazard rather than the probabalistic approach adopted in New Zealand. It has been important, therefore, to ensure that there is consistency in the overall level of hazard predicted across each country.

The standard is to be cited as a verification method in the Building Codes in each country. A verification method is intended to define a minimum standard and not necessarily best practice. It is also intended that a verification method define a standard that can be reproduced by all designers. This is a departure from the situation that has prevailed in New Zealand in the past where the earthquake loadings standard has attempted to convey best practice and has provided guidance on the use of a number of design techniques that have not always been well defined. For example the use of inelastic time history (ITH) analyses has long been recognised as a valuable tool for assessing the effects of earthquake shaking on buildings. It is also recognised that the use of this analysis technique requires significant judgement that can not be easily prescribed in a series of codified clauses. The ITH method is therefore not included in a mandatory section of the standard.

The commentary relating to the earthquake provisions identifies that while damage under design intensity events is considered acceptable, collapse is to be avoided in extreme events. These provisions have implications for designs which do not use capacity design to eliminate the potential for rupture of key support elements. Such buildings are expected to be prevalent in low seismicity regions. In New Zealand, avoidance of collapse under extreme events is addressed by limiting the Zone factor to be not less than 0.3. In Australia this additional provision is not considered as necessary.

3 OBJECTIVES

The formulation of the new standard is based on achieving three principle objectives. The objectives are that structures should be able to;

- Resist frequent earthquake shaking (such as might reasonably be expected at least once in the design life of the structure) with a low probability of damage that would be sufficient to prevent the structure being used as originally intended without repair,
- Withstand major earthquake shaking with a reasonable margin against collapse,

- Withstand the most severe earthquake shaking that the structure is likely to be subjected to, with a small margin against collapse.

The first objective is simply a restatement of the *serviceability* limit state. Objectives 2 and 3 represent two separate limit states but for the purposes of the proposed standard they are intended to be achieved by meeting one *ultimate* limit state. In moderate to severe seismic hazard areas it is expected that objective 3 will be met if objective 2 is met while in low seismic hazard areas it is expected that objective 3 will usually govern.

For New Zealand, verification of objective 2 is achieved using a loading level with the required level of risk. Compliance with objective 3 is achieved by defining a lower limit on the seismic hazard. The intention is to ensure that there is a reasonable chance that any structure will be able to survive earthquake shaking approaching the maximum credible for the area without collapse. The maximum credible shaking has been defined as that having a 2500 year return period or that resulting (with a 16% probability of exceedence) from a magnitude 6 earthquake with an epicentre say 20 km from the site, whichever is the greater.

The frequency of occurrence of the earthquake motions referred to in the objectives listed above and therefore the design load level for the various defined functional categories is varied depending on;

- the design working life
- the structures importance to the community
- the importance of the contents to the community, and
- the importance of the structure and/or the contents to the recovery period for a large earthquake.

The required hazard level (annual probability of exceedence) for the defined functional categories (Table 5) are shown in Table 4.

Table 1. Annual Probability of Exceedence for Ultimate and Serviceability Limit States

Functional Category	Ultimate Limit State				Serviceability Limit State
	Design Life				
	<5	25	50	100+	
I	*	*	*	*	*
II	1/1000	1/1000	1/2000	*	1/50
III	1/500	1/500	1/1000	1/2000	1/20
IV	1/200	1/200	1/500	1/1000	1/20
V	1/20	1/100	1/200	1/500	1/5

* indicates cases that are not covered by the standard

Table 2. Functional Categories

Functional Categories	Examples
I	Special high hazard structures or structures where a very low risk of failure is required
II	Post disaster functions
III	Hazards to crowds
IV	Normal structures
V	Low hazard to life or to other property

4 GENERAL FRAMEWORK

There has been a deliberate attempt in drafting the standard to separate the derivation of the seismic hazard from the derivation of the design earthquake actions. In this way the adjustments typically made to obtain design coefficients and spectra (eg adjustments at short periods) are transparent. This avoids the confusion that typically results when a site-specific earthquake hazard analysis is carried out as the results from such analyses are directly comparable with the hazard results provided in the standard and the same adjustments as defined in the standard can be made to obtain design values of earthquake load.

It is intended that the design process proceed as follows;

1. Determine seismic hazard
2. Determine structural characteristics
3. Determine design earthquake actions
4. Carry out the structural analysis
5. Verify that the requirements of the building code are met

5 SEISMIC HAZARD ANALYSIS

The site hazard spectra (elastic) for New Zealand presented in the standard have been derived from a probabilistic seismic hazard analysis carried out recently by the Institute of Geological and Nuclear Sciences (5). The seismic-source model incorporates 305 active faults and a grid of distributed seismicity sources with recurrence parameters estimated from the available catalogue of historic earthquakes. The attenuation relationships used are those derived for crustal earthquakes and subduction zone earthquakes derived by McVerry et al (6) for New Zealand. The hazard spectrum ordinates are derived directly from the analysis.

The relative contribution to the hazard of the various magnitude earthquakes has been determined using a weighting procedure which attempts to recognises the greater damage potential of larger magnitude (and therefore generally longer duration) earthquakes.

For convenience the hazard spectra are defined by the stylised equation;

$$C(T) = C_h(T)ZR \tag{1}$$

where; $C_h(T)$ are the ordinates of the hazard spectrum normalised to the hazard spectrum ordinate at 0.5 sec for site class B for a 500 year return period

Z is the geographical zone factor (0.5 sec period hazard spectrum ordinate for site class B)

R is the return period factor (ranges from 0.1 to 1.8 for 5 and 2500 year return period respectively)

The 0.5 sec reference period was chosen because at this period the best match was obtained between the Equation 1 above and the hazard results across all site soil classes, return periods and geographical locations.

The values for Z are shown as a contour zoning map. Refer Figure 1.

6 STRUCTURAL CHARACTERISTICS

Structural characteristics to be determined include; periods of vibration, seismic weight/mass, structural ductility factor μ, structural performance factor S_p, structural regularity and structure functional category.

The structural performance factor is used in the derivation of the design earthquake actions (refer 4.7) and currently is taken as 1.0 for m less than 1.25 and 0.67 elsewhere. The structural performance factor provides the correlation between the hazard spectrum for a given return period and the design spectrum necessary to ensure that the building will meet the requirements of the particular limit state at that return period. Consideration is being given to making the structural performance factor dependent also on redundancy.

7 DESIGN EARTHQUAKE ACTIONS

The design earthquake actions are defined typically in Equations 2 and 3 as follows;

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

$$C_d = C_h(T)ZR_uS_p/\mu \qquad \text{for the ultimate limit state} \tag{3}$$

For the equivalent static method the design actions are defined at $T = T_1$.

The design actions are applied at an accidental eccentricity of 0.1 times the plan dimension of the building at right angles to the direction of loading.

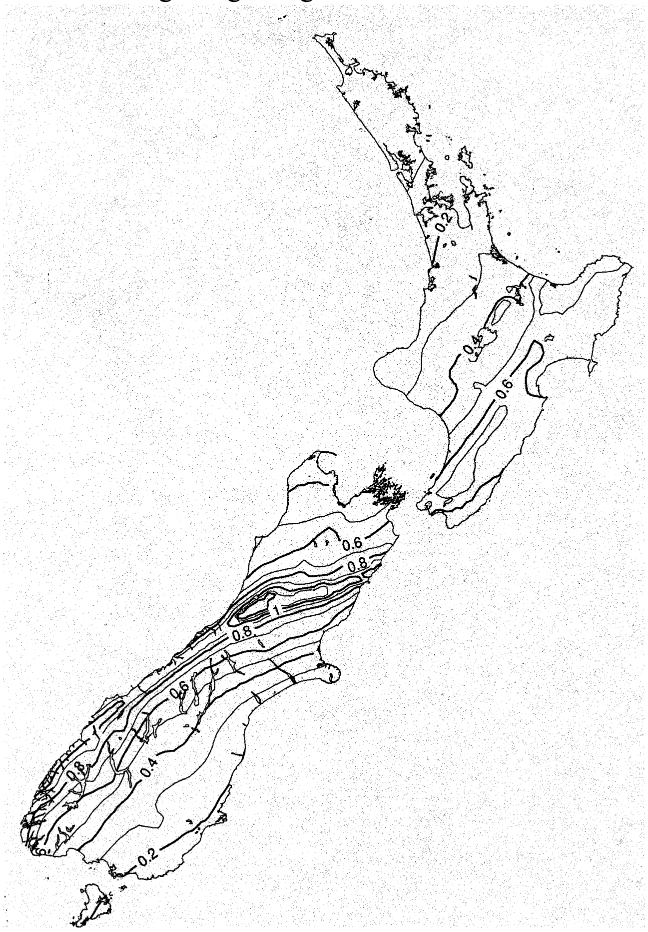

Figure 1 Zone Factor, Z

8 STRUCTURAL ANALYSIS

The structural analysis is carried out using either the Equivalent Static Method or the Modal Response Spectrum Method. Restrictions are placed on the use of the equivalent static method. Use of the Numerical Integration Time History Method is not included within the standard although recommendations are given in an informative appendix. If this method of analysis is used it is intended that it be carried out as a special study.

Assessment of P-delta effects is required for the ultimate limit state unless given height, period, structural ductility or stability coefficient limits are satisfied. Two methods of analysis are given. Method A is based on the assumption that the structure is displaced to the maximum allowable interstorey displacements given in the standard. The method is easily applied but is a conservative approach. Method B follows the approach set out in the commentary to the current standard.

9 DEFORMATION CONTROLS

Design deflections and interstorey displacements are determined using the same approach set out in the current standard. Deflection and interstorey drift limits are prescribed for the ultimate limit state to ensure satisfactory performance of the building as a whole and to provide some protection to neighbouring property. There is also a requirement that under the serviceability limit state the deflection of Parts shall be limited so as not to impair their function nor that of other building components.

10 **VERIFICATION**

The minimum acceptable verification method (refer Table 6 following) is determined by reference to:

a) *the site hazard spectra*, $C_h(0.5)$, from tabulated period-dependent data for each of 4 soil types normalised to 0.5 seconds (Note: for NZ conditions these range from 1.0 for rock sites to 1.35 for soft soils).

b) *the seismic zone factor*, *Z*, by reference to isoseismal zonation maps (Note for NZ, Z ranges from 0.15 in Northland to 1.1 in the Alpine fault region but currently has a minimum set at 0.3)

c) *the return period factor*, *R*, by reference to the building classification table given in Part 0 of the loadings standard and a magnification factor which adjusts the base spectra.

Within low and moderate seismicity regions (ie where the base coefficient < 0.35, which for New Zealand includes Auckland, North Auckland, coastal South Canterbury and Dunedin and most of Australia on all but soft or very soft soil sites, capacity design provisions can be waived provided the ductility level is not greater than 3. In such cases compliance is required with the limited ductility provisions within the various materials standards. In such cases, concurrent actions (100% plus 30% orthogonal) is required for elements that are common to two orthogonal primary load resisting systems. This is in recognition that without a rational capacity design, elements cannot be relied upon to maintain their load-carrying capacity under overload conditions. Conversely, the additional detailing required within plastic hinge zones (for structural ductility of μ>3) can reasonably be expected to limit the consequences of concurrent actions.

Table 6. Earthquake Design Verification Methods

Base Parameter $C_h(0.5)ZR$	Verification Method	Implications
≤0.1	No earthquake provisions	
≤0.15	VM I	▪ A primary lateral load resisting system capable of resisting 1% of the seismic mass. ▪ Connections capable of resisting 5% of the vertical self weight and imposed actions.
<0.35	VM II	▪ Earthquake action from equivalent static or multi-modal analysis. ▪ Strength and detailing from material standards but with structural ductility ≤ 3.0
≥0.35	VM III	▪ Earthquake action from equivalent static or multi-modal analysis. ▪ Yielding and non-yielding primary structural elements differentiated (ie capacity design approach implied) ▪ Detail in yielding zones according to material standards

Verification Methods II and III require either a equivalent static analysis or a multi-modal analysis to be undertaken to determine the base shear and/or modal shape upon which the base shear is to be distributed up the structure.

Drift limits for both serviceability and ultimate limit states are to be checked to ensure interstorey drift does not impair functionality (at SLS) and that overall lateral deformation is maintained within acceptable limits to avoid either significant P-Δ effects or pounding with adjacent properties.

11 **PARTS AND COMPONENTS**

The assessment of the effect of earthquakes on parts and components has been tiered to permit either a simplified (conservative) approach or a more complex detailed approach as necessary. The process involves the calculation of ground and maximum expected floor accelerations, and assessment of the loads on the particular part which are dependent on the part period, hazard presented by the part and part ductility.

12 FUTURE DIRECTIONS

The future development of standards is of concern to many within New Zealand. As SNZ operates on a "user pays" basis and receives no direct government funding, there is an imperative need to cover the costs of Standards New Zealand managing either the revision of an existing standard or the development of a new standard. It is unfortunate that in almost all cases, the sales of standards do not anywhere cover SNZ's costs of development. This raises the economic bar significantly. Now not only are committee representatives required to commit their own time and cover their own expenses, their organisations are also being targeted to provide direct funding for Standards New Zealand to cover its costs. The cost structure developed by Standards ranges from a relatively modest amount (≈ \$2500) when an international standard can be simply adopted for use in New Zealand, to \$7,000 for the development of a joint NZ/Aus standards regardless of where the secretarial responsibility lies, to between \$20,000 and up to \$100,000 for a New Zealand only standard. These costs are to cover the Standards operations only and do not cover the time or disbursements of committee representation.

Changes in the workplace are also having an impact on the direction of future standards. The university staff are under strain with their teaching and research activities. Participation in the development of standards is not acknowledged as being significant when compared with technical reports and journal articles when promotion is being considered. Similarly time constraints within consultancy practices are very much productivity orientated and being able to release usually senior staff for committee participation is becoming a huge strain on attaining productivity targets.

With this background it is expected that changes will occur in the development of future structural standards that we use in New Zealand. The most obvious alternative is to adopt overseas design and material standards. A huge effort has gone into the development of the structural European design standards. Eurocode 8 (ENV 1994) encapsulates the seismic design provisions. Although it has been operating in trial mode with national norms being used to calibrate its provisions with the national standards, this is now being reviewed with a view to these values being settled across all member economies.

The Technical Committee 98 – Bases for the Design of Structures, of the International Organisation for Standardisation, ISO, has had a working group developing a draft revision to ISO 3010 – Seismic Actions (ISO/FDIS 3010) which has recently been released as a final committee draft. The preamble to this draft clearly defines it as being intended as a guideline document from which national organisations can develop their own seismic design provisions. As such it is inappropriate as a working design standard.

Traditionally many New Zealand structural design standards evolved from US practices. As with Europe, there has been considerable effort being expended in upgrading the design requirements within the US particularly with the merging of the three building codes into the International Building Code. IBC 2000 has been published and is being adopted by most jurisdictions where the previous three model codes were employed. California is a major exception where it has been decided that the Uniform Building Code (ICBO 1997) will continue to be applied at least until the publication of IBC 2003. Somewhat independent of this was the development of the National Earthquake Hazard Reduction Programme (NEHRP) provisions published as FEMA 302/303-1997 (FEMA 1997). The NEHRP provisions have their geneses in ATC 3 and are written as a source document or code writers guide rather than a standard. As such they are unsuitable to be cited as a means of compliance within the model building codes without them being translated in 'standards language'. This process is now being undertaken by the American Society of Civil Engineers (ASCE) who have approved standards writing endorsement. The process is underway although it appears unlikely that it will meet the deadline for inclusion in the 2003 revision of the IBC. One area of significant difference between the seismic provisions of the UBC 1997 and the NEHRP provisions is that the former continues to use seismic zones (with an enhancement for near fault effects) while within the later design levels are based essentially on 2/3 of the USGS contour maps for the Maximum Credible Earthquake (2500 years with some twists in active regions). In fact the results derived from either approach are quite similar in most instances.

With this background of ISO standards, Eurocode and US regulatory changes to their model building codes and the conversion of the NEHRP provisions to Standards, the concept of New Zealand being able or interested in adopting one or other internationally recognised standard is remote. The uniform hazard spectra approach adopted in New Zealand has a much stronger rationale that more closely aligns to the performance-based regulations under which the New Zealand Building Code operates than the alternatives. The ongoing reluctance of the seismic design procedures to recognise the non-linear concepts embodied within the capacity design approach applied in New Zealand since the early 1980's is of sufficient concern that seismic design standards which are to be used here would need to clearly encapsulate this concept.

A major difficulty that the NZS 4203 review committee was faced with however was the edict from BIA that to be called up as an verification method within the Approved Documents, the revision must be clear, correct and complete. It was also intended that it would result in buildings which met but did not markedly exceed the structural performance expectations implicit within Clause B1 of the NZBC. Another constraint that only more recently became apparent was that the requirements of the standard were to be 'economically neutral' (ie any increased building costs resulting from the application of the standard were to be offset by economically assessed benefits). The implications of this concept have yet to be put to the test. The committee was however told that the existing NZS 4203 would be unacceptable as a verification method under the NZBC if it were to be submitted for endorsement in 2001. Yet it remains the 'benchmark' against which the future standard is to be assessed.

It is the authors belief that in future, new or innovative alternative design techniques (eg displacement based design) will evolve from a guideline or pre-standard prepared from an informed sector of the engineering fraternity which will operate as the basis for alternative solutions being offered for specific building consents for a period. Once its practice has become sufficiently widely accepted, the pre-standard is likely to be further refined and could then be offered to BIA as an alternative verification method to the traditional Loading Standard and ultimately included as part of the standard itself.

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