Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 51, No. 4, December 2018

OPINION PAPER

REFLECTIONS ON NEW ZEALAND'S EARTHQUAKE RESISTANT DESIGN APPROACH – CAN IT BE IMPROVED?

Donald K. Kirkcaldie[1]

(Submitted *June 2018*; Reviewed *July 2018*; Accepted *October 2018*)

ABSTRACT

Perceived shortcomings in NZS 1170.5 [1] and some other Standards are highlighted and areas for improvement are suggested. A particular focus is placed on achieving the principal objective of achieving life safety at the limit state at which structural collapse is to be avoided. Topic areas commented on include:

- The objectives of earthquake resistant design, especially that of avoiding the collapse of structures
- The appropriateness of current classifications of buildings into importance levels
- The currency and adequacy of the design seismic hazard spectra requirements
- The justification for, and application of, a structural performance factor
- The force-based and displacement-based methods of analysis and design, and the effects of plastic hinging relieving member permanent load moments at plastic hinges adjacent to points of support
- Consideration of displacement effects, and effects on displacements, at the limit state at which collapse is to be avoided
- Achieving reparability
- Some shortcomings in the material Standards for both structural steel and reinforcing steel
- Consideration of site conditions, and in coastal locations the tsunami risk
- Comparability of New Zealand design requirements with other major design codes.

INTRODUCTION

This paper seeks to highlight shortcomings perceived by the author to exist in NZS 1170.5 [1] and related Standards and to suggest areas for improvement in New Zealand's practice of earthquake resistant design. In particular, if life safety is to be ensured the author considers that the limit state at which structure's collapse is to be avoided needs to be specifically considered and ensured. Collapse avoidance cannot be simply assumed based on an arbitrarily assumed margin of additional capacity that in reality may not exist for many structures, especially in non-ductile elements of the total structural system such as the foundation soils, or in elements whose performance in larger than ULS events is dependent on their displacement capacity and ability to maintain their integrity and stability. While the NZS 1170.5 Standard [1] makes no mention of a collapse limit state or a limit state at which collapse is to be avoided the author believes it to be alluded to in the NZS 1170.5 Commentary [1], in particular by clause C3.1.4, but also by clause C2.1.

Thinking particularly about the ability of structures designed to current standards to avoid collapse at the limit state at which collapse is to be avoided, the following set of questions outline the scope of this paper and the questions to be addressed:

- Do structures designed to the current standards have adequate protection against collapse?
- Does the existing classification of structures into importance level provide an appropriate hierarchy of the importance of various buildings types and other structures?
- Are the currently specified design hazard spectra up to date and reflect the current state of knowledge in this area and current practices in deriving design hazard spectra? Is probabilistic seismic hazard analysis producing design hazard spectra that adequately address the risk of structural collapse in all regions and cities of New Zealand?
- Scaling down by a factor of 1/1.5 is applied from the collapse avoidance limit state (CALS) event to the ULS event, regardless of structural form, and then further scaling down is applied by a structural performance factor (S_p). For fully ductile structures S_p can be taken as low as 0.7 resulting in a total scaling down by a factor of 0.46. Can this level of scaling down be justified?
- In recent times an alternative method of structural analysis and design, displacement-based design, to force-based design, has been promoted. How do the methods compare? Also, are current design approaches fully taking into account the effects of plastic hinging occurring in structural members?
- Does the existing NZS 1170.5 [1] adequately assess the displacements that will occur at the CALS and address the maintenance of integrity and stability of structural elements necessary for the avoidance of collapse?

[1] *Semi-retired Engineer, Advisor to NZ Transport Authority, Wellington, donald.kirkcaldie@xtra.co.nz (Past Member)*

- The recent 2011 and 2016 earthquakes resulted in a huge number of buildings being demolished, calling into question the repairability of structures designed to dissipate earthquake energy though plastic hinging in structural members. Are there different approaches that can be adopted that will achieve repairability of earthquake damaged structures?
- Do the construction materials in use measure up to the quality required of them to achieve the performance needed to avoid structural collapse under CALS earthquake events?
- Do the existing requirements give adequate consideration to site conditions such as its location, topography, and nature of its soil conditions, and to other associated risks such as tsunami?
- How do New Zealand's design requirements for earthquake resistance compare with those of other major design codes internationally?

EARTHQUAKE RESISTANT DESIGN OBJECTIVES

New Zealand Building Code Specified Objectives

Within the New Zealand Building Code (NZBC) these are captured by Clause B1 Structure [2] which defines the objectives of the provision as being to:

1. Safeguard people from injury caused by structural failure,
2. Safeguard people from loss of amenity caused by structural behaviour, and
3. Protect other property from physical damage caused by structural failure.

Dictionary definitions of amenity include:

- a desirable or useful feature or facility of a building or place.
- the pleasantness or attractiveness of a place.
- In real estate and lodging, an amenity is something considered to benefit a property and thereby increase its value.

Thus loss of amenity may be taken to include a community's loss of the financial investment in a structure or facility should it collapse.

Note that these objectives do not have any probability of exceedance, (i.e. of not being met), associated with them. The probability of exceedance may therefore potentially be different for each objective.

AS/NZS 1170.0 and NZS 1170.5 Specified Requirements and Commentary

NZS 1170.5 Structural Design Actions – Earthquake Actions – New Zealand [1] is the sole Standard cited by the NZBC Clause B1 [2] for earthquake design actions, even though it excludes a range of structure types. None the less, other design standards, e.g. NZTA's Bridge Manual [4], have generally sought to align with this standard in order to meet NZBC [2] and Building Consent requirements. NZS 1170.5 [1] defines performance requirements for the ultimate limit state (ULS) and the serviceability limit state (SLS), but it should be noted that the ultimate limit state as defined in NZS 1170.5 [1] is not the true ultimate limit state as defined by AS/NZS 1170.0 [3] but is defined in the following:

"In its application to earthquake resistant design, the ultimate limit state is defined as being when the capacity of an element and/or the structure is reached, based on the design strength, strain, ductility and deformation limits that are specified for the ultimate limit state in this and appropriate material standards. The structure as a whole may have sustained significant structural damage, but shall have reserve capacity to avoid structural collapse."

Calling this limit state the ultimate limit state is a misnomer, is misleading, and should be dispensed with. A possible alternative name for this limit state could be the "damage control limit state", or to call the design earthquake event the "design basis earthquake".

For the ULS, the NZS 1170.5 Standard [1] specifies the performance requirements as being:

1. Maintenance of overall structural integrity and gravity support, while accounting for horizontal and vertical deflections, soil structure interaction, and sliding of the structure or its parts;
2. Maintenance of stability against overturning;
3. Avoidance of collapse or loss of support of parts of categories P.1, P.2, P.3 and P.4 (refer to Section 8 of NZS 1170.5); and
4. Avoidance of damage to non-structural systems necessary for building evacuation following earthquake that would render them inoperative.

Let us now consider the NZS 1170.5 commentary [1]. Clause C2.1, under Objective 2, discusses life safety and states:

"Internationally, an accepted basis for building code requirements is a target annual earthquake fatality risk in the order of 10^{-6} (ISO 2394:1998 [5]). In design terms it is generally accepted that fatality risk will only be present if a building fails, i.e. collapses. The maximum allowable probability of collapse of a structure, or part of a structure, is then dependent on the probability of a person being killed, given that collapse has occurred. This conditional probability will be dependent on structural type and other factors and is likely to be in the range 10^{-1} to 10^{-2}. Acceptable annual probabilities of collapse might therefore be in the range 10^{-4} to 10^{-6}.

Given the current state of knowledge of the variables and the inherent uncertainties involved in reliably predicting when a structure will collapse, it is not considered practical to either analyse a building to determine the probability of collapse or base a code verification method around a collapse limit state. It is therefore necessary to adopt a different approach for the purpose of design.

It is possible to consider a limit state at a lower level of structural response, at a level where structural performance is more reliably predicted, and one that is more familiar to designers and then rely on margins inherent within design procedures to provide confidence that acceptable collapse and fatality risks are achieved. In this Standard this limit state is referred to as the ultimate limit state (ULS)."

2 pages further over, clause C2.1 goes on to say:

"It is inherent in this Standard that, in order to ensure an acceptable risk of collapse, there should be a reasonable margin between the performance of material and structural form combinations at the ULS and actions associated with structural failure through loss of strength or stability. For most ductile materials and structure configurations it has been assumed that a margin of at least 1.5 to 1.8 will be available. This is intended to apply to both strength and displacement.

The assumed margin will not necessarily be available in every building, however it is an expectation of this Standard that the risk of the margin falling below that stated will be low.

This Standard compensates for the poorer relative performance of brittle structures and structures of limited ductility compared to that of ductile structures. This is achieved by the specification of different values of S_p. The effect is to raise the design loads for brittle/low ductility structures by a factor of approximately 1.5 compared with those for ductile structures."

As outlined in clause C3.1.4, the margin adopted in the Standard between the ULS design earthquake action and the maximum earthquake action under which collapse is to be avoided (i.e. the collapse avoidance limit state (CALS) event) is a factor of 1.5.

Unpicking All of the Above

- While the NZBC [2] is primarily focused on achieving life safety under ULS events (as defined by AS/NZS 1170.0 [3]), preserving the value of the financial investment in structures and facilities is also an objective. This is reflected in AS/NZS 1170.0 [3] as explained in its commentary clause C3.4 which discusses the adjustments made to the annual exceedance probabilities (AEPs) for design working lives other than 50 years as providing for the safety of the structure as a function of its importance to the community. Some people argue that the risk to human life is independent of the life of the structure. However, the risk to the structure posed by earthquakes is dependent on the duration that the structure is exposed to the risk, its working life.
- If collapse is to be avoided under the CALS event then clearly all of the NZS 1170.5 Standard's [1] ULS performance requirements need to be achieved as CALS objectives. They should be specified as such in the NZS 1170.5 Standard [1].
- While it may not be practical to predict when a structure will collapse with any reliability, it is certainly possible to design to avoid collapse at a defined higher level of earthquake response, i.e. under the CALS event. The author believes this is what should be done, but this is currently not required by NZS 1170.5 [1].
- By extrapolating the NZS 1170.5 Commentary [1] Figure C3.3 graph for the R factor values, the lower limit annual probability of collapse of 10^{-4} corresponds to a Return Period Factor of $R \approx 2.7$. Dividing by 1.5, the adopted margin between the ULS and CALS events, results in the lower limit appropriate Return Period Factor for the ULS being $R = 1.8$ which equates to an annual exceedance probability event of 1/2500, i.e. a design return period of 2500 years, which is far greater than adopted by AS/NZS 1170.0 [3] Table 3.3 for all but the most important and/or longest design working life structures.

  However, recent evaluations of earthquake design standards suggest that the stated value of the order of 10^{-6} for the target annual earthquake fatality risk is too optimistic and that from an evaluation of US standards suggest that the annual probability of collapse provided by modern seismic design codes is around $2x10^{-4}$, i.e. 1% probability in 50 years [30]. This leads to an annual fatality risk of $2x10^{-5}$ to $2x10^{-6}$, i.e. from 2 to 20 times that stated in the NZS 1170.5 Commentary. Applying the same approach as in the preceding paragraph, adopting an annual probability of collapse of $2x10^{-4}$ corresponds to a Return Period factor of $R \approx 2.2$, then dividing by 1.5 results in a Return Period Factor for the ULS of $R \approx 1.46$ which equates to an annual exceedance probability event of about 1/1300, i.e. a design return period of 1300 years. Again this is considerably greater than adopted by AS/NZS 1170.0[3] Table 3.3 for IL2 structures.
- The assumption that in general most structures will be composed of ductile materials and structural configurations tends to ignore the fact that the ground on which the structure is supported will not be ductile. Adjusting the S_p factor upward for brittle/limited ductile structures does not alter the application of a 1.5 margin factor to scale down the CALS action to the ULS design actions. Instead it suggests an inappropriate double counting of the factors used to justify the S_p factor. By focusing on designing for a design ULS event consideration is seldom given to how the performance and behaviour of the structure may change under larger events, e.g. whether the ground may fail or liquefy, whether base isolation devices may exceed their displacement capacities, or whether floor slabs may unseat. It simply cannot be assumed that the assumed margin will exist in the case of displacements. That must be specifically provided for by design. It simply cannot be assumed that the risk of the margin falling below that stated will be low in situations, for example, where ground supporting the building fails under earthquake response exceeding the ULS but is less than the CALS response.

CLASSIFICATION OF STRUCTURES BY IMPORTANCE

AS/NZS 1170.0 [3] Table 3.2 provides a categorisation of building types into Importance Levels. Within Importance Level 3, in particular, while a number of facilities with lower occupancy levels are there because of other factors such as importance of their functions or contents to the community, societal vulnerability of their occupants, or lack of free choice in their occupants being in them, none the less there is perceived to be considerable inconsistency between the examples (a) to (i). For example, a multi-occupancy commercial office building accommodating less than 5000 people, but that can be accommodating a 1000 people or many more can be classified as a normal structure of importance level 2, while numerous other facilities accommodating only 500 or fewer people are accorded an importance level of 3. This surely is inconsistent. Too great an importance has been applied to the nature of the occupancy and not enough to the number of people exposed to the risk.

The importance levels need to be reviewed and revised. As this part of the AS/NZS 1170 Standard [3] does not include the exclusion of structure types presented in NZS 1170.5 [1] this table should also give guidance on the importance to be assigned to a wider range of structures such as bridges, tanks, dams, retaining walls and cut slopes. (NZTA's Bridge Manual [4] has attempted to address some of these but does not have the status of being an NZBC [2] cited document.).

DESIGN HAZARD SPECTRA

Site-Specific Seismic Hazard Studies

Site-specific seismic hazard studies can provide the benefits of a much improved understanding of the actual hazard at the site, and also a deaggregation of the contribution of individual faults to the probabilistic seismic hazard analysis which is a useful input to the geotechnical assessment of a site's stability and liquefaction potential. Apart from the very general requirements for Special Studies given in AS/NZS 1170.0 [3], NZS 1170.5 [1] does not specify any requirements in respect to how site-specific seismic hazard studies are to be undertaken. Some guidance is provided by the NZTA's Bridge Manual [4] but this also falls short of fully specifying the methodology to be applied. Requirements for site-specific seismic hazard studies should be fully specified in NZS 1170.5 [1].

In different locations around New Zealand site-specific seismic hazard spectra can vary markedly from the seismic hazard spectra specified by NZS 1170.5 [1]. This is due in part to the application of approximate R-factors across the country as a whole which vary somewhat from the values appropriate in different regions as the design return period differs from 500 years, as illustrated by NZS 1170.5 Commentary [1] Figure

C3.3. This deviation in the NZS 1170.5 Standard's [1] specified design seismic hazard spectra for varying return periods from what should actually be applicable in different regions of the country, as well as variations in spectral shapes with both return period and location, should be possible to be overcome through the provision on the internet for use by design consultancies of site-specific seismic hazard software implementing GNS Science's National Seismic Hazard Model. GNS Science is currently seeking funding to develop this software.

Maximum Considered Earthquake and National Seismic Hazard Model

NZS 1170.5 [1] has adopted as the upper bound limit on the maximum considered earthquake (MCE) an 8.1 magnitude earthquake event occurring in close proximity, as might be generated by an alpine fault event. Does this still remain the appropriate upper bound for the MCE event ? Deep drilling research recently conducted in Marlborough into subduction zone earthquakes has revealed that the last southern Hikurangi margin subduction earthquake occurred approximately 500 years ago and that the event before that occurred approximately 350 years previously. As noted in the Clark et al, 2015 paper [6], this is a shorter time interval than considered in current seismic hazard models. There has also been the suggestion, still under debate, that part of the 2016 Kaikoura earthquake faulting actually occurred on the subduction zone. (Dominion Post, 15.11.2016 [7]). Past modelling has considered this subduction zone to be capable of generating earthquakes up to magnitude 9.0. This suggests that the National Seismic Hazard Model is likely to be in need of revision and that the upper bound MCE event should be reconsidered.

Comment received from within GNS agrees that revision of the upper-bound MCE motions may be required shortly, but takes the view that these revisions are required not because of increased magnitudes and reduced return periods of some of the large subduction interface events in themselves, but because revised modelling suggests that they may produce stronger ground motions than near-source motions from large crustal earthquakes. These new ground-motion models they consider require further evaluation before they become the basis for potentially vastly increased hazard estimates.

The limitation in NZS 1170.5 [1] clause 3.1.1 that ZR_u should not exceed 0.7 should be discarded. This represents a scaling down of the upper bound MCE by the factor 1/1.5, but for areas of higher seismicity and structures of higher importance this can result in a scaling down of the return period of the ULS design earthquake event which is likely to result in those structures incurring significant damage in events of lesser return period than intended. The upper bound MCE should apply as the upper bound for both the CALS and the ULS.

Deterministic v Probabilistic Earthquake Events as the Basis for Design

NZS 1170.5 [1] seismic hazard spectra, and site-specific seismic hazard studies are based on probabilistic seismic hazard analysis. But perhaps, particularly for higher seismicity regions, greater consideration should also be given to the deterministic events most likely to be highly damaging to a region, giving consideration to when those faults last ruptured and the likelihood of them rupturing again within the design life of the structure. Consider Wellington for example. As reported by Rhoades et al [8], the Wellington fault has an estimated probability of rupture within the next 100 years of ~11%. Figure 1 compares the Wellington fault seismic hazard spectra for the rock site of the Shell Gully bridges with the NZS 1170.5 [1] 500 year return period spectra for a Wellington rock site. As can be seen in the figure the 84th percentile Wellington fault spectra is significantly greater than the Standard's [1] 500 year return period spectra adopted for the design of large CBD commercial buildings, and the Wellington fault's 50th percentile spectra is only exceeded by the Standard's [1] 500 year return period spectra in the limited period range of 0.3 to 0.7 seconds. McVerry and Holden [9] have studied the possible spectra from an 8.4 magnitude Hikurangi subduction zone earthquake and compared that with a Wellington fault event. If the stress drop from the subduction zone event is high then the spectra for the subduction zone event is predicted to exceed that of a Wellington fault event, but given what is said above about the recurrence interval between subduction zone events and the potential magnitude of subduction zone events, such events are likely to have a higher probability of occurring than the Wellington fault event, have potential to be of a higher intensity of shaking, and as damage is caused not only by the intensity of

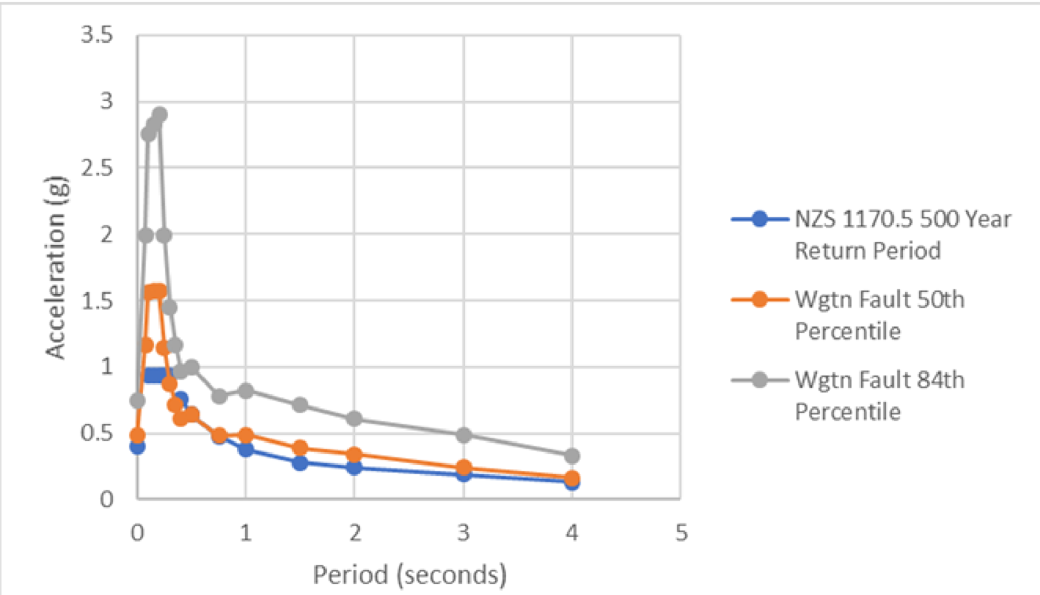

Figure 1: Comparison of Wellington Fault 84th and 50th Percentile Site Hazard Spectra with NZS 1170.5 Specified Spectra for a 500 Year Return Period for Wellington Rock Sites (maximum directivity included as for sites < 2 km from the fault).

shaking but also by the duration of strong shaking, if of larger magnitude will shake for longer doing greater damage. The deterministic subduction zone event has the potential to be a more likely and significant design case that should be considered for the design of Wellington CBD commercial buildings than the current NZS 1170.5 Standard's [1] probabilistically derived 500 year return period design event.

Advice from GNS is that recent estimates of the probabilistic spectra for Wellington are much higher than those given by NZS1170.5, but they are based on much revised models and approaches. This is an issue of lack of updating of the NZS1170.5 hazards section, which is based on models that are now about 20 years old. Whether the design spectra should be probabilistically or deterministically based is a separate issue.

It is also noted that the California Department of Transportation [10] in general adopt as the design response spectrum for bridges the greater of:

- A probabilistic spectrum based on a 5% in 50 years probability of exceedance;
- A deterministic spectrum based on the largest median response resulting from the maximum rupture of any fault in the vicinity of the bridge site.

Structural Performance Factor S_p

The NZS 1170.5 Commentary [1] provides no explanation of the factors considered to contribute to the 1.5 to 1.8 margin of additional capacity expected to be available to withstand the CALS earthquake event. In clause C4.4 it does, however, outline factors considered to justify the application of the structural performance factor, S_p. These are surely the same factors that provide the margin of additional capacity used to justify the factoring down of the CALS event to the ULS event to be used for design purposes. In effect these factors have been double counted in the NZS 1170.5 Standard [1].

Many of these factors can either be directly taken into account in design or, for both building and non-building structures, may not be applicable. This is discussed in the report MacRae et al, "Review of the NZ Building Codes of Practice" [11], excepts from which are included below, together with comments by the author on each of them as they relate to the design of bridges:

1. Calculated loads correspond to the peak acceleration which happens only once and therefore is unlikely to lead to significant damage.

   The writer's view:

   At the CALS response the earthquake shaking will last for much longer than at the ULS level with the likelihood of multiple cycles at or close to the peak level of acceleration. Also design response spectra are derived from earthquake records recorded in two orthogonal directions and do not represent the resultants of earthquake acceleration at any instant in time. The resultant response may be greater.

   MacRae et el [11] also comment:

   "This is based on the idea that inelastic cycles will result in strength and stiffness deterioration. Therefore, if the maximum magnitude of the cycle occurs more than once there will be more damage, and hence a greater probability of failure, than if it occurs only once. This is more likely to be true for reinforced concrete structures that are designed for ductility, than for modern steel structures, which tend to have a very low degradation in strength with cyclic loading.

   Future large earthquakes, such as those anticipated from the NZ Alpine Fault, are expected to result in many cycles of shaking for a long duration, which would likely cause a number of cycles at very high displacement therefore invalidating the concept about the number of displacements being important"

2. Individual structural elements are typically stronger than predicted by analysis (higher material strength, strain hardening, strain rate effects)

   The writer's view:

   Based on Priestley et al [12], the methods for design specified in the recently revised Section 5 of NZTA's Bridge Manual [4] are based on using probable material strengths, not lower characteristic material strengths as has been past practice, and by use of moment-curvature analysis strain hardening is also able to be taken into account.

   MacRae et al [11] also comment:

   "In some cases the members will be significantly stronger than that needed in design. However, they may also be weaker than the nominal or "ideal" values. For this reason we use a strength reduction factor, φ. Higher member strength cannot be used as justification for a S_p factor less than unity."

3. The total structural capacity is typically higher than predicted (redundancy, non-structural elements)

   The writer's view:

   Bridge structures generally have little or no redundancy and no structural elements that will contribute additional strength. Examples of other structures also lacking redundancy include car parking buildings and chair lift pylons.

   MacRae et al [11] also comment:

   "The non-structural elements, slabs, etc. may also have been beneficial in reducing some of the demands on the main seismic resisting frame, even if, in some cases, they sustained significant damage. This may justify the use of the S_p factor in much existing construction.

   However, buildings may be designed according to the present code in which non-structural elements do not significantly contribute to the seismic response, and in which the effects of other elements (such as the floors and gravity systems) are explicitly included in the strength/stiffness. Such systems may be parking structures with no non-structural elements, or newer low damage systems in which non-structural elements are either seismically separated or included explicitly as part of the structural system.

   In frames in which the non-structural elements, slabs, and gravity systems are separated, or explicitly included from the structural calculations, this argument about the presence of non-structural elements alone is insufficient to justify an S_p factor less than unity. It should be noted that buildings of this type are permitted according to current standards, and that they are likely to become more popular as a part of the "low structural damage" proposals for new construction, which are currently being proposed for reconstruction in Christchurch"

4. The energy dissipation of structure is typically higher than assumed (damping from non-structural elements and foundations)

   The writer's view:

   The design methods specified in the recently revised Section 5 of the Bridge Manual [4] directly take into account foundation damping, and bridges possess no relevant non-structural elements able to provide additional damping.

MacRae et al [11] also comment:

"Ductile buildings may be built on a variety of foundation types and consequently radiation damping will not decrease the response of all buildings in the same way. If a higher value of soil damping is already included in the analysis then there is some "double dipping" in this justification for S_p. It seems that the appropriate place to consider foundation damping is in a specific factor for this, rather than in the S_p factor. "

Application of the S_p factor has the effect of reducing the earthquake return period being designed for at the ULS. So, for example, for a commercial building assigned an importance level of 2 and a corresponding annual exceedance probability of 1/500, application of an S_p factor of 0.7 effectively reduces the earthquake design return period from 500 years to 200 years, an event that has a significantly greater probability of occurring in the nominal 50 year design life of the IL2 building. (22% if treated as being a normally distributed random event.)

The structural performance factor, S_p, should be discarded.

ANALYSIS METHODS

Force-Based Design

NZS 1170.5 [1] and the materials design standards (e.g. NZS 3101, NZS 3404) are currently based on the use of force-based design.

Differently to displacement-based design, force-based design utilises the initial tangent stiffness of the structure as the basis for determining the period of response of the structure for determination of the seismic acceleration to be applied to the structure.

In the NZS 1170.5 [1] design approach using either the equivalent static force method or the modal response spectrum method, as outlined in clause 5.2.1.1, the design seismic hazard spectra is scaled down by dividing by a factor k_μ and by multiplying by the S_p factor, to derive the design seismic response spectra. The factor k_μ is a function of the assumed ductility factor, μ. As noted above, scaling the response spectra down by S_p effectively reduces the design return period of the earthquake event that the ULS design seismic response spectra and displacement relates to. Then, as outlined in clause 7.2.1.1(a), and leaving aside the alternative side sway mechanism deflections approach and P-delta effects, ULS displacements are derived by factoring by the assumed ductility factor μ the displacements determined from an analysis in which the design seismic response spectra has been applied to the structural model.

For structures with longer fundamental periods, following the equal displacement assumption, $k_\mu = \mu$, so, recognizing that the scaling by S_p is effectively a scaling down of the design return period, then otherwise the scaling down for forces and up again for displacements is the same and the ULS displacements are independent of the level of ductility, μ.

For structures with short fundamental periods, i.e. between T=0 and for firm soils T<0.7 seconds, or for Class E sites T<0 seconds, there is a transition zone in which the equal displacement concept does not apply. At T=0 the displacement is zero and ductility has no effect on reducing seismic force. This transition zone is handled by the k_u factor where k_μ is less than μ. For short fundamental period structures analysed by the equivalent static forces or modal response spectrum methods, the relationship between the structural ductility factor μ and k_μ for the different site subsoil classes has been determined from many studies undertaken at the University of Canterbury utilizing inelastic time history analyses.

The NZS 1170.5 Commentary [1] provides no explanation, for the short period structures, of why the scaling factor used to scale down the seismic response from the seismic hazard spectra is different to the scaling factor used to scale up the displacements derived from the analysis. The basis for the differences in the scaling factors needs to be explained in the Commentary [1].

It is also perceived that commonly designers fail to confirm that the structural ductility factor assumed as the basis for their design will actually be realised, and if not confirmed, then often it may not be realised.

Displacement-Based Design

Displacement-based design utilises the secant stiffness of the structure at maximum response as the basis for deriving the period of response of the structure. This method is considered to provide a more realistic assessment of the effective stiffness, displacement, and ductility demand at the level of maximum response.

Displacement-based design based on using the inelastic secant stiffness, as promoted by Professor Nigel Priestley, has gained some acceptance internationally being taught at the ROSE School, Istituto Universitario di Studi Superiori di Pavia in Italy and is presented as an alternative design method in the Australian Standard: AS 5100.2 Bridge Design, Part 2: Design Loads [12]. It is also permitted by the NZTA's Bridge Manual [4]. Revision of NZS 1170.5 [1] and the New Zealand materials design standards is required to permit and accommodate the use of displacement-based design. The method is already well documented in such publications as Priestley M.J.N., Calvi G.M. & Kowalsky M.J, 'Displacement-Based Seismic Design of Structures" [13] and Priestley M.J.N., Seible F., & Calvi G.M., "Seismic Design and Retrofit of Bridges" [14], and Sullivan T.J., Priestley M.J.N. & Calvi G.M. (Editors) "A Model Code for the Displacement-Based Seismic Design of Structures" (DBD12) [15].

Amendments to Traditional Design Approaches

Documented in the first of the Priestley et al publications [13] listed above is a proposal for probable material strengths to be used as the basis for the design of yielding elements and that gravity and permanent load actions do not need to be combined with seismic actions in the design of these elements. There are some additional considerations that need to be taken into account in applying this approach as discussed below. A method for taking into account foundation damping is also described. These are applicable to both the force-based and displacement-based methods of design.

Elements yielding flexurally under the action of seismic moments will experience a relieving of gravity and permanent load moments acting in these elements with a resultant redistribution of the gravity and permanent load actions in the structure. Hence combining the seismic moments acting on the yielding elements with gravity and permanent load moments in the design of the yielding elements is not necessary. However, the following conditions need to be taken into account:

- During the earthquake response and in the post-earthquake condition of the structure, the redistribution of gravity and permanent load actions will result in increased positive moments now acting in the midspan region of beams that are plastic hinging at their supporting columns. This needs to be considered in the design of these beams both for their strength at the ULS and for their serviceability at the SLS. It is believed that this has not traditionally been done, and that it should therefore be specified to be required by NZS 1170.5 [1]. This sequence of events giving rise to the relieving of permanent load moments is illustrated in Figure 2.

- The stability of the structure must be maintained and ratcheting avoided during and after the earthquake. The stability of beams cantilevering off portal frames as the extension of beams that are plastic hinging must be maintained by their cantilever moment being entirely reacted by their supporting columns. In structures with unbalanced lateral strengths and /or eccentric gravity loading causing ratcheting, the requirements of NZS 1170.5 clause 4.5.3 [1] need to be complied with to counteract the tendency for ratcheting to occur.
- Prior to the earthquake, during SLS earthquake response, and following the earthquake if the structure is expected to be repaired and returned to service, the design of the structure must also satisfy serviceability limit state requirements. Redistribution of gravity load moments is not permitted in the serviceability limit state unless a detailed study is undertaken to ensure a stable shake down situation arises and the deformations associated with this state do not conflict with serviceability requirements.

Actions and support conditions acting on the structure can change as seismic response is increased from the ULS up to the CALS. In order to avoid the collapse of structures in events approaching the magnitude of the CALS these changes in actions and/or support conditions must be considered. Such changes may arise for example, due to softening or liquefaction or slope failure of the structure's supporting ground.

DISPLACEMENTS

In order to avoid the collapse of structures and of parts of structures at the CALS there is a need to consider stability, displacements, inter-storey drifts and frame elongations all at the CALS. All the following should be considered:

- Base isolation devices need to be designed to remain stable and maintain their integrity and functionality at the CALS displacements.
- Limited application of displacement-based design to date has revealed that CALS displacements determined by that method may significantly exceed $1.5/S_p$ x ULS force-based design displacements. With factoring up of the design displacement from the ULS to the CALS event the secant stiffness of the structure is altered resulting in a period shift effect in addition to the factoring up.
- In CALS events the duration of strong shaking will be longer. In ductile structures, the effect of stiffness softening with repeated cycles of load can be expected to increase displacements.
- In elastic structures, the effect of earthquake energy not dissipated by damping being stored and then returned to the structural system with repeated cycles of loading can be expected to increase displacements. Appropriate MCE level records need to be developed for response spectra so that displacements generated by energy not dissipated by damping are considered.

These last two bullet point items are likely to require the application of time-history analysis, and in the first case incorporation of modelling of structural stiffness softening, using earthquake time-history records appropriate to CALS events in order to enable these effects to be assessed. It is noted that the NZBC [2] excluded time-history analysis from the verification method B1/VM1 under which NZS 1170.5 [1] is cited, and placed caveats on its use. However, it is believed that the method is being routinely used by some design offices.

REPAIRABILITY

The huge number of buildings demolished in Christchurch following the 2011 Canterbury earthquakes, and the significant number of buildings in Wellington, some already demolished and a good many more that are expected to be demolished following the 2016 Kaikoura earthquake, calls into question whether current design practices have resulted in structures that are not readily amenable to being repaired. Insurance policies that have provided to replace or reinstate buildings to their pre-earthquake condition is believed to have promoted this high level of building replacement, perhaps unjustifiably.

As noted in the paper: Russell, 2013 [16], plastic hinging in reinforced concrete buildings results in irrecoverable plastic deformation of the reinforcing steel when the reinforcing steel is strained beyond its elastic limit. This reduces the capacity of the reinforcing steel for further inelastic straining before rupture will occur in future earthquake events. The paper discusses the practicality of repair and how in many situations the option to replace rather than repair the structure has, or is being, taken. However, it has been suggested that substantial strain hardening is needed to have occurred before demolition and replacement is really justified.

Where steel is yielded in both tension and compression in plastic yielding elements a degree of low cycle fatigue may also be induced which again is an irrecoverable form of damage.

While amending the nature of insurance policies is one way of overcoming the high level of building replacement being incurred another way is to place much more focus on designing structures in a manner such that plastically yielding elements can be removed and replaced. Where the conditions are suitable for their use, greater use of base isolation devices incorporating mechanical energy dissipation are an option that can be considered. For bridges, allowing piers to rock on their pile caps or spread footing foundations, with mechanical energy dissipating devices arranged around the base of the pier, and a

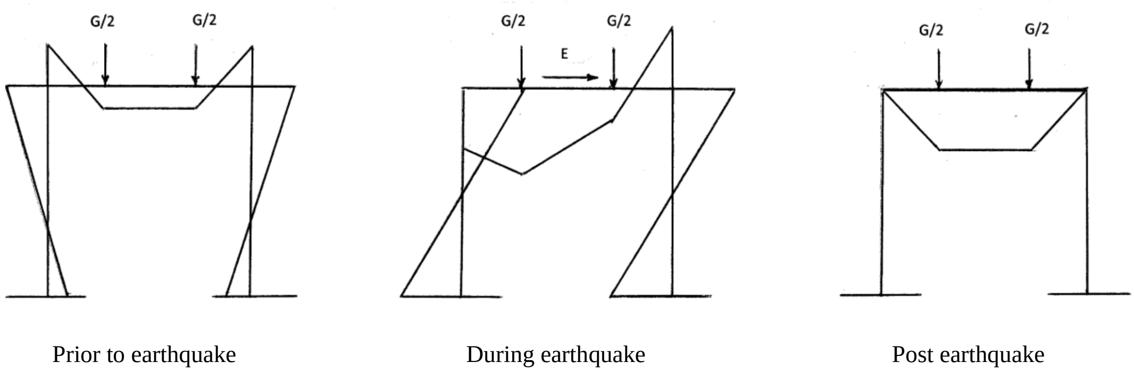

Figure 2: The relieving of member permanent load moments by earthquake induced at plastic hinge locations in a portal frame.

restoring force to return the pier to vertical provided by a prestressing cable down through the centre of the pier, could be another option. In recent times government project procurement approaches appear to have discouraged such approaches to earthquake resistant design. From 1973 up until about 1998 ~50 base isolated bridges were built in New Zealand, but since then only one further base isolated bridge has been constructed that is known of. That needs to change. Allowing building frames to rock could even be a possibility though there are many challenges that require careful consideration in the design.

MATERIALS

Capacity design needs the maximum strength of yielding materials to be limited so that non-yielding elements can be designed to possess sufficient strength to withstand the maximum forces induced in them by earthquake response that mobilises the maximum strength of the yielding elements. Structural steel materials standards fail to impose upper bounds on the ultimate tensile strengths of the standard steel grades that are produced. This should be done in a similar manner to that done in AS/NZS 4671 [17] for seismic grade reinforcing bar steels.

NZS 3404: 1996 [18], NZS 3404.1: 2009 [19], and AS/NZS 5100.6 [20] all require structural steel to be brittle fracture resistant. This requirement is even more critical for steel elements subjected to yielding under seismic actions for which more stringent limits are applied to the allowable service temperature. AS/NZS 4671 [17] specifies no similar requirement for reinforcing steel to be brittle fracture resistant, but surely it should. NZS 3101 [21] has moved to require that reinforcing bar mechanical anchorages and couplers be brittle fracture resistant but has neglected to require that the reinforcing bars themselves be brittle fracture resistant except in areas where the ends of the bars have been processed in some manner to effect the coupling. Brittle fracture resistance is usually assessed by use of Charpy impact testing. The NZS 3404 Commentary Clause C2.6 [18] provides guidance on the assessment of brittle fracture resistance using this method which is not entirely clear. However, Reid Construction Systems Product Catalogue & Design Guide 07/08 [22] states “Recent tests have shown values of Charpy impact resist for Grade 500E RB 32 at -15ºC at around 17 joules.” This is a sufficiently low value that it can be concluded that Reid bar is not sufficiently brittle fracture resistant for unrestricted use at any location in New Zealand. It also calls into question the brittle fracture resistance of grade 500E reinforcing steel in general, a product currently the subject of unresolved issues in respect to its quality drawn to MBIE’s attention and awaiting their action.

Charpy impact testing is considered by some experienced in such testing to provide a poor correlation to brittle fracture resistance. As a test method it is also impractical to apply to the testing of small elements such as the couplers and anchorages for the smaller reinforcing bar sizes. An alternative method employing high speed tensile testing has been proposed that would be capable of testing reinforcing bar coupler assemblies, but testing is required to prove the method. A proposal to undertake such testing has been prepared and submitted to both MBIE and to Callaghan Innovation requesting funding but no response has been received back.

In the construction marketplace there are many proprietary products promoted and in use for which the quality assurance is perceived likely to be highly questionable. Policing the quality assurance of such products is surely an MBIE responsibility but are they exercising that responsibility ? Examples of such products that should be subject to scrutiny include: reinforcing bars, couplers and anchors for reinforcing steel, fibre reinforced polymer products, prestressing systems, geotextiles for soil reinforcement, and proprietary retaining wall systems.

SITE CONDITIONS

The design of structures requires a holistic approach to be taken giving consideration to all aspects of the structure including its supporting ground and including all actions that its use and environment may impose on it.

In NZS 1170.5 [1] there is a complete lack of consideration of foundation integrity, stability and liquefaction potential. This is a very important aspect. For example: In Wellington we have new multi-storey commercial buildings recently built on our waterfront now being demolished due to damage from the Kaikoura earthquake, and another new building currently under construction. The ground at the sites of these building is site soil class D as reported in the paper Semmens et al, 2011 [23]. What will happen at these sites and many more around the Wellington waterfront when a CALS event occurs locally? Experience from large earthquakes around the world suggest that the waterfront area will be extensively damaged with the poor ground probably losing stability and slumping into the harbour taking buildings founded on it with it.

Guidance on the consideration of site geotechnical conditions can be found in the NZTA’s Bridge Manual [4] and in design guides produced by the New Zealand Geotechnical Society.

Many coastal areas around New Zealand are highly developed with new structures continuing to be built and yet there are no requirements in NZS 1170.5 [1] for the consideration of tsunami which frequently accompanies earthquakes.

COMPARABILITY WITH INTERNATIONAL CODES

Fenwick et al, (2002), [24] reports on a study in which a series of ductile moment resisting reinforced concrete frames were sized to meet the minimum seismic provisions of the New Zealand Loadings Standard NZS 4203: 1992 [25], the draft NZ /Australian Loadings Standard [26], the Uniform Building Code IBC-1997 [27], the International Building Code IBC 2000 (1998 draft) [28], and Eurocode 8 (1998 draft) [29], based on being located in both high and low regions of seismicity. Comparative analyses were made of these buildings and these showed that the strength and stiffness requirements of both NZS 4203 [25] and the draft Standard [26] that has subsequently become NZS 1170.5 [1] were low compared with the other Standards in the high seismic zone. While this study is somewhat dated it is believed that this remains the case.

CONCLUSION

This paper outlines many areas considered to be deficiencies in New Zealand’s practice of earthquake resistant design. In response to the questions posed in the Introduction:

- In both the NZBC [2] and NZS 1170.5 [1] performance objectives need to be revised and appropriately expressed in terms of the limit states at which they are targeted, including the collapse avoidance limit state.
- There are inconsistencies in the existing importance level classifications of buildings. The classification of large high rise commercial buildings capable of accommodating large numbers of occupants as importance level 2 is a particular example of an inappropriate classification.
- The existing specified design seismic hazard spectra are out of date and require revision. The 2010 seismic hazard model should be reviewed and revised to bring it up to date with more recent research findings. Requirements for the performance of site specific seismic hazard studies need to be incorporated into NZS 1170.5 [1] to ensure consistency

in how these are undertaken. In the high seismicity areas of NZ it is suggested that greater consideration and weight needs to be given to the deterministic events likely to be highly damaging to the region.

- The structural performance factor, S_p, should be discarded. It amounts to a double counting of the factors justifying the scaling down of the seismic hazard from the collapse avoidance limit state to the ULS for the purpose of design, and in effect it scales down the return period being designed for at the ULS away from that specified by AS/NZS 1170.0 Table 3.3 [3].
- Displacement-based design is being utilised by some designers as an alternative design method to force-based design. It allows the ductility demand to be directly calculated in the design process instead of being assumed and can result in greater displacements being assessed than predicted by force-based design. Revision of NZS 1170.5 [1] and the materials design standards is required to accommodate displacement-based design if the method is to be permitted.
- The formation of plastic hinging in members under seismic response will also relieve the member fixed end moments due to permanent actions resulting in redistribution of the moments due to permanent actions within the structure which need to be designed for.
- Displacements need to be considered at the CALS. These include the following effects:
    - The capacity of base isolation devices to maintain their stability, integrity, and functionality;
    - At the CALS the longer duration of strong shaking can be expected to result in stiffness softening in ductile structures increasing their displacement above that predicted;
    - For elastically responding structures, appropriate MCE level records need to be developed for response spectra so that displacements generated by energy not dissipated by damping are considered.
- Recent earthquakes have resulted in the loss of a huge number of structures calling into question the adequacy of the ductile design approach. A shift in design approach is needed to designing to minimise damage, e.g. through adoption of base isolation, or by making inelastically deforming elements able to be readily inspected and replaceable.
- Amendments are required to the Standards for structural steel to impose an upper bound limit on the ultimate tensile strength of each commonly used grade of steel to facilitate reliable capacity design, and to the Standard for reinforcing steel [17] to incorporate a requirement for the steel to be brittle fracture resistant.
- The quality of proprietary products in use in the construction industry needs to be assured and the responsibility for this surely rests with the Ministry of Business, Innovation and Employment. The unresolved issue with the quality of reinforcing steel being supplied to the market urgently needs resolution.
- Site conditions must be considered as an integral component of the design of structures and the building of structures in inappropriate locations such as on waterfront soft ground should be avoided. Tsunamis can accompany earthquakes and that risk too should be considered for structures sited in coastal locations.
- Based on a 2002 study, the strength and stiffness requirements for buildings in high seismicity regions of NZS 1170.5 [1] are believed to be low compared to other major international codes.

It is the author's hope that this paper will stimulate discussion among structural design professionals and action that results in these issues being addressed. Action that at the present time there appears to be too little of.

ACKNOWLEDGEMENTS

Grateful thanks is expressed to those who kindly gave of their time and expertise to review this paper and provide feedback, namely: Phil Gaby, Associate Professor Greg MacRae and Professor Des Bull. I would also like to acknowledge and thank NZSEE's reviewers, Professor Richard Fenwick and Dr Graeme McVerry for their comments and advice.

REFERENCES

1. Standards New Zealand (2004). *"NZS 1170.5 Structural Design Actions, Part 5: Earthquake Actions – New Zealand and Associated Commentary (Including Amendment 1)"*. Standards New Zealand, Wellington, NZ, 81pp & 104pp.
2. Ministry of Business, Innovation and Employment (1992). "*New Zealand Building Code*: *Clause B1 Structure (Including Subsequent Amendments)*". MBIE, Wellington, New Zealand, 87pp.
3. Standards New Zealand (2002). "AS/NZS 1170.0 Structural Design Actions, Part 0 – General Principals and associated Commentary (Including Amendments 1, 2 & 4)". Standards New Zealand, Wellington, New Zealand, 34pp & 31pp.
4. New Zealand Transport Agency (2013). *"Bridge Manual Third Edition, (Including Amendments 1, 2 and 3)*". NZTA, Wellington, New Zealand.
5. International Organisation for Standardisation (1998). *"ISO 2394 General Principles on the Reliability for Structures"*. Geneva, Switzerland, 79pp.
6. Clark KJ, Hayward BW, Cochran UA, Wallace LM, Power WL and Sabaa AT (2015). "Evidence for Past Subduction Earthquakes at a Plate Boundary with Widespread Upper Plate Faulting: Southern Hikurangi Margin, New Zealand". *Bulletin of the Seismological Society of America,* **105**(3): 30pp.
7. Gorman P (2016). "*Quake Stresses Other South Island Faults*". The Dominion Post, 15 November 2016.
8. Rhoades DA, Van Dissen RJ, Langridge RM, Little TA, Ninis D, Smith EGC and Robinson R (2010). "It's Our Fault: Re-Evaluation of Wellington Fault Conditional Probability of Rupture". *Proceedings of the 2010 NZSEE Technical Conference,* Wellington, New Zealand, 26-28 March, Paper No. 23, 8pp.
9. McVerry GH and Holden C (2014). "*A Modified Ground-Motion Prediction Equation to Accommodate Spectra of Simulated Hikurangi Subduction Interface Motions for Wellington*". Consultancy Report 2014/131, GNS Science, Lower Hutt, New Zealand, 36pp.
10. Caltrans (2013). "*Caltrans Seismic Design Criteria"*. Version 1.7, California Department of Transportation, Sacramento, USA, 179pp.
11. MacRae G, Clifton C and Megget L (2011). "*Review of NZ Building Codes of Practice"*. Report to the Royal Commission of Inquiry into the Building Failure Caused by the Christchurch Earthquakes, ENG.ACA.0016.1, 57pp.
12. Standards Australia (2017). *"AS 5100.2:2017 Bridge Design, Part 2: Design Loads"*. Sydney, Australia, 132pp.
13. Priestley MJN, Calvi GM and Kowalsky MJ (2007). *"Displacement-Based Seismic Design of Structures"*. IUSS Press, Istituto Universitario di Studi Superiori di Pavio, Italy, 738pp.

14. Priestley MJN, Seible F and Calvi GM (1996). "*Seismic Design and Retrofit of Bridges". John Wiley & Sons Inc., New York, USA, 701pp.*
15. Sullivan TJ, Priestley MJN and Calvi GM (2012). *"A Model Code for the Displacement-Based Seismic Design of Structures"*. (DBD12), IUSS Press, Istituto Universitario di Studi Superiori di Pavio, Italy. 121pp.
16. Russell AP (2013). "Strain Hardening of Reinforcement in Concrete Buildings during Earthquakes". *Proceedings of the New Zealand Concrete Industry Conference*, Queenstown, New Zealand, 3-5 October 2013, 10pp.
17. Standards New Zealand (2001). *"AS/NZS 4671:2001 Steel Reinforcing Materials (Including Amendment 1)*". Standards New Zealand, Wellington, New Zealand, 43pp.
18. Standards New Zealand (1997). *"NZS 3404:1997 Steel Structures Standard and Associated Commentary (Including Amendments 1 & 2)*". Standards New Zealand, Wellington, New Zealand, 396pp & 283pp.
19. Standards New Zealand (2009). *"NZS 3404.1:2009 Steel Structures Standard, Part 1: Materials, Fabrication and Construction"*. Standards New Zealand, Wellington, New Zealand, 158pp.
20. Standards New Zealand (2017). *"AS/NZS 5100.6:2017 Bridge Design, Part 6: Steel and Composite Construction"*. Standards New Zealand, Wellington, New Zealand, 331pp.
21. Standards New Zealand (2006). *"NZS 3101:2006 Concrete Structures Standard and Associated Commentary (Including Amendments 1, 2 & 3)"*. Standards New Zealand, Wellington, New Zealand, 305pp & 414pp.
22. Reid Construction Systems (2007). *"Product Catalogue & Design Guides 07/08"*. 331pp.
23. Semmens S, Perrin ND, Dellow G and Van Dissen R (2011). "NZS 1170.5:2004 Site Subsoil Classification of Wellington City". *Proceedings of the Ninth Pacific Conference on Earthquake Engineering*, Auckland, New Zealand, 14-16 April 2011, Paper Number 007, 8pp.
24. Fenwick R, Lau D and Davidson B (2002). "A Comparison of the Seismic Design Requirements in the New Zealand Loadings Standard with Other Major Design Codes". *Bulletin of the New Zealand Society for Earthquake Engineering*, **35**(3): 190–203.
25. Standards New Zealand (1992). *"NZS 4203:1992 Code of Practice for General Structural Design and Design Loadings for Buildings and Associated Commentary (Including Amendment 1)*, Standards New Zealand, Wellington, New Zealand, 134pp & 95pp.
26. Standards New Zealand (2002). "*Draft Revision of AS 1170.4 and NZS 4203:1992*". Standards New Zealand, Wellington, New Zealand.
27. ICBO (1997). "*Uniform Building Code 97: Vol. 2*". International Conference of Building Officials, Whittier, California.
28. BOCA and ICBO (1998*). "International Building Code, Section 16 and 19"*. July 1998 Final Draft, International Conference of Building Officials, Whittier, California.
29. Eurocode 8 (1998). "*Design Provisions for Earthquake Resistant Structures: Parts 1.1, 1.2 & 1.3*". Draft for Development, British Standards Institute.
30. Building Seismic Safety Council (2015). "*NEHRP Recommended Seismic Provisions for New Buildings and Other Structures, FEMA P-750*". Washington, DC, 388 pp. https://www.fema.gov/media-library-data/1440422982611-3b5aa529affd883a41fbdc89c5ddb7d3/fema_p-1050-1.pdf