COMPARISON OF NEW ZEALAND STANDARDS USED FOR SEISMIC DESIGN OF CONCRETE BUILDINGS

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ABSTRACT

Major changes have occurred over the last six decades in New Zealand design codes for seismic resistance of structures. This paper describes the changes in the required design strengths, stiffness levels and capacity design provisions with particular reference to buildings where the lateral force resistance is provided by reinforced concrete moment resisting frames. It is shown that simple comparisons of response spectra and limiting inter-storey drifts can give misleading conclusions regarding relative strength and stiffness requirements unless allowance is made for many other interacting factors. To illustrate this, minimum design requirements defined in codes (or standards) over the last six decades are compared with the corresponding 2009 design requirements for regular buildings in which the lateral force resistance is provided by moment resisting frames. The approach that is described can be applied to other forms of structure. The paper is intended to provide background information for engineers planning to assess the need for seismic retrofit of existing buildings and to show the different factors which should to be considered in assessing existing structures against current design criteria.

1.0 INTRODUCTION

In the initial stages of assessing the need for retrofit of an existing building it is useful to know how the design requirements existing at the time the building was designed compare with current provisions.

In seeking to compare the requirements of previous codes/standards with current standards the following points should be considered;

1. What factors should be considered when comparing codes?
2. How much effect do these factors have?
3. Do previous code comparisons reported in the literature make sense?
4. Are there any surprises found while making these comparisons?

Answers to these questions can help in identifying aspects that need to be considered in depth and those that may not. The intention behind writing this paper is to provide some background to this problem and in particular to indicate how allowance should be made for many different interacting factors in making comparisons between previous and current design requirements. For example simple comparisons between inter-storey drift limits and strength requirements in different decades can give misleading conclusions unless allowance is made for additional factors such as;

- The way in which section properties were assessed (cracked or gross sections);
- The method of section design, that is working stress or ultimate strength design;
- Allowance made for inelastic deformation;
- Allowance for P-delta actions;
- The modification factors between equivalent static and modal analysis;
- The drift modification factor;
- Changes in strength reduction factors;
- Introduction and subsequent changes in capacity design provisions;
- Changes in sections used to calculate nominal strength and over-strength moment capacities.

It is assumed that the reader is familiar with current 2009 New Zealand design standards, “Earthquake Actions” [1] and the “Structural Concrete Standard” [2]. The approach that is followed is to relate the content of codes or standards in previous decades to the two current design standards (2009). Brief descriptions of previous design codes/standards are given to show when different concepts were introduced. To illustrate how seismic design provisions have changed over the years the minimum strength and stiffness requirements of regular buildings, in which moment resisting frames provide the lateral force resistance designed to previous standards, are compared to current (2009) requirements. The approach can be readily applied to other structural forms, such as buildings where structural walls resist a significant proportion of the lateral forces. To keep the paper as simple as possible the numerical comparisons of different codes/standards have been based on the equivalent static method. It should be noted that space and time do not permit comprehensive comparisons to be made of detailing requirements and only major features are noted. Prior to the 1980s the upper characteristic yield strength of reinforcement and its strain hardening characteristics were

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not as clearly defined in standards for reinforcement properties as is the case today. In a limited number of cases this could influence the hierarchy of failure and hence limit the overall ductility of a structure.

To allow comparisons to be made with loadings and concrete standards from previous decades to current standards they have been grouped as listed below. This grouping does not cover all potential combinations. The groups are;

Group A: Current 2009 standards, NZS 1170.5 and NZS 3101:2006;

Group B: NZS 4203; 1992 and NZS 3101: 1995;

Group C: NZS 4203: 1984 and NZS 3101: 1982;

Group D: NZS 4203: 1976 and ACI 318: 1971 or provisional NZ Concrete Structures Standard;

Group E: NZSS 1900, Basic Design Loads Chapter 8 (1965), and Design and Construction, Concrete, Chapter 9.3 (1964);

Group F: NZSS 95, Pt. IV, Basic loads to be used and methods of application, and then current UK concrete code of practice, CP114: 1957.

2.0 MAJOR HISTORICAL CHANGES IN LOADINGS CODES AND STANDARDS

Major changes over the past six decades include:

i) Ultimate strength design replaced working stress methods for the design of individual structural components (such as beams, columns, walls etc.) in the early 1970s;

ii) Capacity design approaches were initially introduced in the 1970s and their scope has been increased and modified in all subsequent decades;

iii) Prior to the early 1970s, reinforced concrete section properties used for analysis were generally based on gross section dimensions. In later years the stiffness was reduced to allow for flexural cracking with the magnitude of this reduction changing over time as knowledge increased and with the introduction of higher grade reinforcement;

iv) Starting in the 1980s allowance was introduced for P-delta actions. However, the approach was changed in subsequent decades.

In the following sections a more detailed description is given of the significant changes in the Standards relating to seismic design. This includes design forces, stiffness requirements and capacity design provisions as they relate to the design of reinforced concrete moment resisting frame buildings of ordinary importance, level 2 in terms of AS/NZS 1170.0. Many details are omitted due to time and space limitations.

The Loadings code of 1955 [3] contained just two pages that specifically related to seismic design and the concrete codes of that decade did not contain any detailing requirements specific to seismic design. In 2004 the current "Earthquake Actions Standard [1]" has 74 pages specifically related to seismic design with a further 79 pages of commentary. The current "Structural Concrete Standard" [2] has approximately 65 pages specifically on seismic design with a further 120 pages of commentary. This reflects the major change that has occurred during the period!

3.0 Development of seismic provisions in loadings and structural concrete standards

In this section more detailed descriptions are given of the significant content of standards relating to seismic design, including design forces, stiffness requirements and capacity design provisions, as related to reinforced concrete moment resisting frame buildings.

3.1 Pre-1955 Seismic Design Codes

The first set of model building by-laws was published in 1935 [4]. As a result of a number of major earthquakes in the late 1920s and early 1930s the first provisions for seismic design were included in this document. Ordinary buildings were designed to sustain lateral forces of 0.08 times their weight at each level and for major public buildings this coefficient was increased to 0.10. Working stress design was used at the time and consequently the equivalent ultimate strength seismic lateral forces would be appreciable higher than the working stress design force levels.

3.2 Group F, mid 1950s to mid 1960s; NZSS 95, Pt. IV, Basic loads and CP114 (UK), 1957

The seismic provisions in the 1955 Loading Standard, "NZS 95 Pt. IV, Basic loads to be used in Design and their Methods of Application" [3] were similar to those in the 1935 document [4]. No limits were set for seismic displacements or inter-storey drifts. For buildings of ordinary importance the specified lateral seismic design actions were taken as the more critical of the actions found by applying lateral forces equal to 0.08 of the weight at each level or by using lateral force coefficients, which varied linearly with height from 0.12 at the uppermost level to zero at ground level. Members were proportioned using working stress design.

To interpret these values in terms of current design standards allowance has to be made for the change from working stress design to ultimate strength design. The strength ratio between ultimate strength and working stress design for the typical flexural strength of beams can be assessed allowing for the ratio of working stress in reinforcement to yield stress, the typical difference in internal lever-arms and the strength reduction factor used with ultimate strength design. For beams with working stress design the reinforcement stress was limited to 55% of the yield stress, which was generally multiplied by 1.25 for earthquake and wind forces. The internal lever-arm varied with reinforcement ratio but a typical value was 7/8 of the effective depth. With ultimate strength design a strength reduction factor of 0.85 is currently used (2009) together with the design yield stress in the reinforcement and an internal lever-arm that is typically between 0.9 and 0.95 of the effective depth. Using these values current standard design flexural strength corresponds approximately to 1.3 times the working stress moment. For columns the ratio would have been more variable, but generally higher than 1.3. Based on the strength of beams the lateral force coefficient of $0.08W_t$ for normal buildings corresponds approximately to $0.104W_t$ in terms of current practice using ultimate strength theory.

A number of different concrete codes of practice were used in the 1950s and early 1960s. In the universities the British code CP 114 (1957) [5] was used and it is likely that it was also used in practice. These codes generally allowed section properties of members to be based on either gross sections, transformed un-cracked sections, or transformed cracked sections. It is likely that in seismic analyses gross section properties would have been used, as this was the simplest option and transformed section properties could not be calculated until the members had been detailed. Approximate hand methods of analysis would have been used to find design actions. The structural concrete codes of the time did not contain any detailing requirements specifically related to seismic design.

3.3 Group E, mid 1960s and early 1970s; NZSS 1900, Basic Design Loads Chapter 8 (1965), and Design and Construction, Concrete, Chapter 9.3 (1964)

Seismic design forces were given in NZS 1900, Chapter 8, "Basic Design Loads", 1965 [6]. Lateral force coefficients were specified for three different seismic zones. The highest was Zone A, the lowest was Zone C and Zone B was an intermediate zone. The lateral force coefficient varied with fundamental period, T, as shown in Figure 1. The fundamental period was to be calculated from substantiated data (empirical equations) or by calculation. However, it was not to exceed by more than 20 percent a value given by Equation 1;

$$T = 0.32\sqrt{\Delta} \tag{1}$$

where the period T was in seconds, Δ was the lateral displacement of the top of the building in inches, calculated assuming elastic response when the building was subjected to lateral forces, which were found by multiplying the seismic weight at each level by a coefficient that varied linearly with height from unity at the top level to zero at the base of the building.

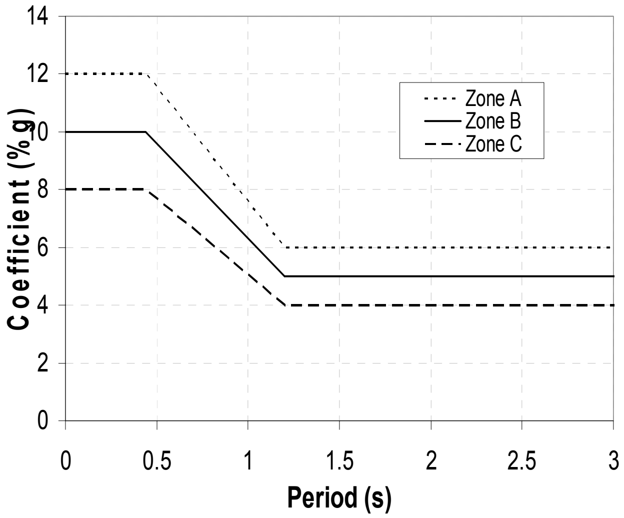

Figure 1. NZSS 1900: basic lateral force coefficients.

The need for ductility was recognized but no specific guidance was given as to how this could be achieved. However, design forces were higher for structural forms recognised as having less ductile characteristics. The seismic design base shear, V, was taken as the total seismic weight of the structure (dead load plus seismic live load), W_t, times the appropriate lateral force coefficient. For moment resisting frame buildings in the highest seismic zone (A), with a fundamental period of less than 0.45s, the design base shear was equal to $0.12W_t$. As indicated in Figure 1 this reduced with increasing period. The corresponding lateral forces for the intermediate and low seismic zones were taken as 5/6 and 2/3 of the zone A values respectively. Members were designed using the working stress method.

The seismic design force, f_i, at a level i in a building was given in Equation 2, where V is the base shear, W_i is the seismic weight at level i and h_i is the height of this level, was given by;

$$f_i = V\left(\frac{W_i\,h_i}{\sum(W_i\,h_i)}\right) \tag{2}$$

The corresponding inter-storey drift (lateral deflection in a storey divided by its inter-storey height) under the applied seismic design forces was limited to 0.005 (clause 8.38.1). No allowance was made for inelastic deformation associated with ductile behaviour and generally stiffness reduction due to flexural cracking was neglected.

Chapter 9 of NZSS 1900 (1964) [7] based design strengths and deformation limits on working stress design. As noted previously the design lateral forces from this period need to be multiplied by a factor of close to 1.3 to give the corresponding design strengths in terms of current methods of ultimate strength design. Generally section properties would have been based on gross section values. As in the previous group of codes generally no allowance was made for reduction in stiffness due to flexural cracking.

A Ministry of Works (MOW) Code of Practice for Design of Public Buildings published in 1970 [8] recommended the use of the ultimate strength method for design of members. This document extended design criteria contained in NZS 1900 by introducing requirements for joint zone reinforcement in beam column joints, requiring columns to be confined and the sum of flexural strengths of columns at a beam column joint to exceed the corresponding sum of the beam flexural strengths. However, it was left to the designer to decide by how much the sum of the column flexural strengths should exceed the beam flexural strengths. No indication was given of the contribution that reinforcement in suspended floor slabs could make to beam strengths.

The ACI 318: 1971 Code of Practice [9] was used as the basis for a NZ provisional Standard. In the ACI code a strength reduction factor of 0.9 was used for beams and a number of provisions were introduced for detailing potential plastic hinge regions. In particular shear reinforcement was required to resist the sum of gravity induced shear and the shear corresponding to flexural strength in the potential plastic hinges. The lapping of bars in potential plastic regions was not permitted. For columns nominal confinement reinforcement was required where the axial load level exceeded 40 percent of the axial load corresponding to balanced conditions. The strength reduction factor for columns was 0.75 where confinement reinforcement was used and 0.7 where they were unconfined. This code also required the sum of column flexural strengths to be greater than the sum of beam strengths, but no minimum ratio was specified.

As noted above the first elements of capacity design were introduced in MOW Code of Practice for Design of Public Buildings [8] and in the ACI 318:1971 code [9]. In later codes/standards the confinement requirements and requirement for shear reinforcement in beam column joint zones was revised extensively.

3.4 Group D, late 1970s and early 1980s; NZS 4203: 1976 and ACI 318: 1971 or Provisional NZ Concrete Structures Standard

The Standard "NZS 4203-1976 Code of Practice for General Structural Design and Design Loadings for Buildings" [10] was based on the ultimate strength design method, but it permitted working stress methods to be used. Ultimate strength design rules for concrete sections were taken from the

ACI 318: 1971 code of practice or the provisional NZ structural concrete standard. Seismic zones introduced in NZS 1900 Chapter 8 [6] were retained for rigid and intermediate soils except the values were increased by a factor of 1.25 to allow for the change from working stress design to ultimate strength design. In addition a new set of lateral force coefficients were introduced for soft soils, as shown in Figure 2. Where working stress design was used the lateral force coefficients were multiplied by 0.8.

The lateral force coefficients were based on a nominal displacement ductility of 4 for a design earthquake with a return period of the order of 150 years. The design base shear varied in recognition of the inherent ductility of different structural forms and materials of construction. Two factors were used for this purpose, namely a structural form factor *S* and a material factor *M*. With these values the minimum design base shear, *V,* was given by Equation 3;

$$V = C_d \; S \, M \, W_t \tag{3}$$

where the lateral force coefficient, C_d, is read from Figure 2 for the fundament period, T, and W_t is the seismic weight. The fundamental period was calculated from properly substantiated data or by calculations but its value was not to exceed the expression in equation 4 by more than 20 percent;

$$T = 0.064\sqrt{\Delta} \tag{4}$$

T is in seconds, Δ is the lateral displacement at the top level in mm (clause 3.4.4) when subjected to the lateral forces defined for Equation 1. For moment resisting frames *S* was 0.8 and for reinforced concrete *M* was 1.0, which implied that the design level earthquake displacement ductility is 5. The lateral seismic forces were based on a return period of 150 years.

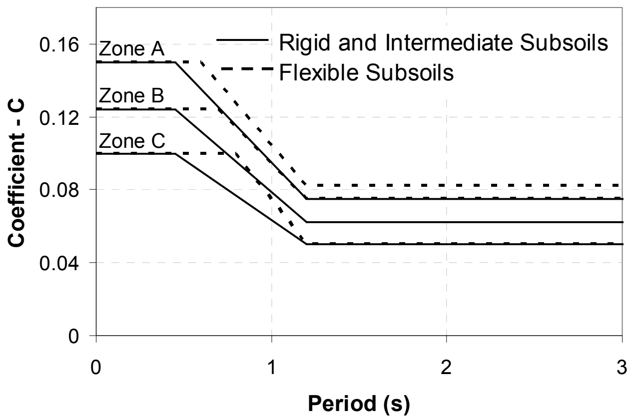

Figure 2. NZS 4203:1976 and 1984, basic seismic coefficient

This standard required the design seismic forces to be offset from the centre of mass by a distance given by an equation, which generally gave a value of the order of 0.1 times the width of the building. This offset allowed for accidental torsion, which are torsional actions arising due to non-uniform distribution of live load, localised un-intentional changes in stiffness and actions induced by torsional ground motion. In an amendment the offset distance given by the equation was replaced by a distance of 0.1 times the width of the building normal to the direction of loading. This requirement increased the lateral strength of a building by an amount which varied with the form of the structure. Typically the lateral strength increase was between 10 and 20 percent but with an overall average value of close to 12.5 percent. This torsional requirement has been maintained in all subsequent loadings standards.

For seismic analysis it was recommended that section properties be taken as 0.75 times the gross section values to allow for stiffness reduction in beams and columns due to flexural cracking. To allow for inelastic deformation for the ultimate limit state the elastic inter-storey drift found in an equivalent static or response spectrum analysis was multiplied by 2/SM with a limiting value of 0.01. This indicates that the inter-storey drift was taken as 50% of the value corresponding to that given by the equal displacement concept [11], which in terms of NZS 3101: 2006 [1] has some similarity to the structural performance factor, S_p, of 0.5.

The distribution of seismic design forces at different levels was similar to that for the 1965 loadings code. However, there was a change when the ratio of the overall height to length ratio of the horizontal force resisting system in the direction of the seismic actions being considered was greater than three. In this case 10% of the base shear is applied at the uppermost level with the remaining 90% being distributed according to the height of the floor above the base as given in Equation 2.

3.5 Group C, mid 1980s to early 1990s; NZS 4203: 1984 and NZS 3101: 1982

The NZS 4203: 1984 "Code of Practice for General Structural Design and Design Loadings for Buildings" [12] was similar to the 1976 edition but the material factor, M, was reduced from 1.0 to 0.8 as a result of increased confidence in the ductile performance of well detailed reinforced concrete. Inter-storey drifts found in an equivalent static analysis were increased by a factor of 2.0/*SM* as before, which implies an overall displacement ductility of 6.25 for ductile moment resisting frame buildings, while for structures designed by the modal response spectrum method the corresponding factor was taken as 2.2/*SM*, implying a corresponding value of 5.7. To control possible adverse P-delta actions inter-storey drift limits were modified. For structures in high seismic zones (e.g. Zone A) the ultimate limit state inter-storey drift limit was taken as 0.01 of the inter-storey height. For the intermediate, B, and low, C, seismic zones the inter-storey drift limits were set at 5/6 x 0.01 and 2/3 x 0.01 respectively (Clause 3.8.3). No recommendations were given on section properties as this was now covered by the structural concrete code, NZS 3101: 1982.

NZS 3101: 1982 "Code of Practice for the Design of Concrete Structures" [13] gave considerable information on detailing for ductility. It contained recommendations on stiffness values of sections that should be used in seismic analyses. It was recommended that the effective section stiffness for beams be based on 0.5 of the gross section properties and the corresponding value for columns be based on gross section properties. The strength reduction factor was defined as 0.9 for flexure in beams and for columns where the concrete was confined. For columns with only nominal confinement reinforcement the strength reduction factor was 0.7 where the nominal design axial load equal or exceeded $0.1 A_g f'_c$ and 0.9 for zero axial load, with linear interpolation between these limits. However, the seismic provisions required all potential plastic hinges in columns resisting seismic actions to be confined. The confinement requirements, which were previously based on the ACI code, were replaced by an expression which varied the required amount of confinement reinforcement with the maximum design axial load level in the column due to gravity load and earthquake actions (clause 6.5.4.3).

In this standard many of the requirements for capacity design were introduced. The over-strength moments in beams were taken as 1.25 or 1.4 times the nominal flexural strength of beams with grade 275 and 380 steel respectively. A method of determining design actions in columns, which ensured that potential primary plastic regions would be confined to the beams except at the column bases, was given in the commentary (appendix C3A) [13]. This was achieved by calculating the maximum over-strength moments that could be applied to a joint zone and scaling the bending moments in the columns, which were found from an equivalent static analysis, so that the joint zone was in equilibrium under these actions. The column moments were then multiplied by a dynamic magnification factor, which allowed for changes in distribution of column moments due to higher elastic and inelastic mode behaviour [14]. For columns, which contributed to two frames the dynamic magnification factor was increased to allow for bi-axial moments acting on the columns. The columns were proportioned to have nominal flexural strengths under the most adverse axial load conditions. Methods of assessing axial forces and column shears consistent with the beams sustaining over-strength actions were also given. The required minimum ratio of the sum of the nominal column flexural strengths to the sum of the nominal beam flexural strengths at beam-column joint zones in one way frames ranged from 1.6 to 2.4. In many cases the minimum ratios were exceeded as the flexural strengths of the column changed between the top and bottom of the joint zone and for practical purposes the same longitudinal reinforcement was used in the column on each side of the joint zone. The minimum ratio varied with the reinforcement grade that was used in the beams and the position of the joint zone in the building. This method of designing columns for seismic actions was adopted into the 1995 edition of NZS 3101 [18] and retained with minor modifications in the 2006 edition of NZS 3101 [2].

The effective width of slabs, which were assumed to contribute to the negative moment design strength and over-strength of the beams were smaller than the values used in later Standards. In this Standard the effective width was taken typically as 2 to 4 times the depth of the slab measured from the column faces (clause 6.5.3.2 (e)). A consequence of this is that both the design strength and over-strength values are likely to be underestimated compared with practice as detailed in the 2006 Standard. The significance of elongation of plastic hinges in the beams pushing the columns apart was not recognised in the Standard and as a result there are two short comings, namely;

- No requirements were given for the length of support ledges for precast floor components and as a consequence some precast floor units were mounted on small ledges and/or on cover concrete. Such supports are now considered inadequate;
- In a limited number of cases elongation of plastic hinges in beams could lead to an underestimate of shear force in the first storey columns and the formation of a plastic hinge forming in the columns adjacent to the first level beams. However, as confinement generally controlled the amount of transverse reinforcement required in the columns this omission is unlikely to have significant implications in terms of retrofit requirements. It should be noted that as forces induced by elongation are internal to the structure they ***cannot*** reduce the storey shear strength.

3.6 Group B, mid 1990s to mid 2000s; NZS 4203: 1992 and NZS 3101: 1995

In the "Code of Practice for General Structural Design and Design Loadings for Buildings", NZS 4203: 1992 [15] the design ultimate seismic actions were based on a return period of 500 years for buildings of normal importance. In previous standards the design return period had corresponded to 150 years. The 1992 Standard introduced the structural performance factor, S_p, which effectively meant the peak displacement ductility, according to the equal displacement concept, was equal to the structural ductility factor divided by the S_p factor (μ/S_p), while the design displacement was equal to S_p times the equal displacement concept value. The introduction of this factor is in line with practice in US codes.

The three seismic zones used in the previous NZS 4203 Loading Standard [12] were replaced by a contour map with zone factors represented by Z, which ranged from 0.6 in low seismic zones, such as Auckland and Northland, to 1.2 in high seismic regions, such as Wellington, the East Coast of the North Island and North of Cheviot in the South Island. Three different soils foundation conditions were recognised, namely rock and stiff soil sites, intermediate soils and flexible soils, with the spectral shapes changing significantly from the earlier standards. Spectral values for different periods were given for different structural ductility factors. A few of the curves are shown in Figure 3 for intermediate soils.

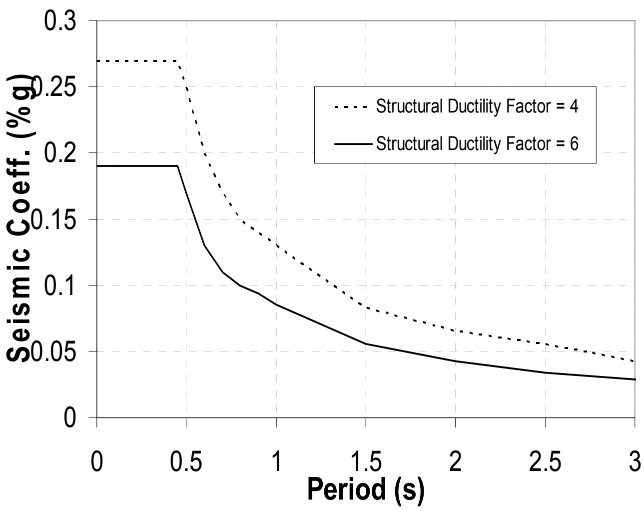

Figure 3. NZS 4203:1992, Acceleration coefficient for soil class B

At any period the design spectral value, or lateral force coefficient, $C(T)$, is given by Equation 5. For the equivalent static method the base shear, V, is given by Equation 6;

$$C(T) = C_h(T,\mu) S_p \, Z \, L_u \tag{5}$$

$$V = C(T)\; W_t \;\; but\, not\, less\;\; than\;\; 0.03 W_t \tag{6}$$

where Z is the seismic zone factor (1.2 for Wellington and 0.6 for Auckland), $C_h(T, \mu)$ is the basic seismic hazard coefficient for period, T, and structural ductility factor, μ, S_p is a structural performance factor taken as 0.67, L_u is limit state factor taken as 1.0 for the ultimate limit state. The value of structural ductility factor, μ, for ductile moment resisting frame structures was equal to or less than 6 for structures with a fundamental period of 0.7 seconds or more. For a period, T, of zero seconds, μ may be taken as 20 with linear interpolation of values between zero and 0.7 seconds. The period was calculated using Rayleigh's theory or an equivalent analytical method (clause 4.5.2).

The distribution of the base shear into lateral design forces at each level is similar to that for NZSS 1900, chapter 8 [6] except that 8% of the base shear was applied at the uppermost level with the remaining 92% being distributed as indicated by the Equation 7;

$$F_i = F_t + 0.92\,V\,\frac{W_i h_i}{\sum W_i h_i} \tag{7}$$

where F_t is equal to 0.08V at the uppermost level and is zero for all other levels, h_i is the height of level i being considered above the base and W_i is the seismic weight of level i.

Comparative analyses using the equivalent static and modal response spectrum methods of analysis showed that the equivalent static method over-estimated the elastic seismic displacement demands in multi-storey structures compared to the modal method. To make some allowance for this effect the Loadings Standard allows the elastic inter-storey drifts found by with the equivalent static method to be reduced by a lateral deflection modification factor, which is 0.85 for buildings with six or more storeys and 1.0 for buildings with one storey. For buildings with 2 to 5 storeys, linear interpolation may be used between these limits. The lateral deflection modification factor was used where lateral deflections were required to calculate storey drifts but it was taken as 1.0 when lateral deflections were required for the purpose of calculating a fundamental period by Rayleigh's method (see clause 4.8.1.5).

The envelope of lateral displacements due to seismic actions found by an equivalent static or modal response spectrum analysis and scaled up to allow for inelastic deformation by two methods (clause 4.7.3):

Method 1: The critical displacement of each floor was found by multiplying the displacement by the structural ductility factor. These displacements were added to the additional displacements found from an analysis for P-delta actions (clause 4.7.5 and Appendix C4.B).

Method 2: The elastic displacement envelope was added to the inelastic deformation such that the total lateral displacement of the top level was equal to the corresponding lateral displacement in Method 1. The inelastic component of deflection was taken as the deflected shape found assuming that all the inelastic deformation occurred in primary plastic hinges and that bending moments did not change as the inelastic rotations increased. This is similar to the deflected shape that would be found in a push over analysis in which strain hardening is neglected. The inelastic component of deflection was determined for each possible collapse mechanism.

The design inter-storey drift was taken as the larger of the differences in lateral displacements found between adjacent floors in the storey being considered from Methods 1 and 2.

The method of controlling P-delta actions by limiting seismic displacements in the 1984 code was replaced by a requirement that the actions and deformations induced by P-delta actions be included in the design. A recommended method was included in the commentary (commentary appendix C4.B) and the basis of this approach is outlined in reference [16]. No allowance was required for low rise buildings, for buildings where the structural ductility factor was less than 1.5, or where the ratio of gravity load resisted by the storey times the inter-storey drift calculated as detailed above was less than 0.133 times the design inter-storey shear force due to seismic actions (clause 4.7.5). The recommended method of allowing for P-delta actions was based on calculating the storey shear forces required to balance P-delta actions associated with the deflected shape defined by Method 1 or 2 above. These actions were then scaled to allow for the incremental increase in deflections associated with repeated inelastic deformation typical of major earthquakes. Typically allowing for P-delta actions in multi-storey ductile frame buildings by this method increased the required design strengths and inter-storey drifts found by equivalent or modal response spectrum analyses by about 40 percent [17].

The limiting ratio of inter-storey drift to storey height was set at 0.025. In comparative analyses of multi-storey frame buildings using the equivalent static, modal response spectrum or elastic time history methods, it was found that the critical inter-storey drifts were under-estimated when compared with drifts found from inelastic time history analyses. The extent of this under-estimate varied with height of the frame. To allow for this difference the design limit of drift ratio was set at 0.015 for buildings with a height of 30m or more and 0.020 for buildings with highs equal to or less than 15m, with linear interpolation between these limits (clause 2.5.4.5). These drift limits correspond to the use of drift modification factors in NZS 1170.5: 2004 [1] of 1.25 for buildings of 15m or less and 1.67 for buildings of 30m or more.

In the Standard, NZS 3101: 1995 "Code of Practice for the Design of Concrete Structures" [18] the strength reduction factor for flexure in beams and flexure and axial load in columns was defined as 0.85 and the option of using a nominally un-confined column with a strength reduction factor of 0.7 was removed. Hence all columns were required to be confined to at least the level of limited ductile members. In addition structural ductility factors in excess of 6, which were permitted by NZS 4203:1992 [15] for structures with short fundamental periods, were excluded for concrete structures.

The Standard required allowance to be made for the reduction in stiffness that occurred in beams and columns due to flexural cracking and the commentary contained detailed recommendations on appropriate values. It was recommended that the effective stiffness for members in ductile frames be taken as a factor times the second moment of area of the gross section. For rectangular beams this factor was taken as 0.4, while for of Tee and L beams the corresponding factor was taken as 0.35. For columns the value varied from 0.4 I_g for an axial tension of ratio ($N/A_g f_c'$) of -0.05, 0.6 I_g at a ratio of 0.2 and 0.8 I_g at a ratio of 0.8, with interpolation for intermediate axial load ratios (clause C 3.4.3.3).

The importance of elongation of beams on the interaction of beams and slabs was recognised and the effective width of slab that was assumed to act with a beam to contribute to negative moment flexural strength was increased. The new limits for the effective width of slab on each side of a beam were taken as the smaller of a quarter of the span of the beam, half the clear span of the slab normal to the beam, or for beams framing into a column at right angles to a free edge the width of the column where there is no transverse beam, or where there is a transverse beam one quarter of the span of this beam (clause 8.5.3.3). In addition there was a requirement that the reinforcement from the slab be considered to be effectively anchored. The effective flange width acting at a section was assumed to be the same for both ultimate strength calculations and over-strength calculations. No allowance was made for the possible contribution of strength due to prestressed reinforcement in precast units in a floor.

Equations for the confinement of columns were revised with the amount of confinement being related to the level of axial load sustained by the column. There was some confusion in the value of this axial load. Appendix A (clause A9) indicates that for ductile structures the axial load should be calculated for capacity design, but elsewhere the axial load is stated as the value corresponding to the ultimate limit state with earthquake actions. It is believed the value calculated from capacity design is what was intended.

The design criteria for beam-column joints were revised and the requirement for both joint ties and intermediate joint zone column bars were reduced compared with the 1982 edition of the Standard.

A new section on design of diaphragms was introduced. Nominal requirements were given for reinforcement to tie the floor into the structure and add to the robustness of the building (clause 13.3.7). Section 13 of this Standard required floors, which acted as diaphragms, be designed to sustain shear forces associated with the ultimate limit state actions and to be capable of resisting actions associated with over-strength actions in potential plastic hinges. In-elastic deformation of diaphragms was not permitted unless justified by theoretical or experimental studies (clause 13.4.1).

Minimum seating lengths for precast floor components after reasonable allowance for construction tolerances were set as the larger of 1/180 of the clear span or 50mm for solid slabs or hollow-core units and 75mm for ribbed members (clause 4.3.6.4).

3.7 Group A, mid 2004 and on; NZS 1170.5:2004 and NZS 3101:2006

In NZS 1170.5, Earthquake Actions Standard, 2004 [1] the shapes of elastic spectra, $C_h(T)$, are given for four different soil classes, namely rock (A and B), shallow soil (C), deep or soft soil (D) and very soft soil (E). The spectral shape for class C soil conditions is shown in Figure 4. The elastic design spectrum for a specific site is obtained from Equation 8, which is given below.

$$C(T) = C_h(T)\, Z\, R\, N(T,D) \qquad (8)$$

Where;

- $C_h(T)$ is the spectral shape value at the period, T, for the appropriate soil conditions (see Figure 4);
- Z is the seismic hazard factor, which is given on a contour map with values varying from 0.13 to 0.6, with values for Auckland, Wellington and Christchurch being 0.13, 0.40 and 0.22 respectively;
- R is a factor for the design return period. For normal buildings R is 1.0 for the ultimate limit state and 0.25 for the serviceability limits state. For important buildings the value of R is increased;
- N(T,D) is a factor which modifies the shape of the elastic response spectrum for sites near a major known fault. For building at a distance greater than 20km from the fault the factor has a value of 1.0. For distances less than this it increases with increasing period, T. The maximum value is 1.72 and it occurs at a distance of 2 km or less from the fault when the period is 5 seconds or more.

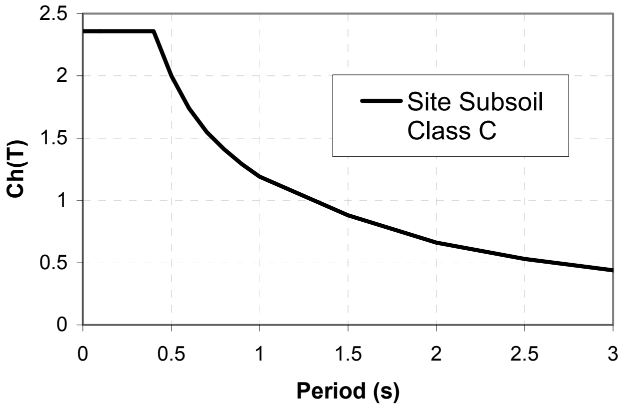

Figure 4. Spectrum shape factor for NZS1170.5 with class C soils.

To obtain the design base shear for the ultimate limit state with the equivalent static method the base shear coefficient, $C_d(T_1)$, is taken from the corresponding elastic response spectrum value at the fundamental period of the building, $C(T_1)$, and modified to allow for the structural ductility factor, μ, of the building. For soil classes A, B C and D it is given by;

$$\begin{aligned} C_d(T_1) &= \frac{C(T_1)S_p}{k_u} \\ &\geq (0.05Z + 0.02)R \quad but\ not\ less\ than\ 0.03R \\ k_u &= \mu \quad for\ T_i \geq 0.7s \\ &= \frac{(\mu - 1)T_1}{0.7} + 1 \quad for\ T_1 < 0.7s \end{aligned} \qquad (9)$$

S_p is the structural performance factor defined in NZS 1170.5 as 1.0 for a structural ductility factor of 1 and 0.7 for a structural ductility factor of 2 or more (clause 4.4.2). However, these values were redefined in NZS 3101: 2006 [2]. The fundamental period, T_1, is calculated from Rayleigh's method or other analytical method.

The equal displacement concept states that the peak lateral displacement of a structure is approximately equal to the corresponding displacement of an elastically responding structure with the same fundamental period. On this basis the peak displacement ductility is equal to μ/S_p. However, in design the lateral displacement is taken as μ times the value found from the base shear given by Equation 9 and it is added to additional deflections associated with P-delta actions. In effect the design displacement is taken as S_p times the value corresponding to the equivalent displacement concept. Several reasons are given for this action in the commentaries to NZS 4203: 1992 and NZS 1170.5: 2004 [15 & 1]. However, the principal reason, which is offered, is that the damage sustained in a earthquake is related more to a displacement that is obtained several times during the earthquake rather than a peak value that is sustained only once.

The base horizontal seismic shear for the equivalent static method, V, is given by;

$$V = C_d\left(T_1\right)W_t \tag{10}$$

where W_t is the seismic weight of the structure. The base shear is distributed into lateral forces up the height of the building as for NZS 4203: 1992, see Equation 7, with the same allowance being made for torsion.

The lateral displacement correction factor introduced in NZS 4203: 1992 [15] to reduce the difference between lateral displacements found with equivalent static and response spectrum modal analyses was retained in this standard, with the minor additional requirement that the reduction coefficient was set at 1.0 for buildings with soft or weak storeys (clause 6.2.3).

The method of calculating lateral displacements allowing for inelastic deformation introduced in NZS 4203: 1992 was retained and the approach of allowing for P-delta in the commentary to NZS 4203: 1992 was incorporated into the standard with the following modifications (see 6.5.4);

- P-delta actions could be neglected in a building when the ratio of gravity load resisted in a storey times the inter-storey drift ratio for the ultimate limit state was less than 0.1 times the design storey shear strength. Previously this limit was set at 0.133;
- In no case was the ratio given above allowed to exceed 0.3;
- Where the designer was required to allow for P-delta actions 2 choices were given. Either a method that was similar to that given in the commentary to NZS 4203:1992 could be used, or a simpler conservative method of scaling the seismic design actions was given (clause 6.5.4).

A formal set of requirements is given for capacity design of ductile structures. These included identifying the location of primary plastic hinges (that is those plastic hinges associated with the chosen ductile failure mechanism) and ensuring the strength distribution in the structure was such that the locations of these potential primary plastic hinges are fixed (clause 5.6).

NZS 1170.5 requires that the minimum detailing level used in each potential plastic region is based on the maximum predicted deformation sustained in that region when the structure is subjected to the specified ultimate limit state with seismic actions (clause 5.6.3.2). This is a major change from previous practice where the detailing was selected on the basis of the structural ductility factor used for determining the seismic design forces.

The Standard NZS 3101: 2006, "Structural Concrete Standard, including amendment 2 (2008)" [2] does not include the design of brittle elements and hence values for structural ductility factors of less than 1.25 are not given. The S_p values given in NZS 1170.5 were replaced by 0.9 for a structural ductility factor, μ, of 1.25 and 0.7 for a structural ductility factor of 3 or more, with linear interpolation between these limits (clause 2.6.2.2).

Minor revisions were made to the section stiffness values recommended in NZS 3101: 1995. There was some reduction is stiffness values where a high grade reinforcement was used. However, the overall effect of the changes from the previous standard is small (clause C6.9.1).

In previous Structural Concrete Standards little guidance was given on serviceability requirements for earthquake loading cases and consequently this condition was not assessed to any appreciable extent. However, to bring this Standard into line with NZS 1170.5: 2004 [1] serviceability limit state with earthquake actions is addressed. Generally these requirements limit the structural ductility that can be used in the ultimate limit state to 5 for buildings of normal importance and in some cases a lower value is required. For the serviceability limit-state a structural ductility factor of 1 is required for SLS1, but a value of 2 may be used for SLS2 (clause 2.6.2.3.1). However, SLS2 is only applied to buildings of high importance (NZS 1170: 2004, clause 5, 2.1.4). Clause 2.6.3.1 requires either (i) that the serviceability design strength is equal to, or exceeds, the serviceability design actions, or (ii) that an analysis shows that crack widths and deflections remaining after a serviceability limit-state earthquake are acceptable. Such an analysis is required to consider the effect of inelastic deformation caused by moment redistribution and other shake down effects associated with repeated inelastic displacements during an earthquake. Serviceability requirements do not need to be considered for nominally or limited ductile structures (clause 2.6.3.1). Strength requirements for the serviceability limit state are related to the average strength of structural sections, with the average strength being taken as the nominal strength with a strength reduction factor of 1.1 (clause 2.6.3.2). (Note the nominal strength is calculated from lower characteristic strengths hence the need to increase the strength reduction factor to correspond to average material strengths).

To bring the standard into line with NZS 1170.5 [1] three classifications were defined for buildings; namely nominally ductile, limited ductile and ductile buildings. This classification depends on the value of the structural ductility factor used to determine the seismic design actions. In addition, three classifications of potential plastic regions were defined; namely nominally ductile plastic regions, limited ductile plastic regions, and ductile plastic regions. The detailing required in each of these regions is given in the appropriate section of the Standard while section 2.6.1.3 sets out the material strain limits, which are defined as curvature limits for plastic hinges. The way in which the curvatures are found from the deflected shape is defined in the clause. It should be noted that there is no direct connection between the type of plastic region and classification of a building.

The section on diaphragms contains similar material to NZS 3101: 1995 [18]. However, there is one major change. Forces induced in the diaphragms associated with the ultimate limit-state, or with actions associated with over-strength in potential plastic regions, are to be designed on the basis of a strut and tie analysis.

Observations of the performance of floors containing precast prestressed units in major earthquakes together with research on floors in New Zealand, and an assessment of over-seas practice, has led to a number of significant changes being introduced on the design of precast floors in NZS 3101: 2006 (plus amendment 2). This aspect is likely to be of major importance in considering the need for retrofit of existing structures. The criteria relating to the use of precast floors, which have been introduced into NZS 3101: 2006 plus amendment 2, include;

(a) Additional requirements relating to the minimum size of supporting ledges and the requirement for low friction bearing strips with hollow-core units (clause 18.7.4);

(b) The need use a thin linking slab between a precast unit and a parallel structural element, such as a beam or wall, which may deflect in a vertical direction relative to the precast unit. This is required to prevent the load transfer between the structural elements causing the precast units to fail (clause 18.6.7.2);

(c) New requirements for shear strength of precast units in zones where over-strength actions can cause tensile stresses to be induced on the top surface of the precast units. In this situation the shear strength is reduced to a value comparable with a non-prestressed beam of the same dimensions (clause 19.3.11.2.4);

(d) The position where reinforcement connecting the precast unit to the supporting structure is cut off or reduced is based on the capacity of the floor to sustain the negative moments and axial tension. These may be induced in the floor when over-strength actions act at the supports and vertical ground motion induces negative moments in the floor (clause 19.4.3.6);

(e) Designers are cautioned against supporting precast units on structural elements that may deform and induce torsional moments as these may lead to torsional failure of the floor unit. This situation can be critical for hollow-core flooring (clause C19.4.3.6).

4.0 CAPACITY DESIGN OF MOMENT RESISTING FRAME BUILDINGS

The Earthquake Actions Standard, NZS 1170.5 [1] sets out the basic requirements for capacity design. This process requires a ductile collapse mechanism to be selected and the primary plastic regions associated with this mechanism located. The required design strengths of these plastic regions are determined and the related over-strengths found. The remainder of the structure is then proportioned and detailed to ensure that inelastic deformation is confined to the chosen primary plastic regions. It should be noted that limited inelastic deformation may occur in secondary plastic regions due to actions such as elongation, dynamic magnification and other effects, which are not considered in an analysis of the structure (clauses 2.3.3 & 5.6.3).

The Structural Concrete Standard, NZS 3101: 2006 [2] sets out the overall requirements for capacity design for ductile and limited ductile concrete structures (clause 2.6.5). Capacity design of ductile moment resisting frames requires the items listed below to be addressed.

(a) To ensure that non-ductile behaviour cannot occur in a major earthquake it is essential to design the columns so that in the event of a major earthquake a beam sway mode develops in preference to a column sway mode. This is achieved by designing the columns so that their nominal strengths are sufficient to resist, by a nominated margin, the maximum design actions that can be transmitted to them through the primary plastic hinges. Two methods for determining capacity design actions for columns are given in this standard. With both methods where a column forms part of two moment resisting frames bi-axial actions must be considered in design (D3.2.3 (d) and D3.2.2).

The first method is based on the one contained in NZS 3101: 1995 Appendix A. A number of modifications were made to this method, namely to consider bi-axial actions more directly and to allow for the effects of elongation of beams on plastic hinge locations. Each column above the primary plastic hinge located at its base of the column is proportioned and detailed with the aim of minimising inelastic deformation that may occur (Method A in Appendix D, clause D3.2).

The second method allows a limited number of potential plastic hinges to be located in the columns provided the remaining columns have sufficient nominal strength to ensure that the storey column sway shear strength exceeds the storey beam sway shear strength in each storey by a nominated margin. The beam sway storey shear strength is calculated assuming over-strength actions are sustained in all the potential plastic regions associated with the storey being considered (Appendix D, clause D3.3).

The detailing requirements for these two methods differ from one another, with the second method having more restrictions on lap positions of longitudinal bars and requiring more confinement reinforcement than the first method.

(b) The significance of elongation of plastic hinges in beams on the actions in columns is recognised. In particular elongation can cause plastic hinges, which are not identified in standard analyses, to form in columns immediately above or below the first elevated level. This can increase the shear forces induced in the columns. However, as the requirement for confinement reinforcement is generally more critical than shear reinforcement this is unlikely to be critical for the shear strength of these columns.

(c) In calculating over-strength actions in beams, allowance needs to be made for the possible material strengths and the increase in stress that may be sustained due to strain hardening. Strain levels are much higher in over-strength conditions than in normal ultimate strength design conditions. As strain levels increase the width of floor slab that acts with a beam increases. Consequently a greater width of slab needs to be assumed to contribute to over-strength than to design strength. This effect is recognised in the NZS 3101: 2006 but it was not recognised in earlier standards (clauses 9.4.1.6.1 and 9.4.1.6.2).

(e) Precast prestressed floor units in a floor slab, which span past potential plastic hinges in a beam, can make a very significant difference to the over-strength capacity of plastic hinges. A method of assessing the strength due to this source is given in the Standard (clause 9.4.1.6.2). In previous standards the contribution of prestressed floor unit to strength of beams was neglected, which may cause a substantial under-estimate of beam over-strength in a limited number of cases.

(f) In addition to the effects detailed above there are two actions relating to the over-strength moments which may be induced into columns by beams that are not quantified in the current standard. Research has not yet advanced to a stage where a method of assessing these actions has been developed.

The first of these actions involves the bending moment that can be induced in a column due to torsional moments in transverse beams. These are only likely to be significant in structures where the seismic actions transverse to the frame being considered are resisted by walls. Where seismic actions are resisted by moment resisting frames in two directions plastic hinges in the transverse beams will greatly reduce their torsional resistance.

The second of these actions involves the strength increase that occurs in plastic hinges in beams where precast prestressed units are parallel to the beams and they are supported on a transverse beams which are located close to the plastic hinges. In this case the precast units tie the floor together so that the floor slabs bend like deep beams to accommodate the elongation from the plastic hinge(s). This deep beam type action partially restrains the elongation. The resultant axial force imposed on the plastic hinge or hinges can increase the flexural strength significantly.

Table 1 lists the major changes which have occurred in capacity design and detailing requirements for ductile moment resisting frame structures. It should be noted that this table only attempts to draw attention to major changes. There are a multitude of other changes that are not shown or discussed in this paper.

5.0 COMPARISON OF STRENGTH AND STIFFNESS REQUIRED IN PREVIOUS STANDARDS WITH CURRENT VALUES

5.1 Introduction

To make this comparison a number of approximate relationships are introduced. Consequently the numerical values and conclusions given in this paper can only be taken as a general guide and each individual structure needs to be assessed. The comparisons are based on the "equivalent static method of analysis". In later standards this method of analysis was restricted for use to structures where either the structure was reasonably regular and had a fundamental period of less than 2 seconds or it was a low rise structure [15]. However, as the values are used for the purpose of comparison this limit has been ignored.

To allow comparisons to be made the codes/standards from previous decades with current standards (2009) they have been put into 6 groups as indicated in section 1 and as shown in Tables 2 and 3.

To compare the required minimum strengths and stiffness requirements for buildings of the different groups of codes/standards a series of regular moment resisting buildings with fundamental periods of 0.5 to 3 seconds in steps of 0.5s are considered. The fundamental periods are assumed to be consistent with section stiffness values in NZS 3101: 2006 (Group A). It is assumed that these buildings are founded on intermediate foundation soils equivalent to Class C in NZS 1170.5 [1]. The required performance of these buildings is then reassessed in terms of the requirements for the other groups of codes/standards (B to F). For each of these groups, B to F, a revised set of fundamental periods is assessed allowing for changes in the way in which section stiffness values were calculated. In terms of soil foundations it is assumed that class C in NZS 1170.5 is equivalent to;

- Intermediate soils in NZS 4203:1992 (category b);
- Rigid and intermediate soils in NZS 4203: 1984 and 1976.
- With response spectral values in earlier codes no distinction was made for soil type.

Analyses are made for sites in Wellington, Christchurch and Auckland.

Two sets of analyses have been made. The relative minimum strengths are assessed in set 1 and the relative minimum stiffness values in set 2.

Set 1

For each set of periods (0.5s, 1.0s etc) the corresponding minimum required base shear strengths are assessed for each group of code/standards (A to F). This base shear is then divided by the value corresponding to Group A to give a ratio, with the results being shown in section 5.2. In addition the equivalent structural ductility factors are found corresponding to lateral force requirements in the Group A standards.

Set 2

For each set of periods (0.5s, 1.0s etc) it is assumed that each building with each group of standards has been proportioned to just satisfy the maximum permitted inter-storey drift limit under the action of equivalent static forces at the centre of the building. On the basis of the assumption that the building sustains the minimum specified base shear at this drift a relative stiffness is found. This value is divided by the value corresponding to Group A Standards (mid 2000s) to give the relative stiffness ratios in section 5.2.

To enable the minimum strength and stiffness requirements of design standards used in different decades to be compared with current values (2009) allowance needs to be made the factors listed below. This inevitably involves a number of approximations, as detailed below.

1. The way in which stiffness of sections is evaluated has changed. With the early design codes/standards either no allowance or different allowances were made of the influence of flexural cracking on the second moment of area *(I)*. A consequence of this is that the fundamental periods and lateral seismic deflections tended to be under-estimated compared with current practice. To allow for this change an average stiffness correction factor is used. This is calculated on the assumption that the lateral deflection is more sensitive to beam stiffness than column stiffness. The correction factor is based on 2/3 of the second moment of area for beams and 1/3 of corresponding value for columns. Using this value the change in fundamental period corresponding to earlier codes/standards can be assessed and the equivalent static base shear corresponding to those codes/standards found. In addition the lateral deflection given in former design codes/standards can be translated into corresponding values related to current requirements (2009).

2. (a) In 1965 the loading standard set the allowable inter-storey drift at 0.005 based on working stress design with no allowance for additional deflection associated ductile behaviour (inelastic deformation). In this case, to get the equivalent inter-storey drift to compare with current design, the limiting 0.005 drift limit needs to be increased to correspond to the limit associated with the required design strengths, as determined by ultimate strength theory. With later standards the change in strength reduction factor needs to be considered.

Table 1: Major short comings of design criteria- detailing and capacity design.

Standard	Details
NZS 3101: 2006	See section 4 item (f) in this paper
NZS 3101: 1995	All of above **Precast floors** • Specified size of support ledge and details (clause 4.3.6.4), are inadequate for precast floor units. Currently acceptable values are given in NZS 3101: 2006 (clause 18.7.4). Provisions to prevent brittle failure of the units listed in (b) to (e) in 3.7 are not covered in this standard but they are addressed in NZS 3101:2006. **Capacity design** • Items in 3.7 (a), (b), (c) and (f) in this paper are not covered by this standard. • Contribution of prestressed floor components to over-strength of beams is not considered (see NZS 3101:2006, clause 9.4.1.6.2). • The difference in effective widths of floor slabs contributing to nominal negative moment flexural strength of beams and to over-strength of beams is not considered (See NZS 3101:2006, clauses 9.4.1.6.1 and 9.4.1.6.2). **Strength design** • Detailing of primary plastic hinges in terms of section curvature sustained under ultimate limit state is not considered (see NZS 3101:2006, clause 2.6.1).
NZS 3101: 1982	All of above **Capacity design** • The effective width of floor slabs assumed to contribute to over-strength is very low (clause 6.5.3.2 (e)). This is likely to lead to an under-estimate of over-strength moments in beams. **Precast Floors** • No details were given for size of support ledges for precast floor components leading to possibility of collapse due to elongation or due to spalling of cover concrete at plastic hinges. **Details** • Confinement of columns is low compared with NZS 3101: 1995 for columns with a high axial load. • Confinement for columns not considered to resist seismic actions was not required. Secondary columns may not have the ductility to sustain seismic design displacements imposed by primary lateral force resisting structure (clause 6.4.7.1 (b)).
ACI 318-1971	All of above **Capacity Design** • No minimum strength ratio of columns to beams strengths defined and over-strength calculations not required. However, the sum of flexural strength of columns at a beam column joint zone was required to be greater than corresponding strength of beams. • Potentially inadequate shear strength of members in potential plastic hinge regions as value of v_c not reduced. • Shear reinforcement in beam column joint zones inadequate. **Details** • Columns sustaining moderate to high axial load ratios inadequately confined compared to later standards. • Inadequate restraint to buckling of beam and column bars in potential plastic hinge regions. • Shear strength of beam column joints likely to be substandard as no specific design criteria were given for shear or for development of beam and column bars passing through joint zones. • Bars may be lapped in inappropriate locations; column bars often lapped at floor level. • Beam bars in external joint zones likely to be bent away from the joint zone core. • Column ties anchored by 90^o bends in cover concrete.
NZSS 1900 **Ch. 8 :1965**	All of above. Essentially no seismic detailing specified. It is likely that reinforcement is inadequately anchored for seismic actions, particularly in columns (note plain bars used extensively at this period).
CP 114: 1957	As above

(b) In 1976 an allowance was introduced for inelastic deformation. With this standard the deformation corresponding to design actions (elastic model subjected to design seismic forces) was multiplied by 2/SM, which as previously noted is equivalent to multiplying by the displacement ductility and a factor of 0.5. In 1992 [15] the term "Structural performance factor" was introduced and it was given a value of 2/3, which effectively replaced the 0.5 factor. The design lateral deflection was taken as the storey drift found from the elastic model subjected to the seismic design forces multiplied by S_p and the structural ductility factor, μ. This approach was maintained in the 2004 Standard [1] but with a modified S_p factor of 0.7 for ductile structures. Hence in NZS 4203: 1976 and 1984 the peak building displacement ductility was taken as the structural ductility factor and the lateral displacement was taken as half the value given by the equivalent displacement concept. In NZS 4203: 1992 and NZS 1170.5: 2004 the corresponding peak building displacement ductility is μ/S_p and the corresponding lateral displacement is taken as S_p times the value given by the equivalent displacement concept.

3. In 1992 allowance was introduced to reduce the discrepancy between the lateral deflections calculated using the equivalent static and modal methods of analysis. Lateral deflections from the equivalent static analysis were multiplied by a factor, which varied from 0.85 for building with 6 or more storeys to 1.0 for a building with one storey, with linear interpolation between these limits. (This factor was previously referred to as the lateral deflection modification factor.) To make an allowance for this factor it is necessary to know the number of floors in the building.

4. In 1992 the maximum design inter-storey drift limit was varied depending on the overall height of the building. It was set at 0.02 for building with a height of 15 m or less and 0.015 for building with a height of 30 m or more, with linear interpolation between theses limits. In 2004 a similar approach was used except the limiting inter-storey drift was set at 2.5%. However, the difference in deflection between storeys was multiplied by a drift modification factor, which was 1.2 for building of 15 m or less in height and 1.5 for building 30 m or more in height. Linear interpolation was used between these limits.

5. In 1976 and subsequent loadings standards an allowance was introduced for accidental torsion, which was maintained in subsequent standards. As noted previously this typically increased the lateral strength of moment resisting frame buildings on average by about 12.5%.

6. In the 1950s and 1960s lateral strength was based on elastic design of sections. In terms of current design practice the average strength of a beam design by elastic theory was of the order of 1.3 times the corresponding current design strength. In 1976 this ratio was set at 1.25, which resulted in sections designed by elastic theory being on the conservative side of sections designed by ultimate strength theory.

7. With the introduction of ultimate strength theory in the early 1970s the strength reduction factor for flexural strength of beams was set at 0.9. This was reduced to 0.85 in 1995.

8. Allowance for P-delta actions has changed markedly. Until 1984 there were no requirements. However, in 1984 limits were placed on the inter-storey deflection with the allowable deflection being reduced in the lower seismic zones. In 1992 the approach was changed with the requirement that these actions be considered directly. A method of allowing for these actions was given in the commentary, which was adopted into the Earthquake Actions Standard (2004). This required additional strength to be added to the lateral force resisting structure and allowance made for additional deflections associated with P-delta actions. Typically for moment resisting frame building with 4 or more storeys proportioned to meet the minimum strength and stiffness requirements, the increase in lateral strength was of the order of 40% and the increase in lateral deflection was also of the order of 40% [17]. For buildings with 2 storeys, typically half this increase in strength and deflection has been assumed in assessments made in this paper.

To determine the required minimum base shear corresponding to NZS 1170.5: 2004 [1] a structural ductility factor of 5.0 has been used as the serviceability requirements in NZS 3101: 2006 [2] make it difficult to use a higher value. The same building is then assessed in turn to find the fundamental periods, which would have been calculated from the criteria contained in each code/standard group from the previous decades. Clearly the change in the way in which section properties are assessed makes a major difference here. With the calculated fundament periods the corresponding base shear can be found and this can be used to find the corresponding structural ductility factor (or its equivalent) in current practice.

To make an allowance for the factors noted in 3 and 4 above the number of storeys and the height of buildings, h, for an assumed fundamental period must be known so that limiting inter-storey drifts given by NZS 4203: 1992 and NZS 1170.5: 2004 can be assessed and the number of storeys can be determined to allow the lateral deflection modification factors given in these two standards to be used. In the UBC code of practice [19] and a number of other codes (IBC etc) an empirical equation is given which relates the height of a concrete building to its fundamental period. This equation is deliberately set to give a low fundamental period to ensure that the base shear used in a design is on the conservative side. Multiplying the period predicted by the UBC equation by 1.5 appears to give fundamentals periods that are reasonably close to those predicted by Rayleigh's method for buildings designed to meet the minimum stiffness requirements of current New Zealand standards in moderate and high seismic zones. For low seismic zones the expression is likely to under-estimate the period by a small margin. On this basis the relationship between fundamental period and height in metres is given by the approximate relationship:

$$T_1 = 0.11\,(h)^{0.75} \tag{11}$$

From this equation the height and number of storeys can be assessed for a given period. The accuracy of these values is of relatively minor importance in this comparison of code requirements.

Table 2 sets out the relative section stiffness values used in the groups of standards. The fundamental period for Groups B to F is found by multiplying the fundamental period for Group A by the square root of the relative stiffness factor, calculated as set out in (1) above. For Groups A and B the relative stiffness values for the columns have been based on an axial load ratio of 0.1, as the practical range of change in axial load ratio makes relatively little difference to the stiffness ratio.

Table 3 sets out the basic properties for buildings, that is the fundamental period, assessed height and number of storeys assessed in terms of current standards (Group A). In addition corresponding fundamental periods assessed on the basis of the assumed section properties defined in each code/standard Group, B to F, are given.

Table 2: Section properties for groups A to F.

Group of codes/standards	I_{beam}	I_{column}	Stiffness Factor	Period relative to group A
A – NZS 1170.5 & NZS 3101-2006	$0.4I_g$	$0.48I_g$	$0.43I_g$	1.0
B – NZS 4203-1992 & NZS 3101-1995	$0.4I_g$	$0.48I_g$	$0.43\ I_g$	1.0
C – NZS 4203-1984 & NZS 3101-1982	$0.5I_g$	$1.0I_g$	$0.67\ I_g$	0.80
D – NZS 4203: 1976 & ACI 318-1971	$0.75I_g$	$0.75I_g$	$0.75\ I_g$	0.76
E - NZSS 1900, Ch.8, 1965 & Ch. 9.3 1964	$1.0I_g$	$1.0I_g$	$1.0\ I_g$	0.66
F - NZSS 95 Pt. IV 1955 & CP114 1957	$1.0I_g$	$1.0I_g$	$1.0\ I_g$	0.66

Table 3: Building properties and fundamental periods calculated for different code/standard groups.

Group	Item	NZS 3101-2006 Periods					
A	**Period (s)**	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**
	Height (m)	7.5	19	33	48	64	82
	No. floors	2	5	9	14	19	24
		Periods for groups B to F with stiffness correction					
B	**Period (s)**	0.5	1.0	1.5	2.0	2.5	3.0
C	**Period (s)**	0.4	0.8	1.2	1.6	2.0	2.4
D	**Period (s)**	0.38	0.76	1.14	1.52	1.9	2.28
E	**Period (s)**	0.33	0.66	0.99	1.32	1.65	1.98
F	**Period (s)**	0.33	0.66	0.99	1.32	1.65	1.98

5.2 Comparative strengths

With ductile moment resisting frame buildings designed to NZS 1170.5: 2004 [1] and NZS 3101: 2006 [2] the required minimum design base shear strength is found from the equivalent static base shear, which is increased to allow for torsional and P-delta actions. In design a strength reduction factor of 0.85 is used. In comparing minimum strengths derived from previous standards allowance has to be made for changes in response spectra and allowances for the other effects listed in 5.1. Thus, for example, the equivalent static base shear for a buildings designed in the late 1960s calculated from the appropriate response spectrum using the appropriate fundamental period given in Table 3, would need to be;

- Increased by a factor of 1.3 to allow for the change of design methods (working stress to ultimate strength);
- Reduced to allow for the effects of P-delta actions (as described in 5.1 (8));
- Reduced to allow for torsional actions by multiplying by 1/1.125 (as described in 5.1 (5)).
- In later standards allowance must also be made to allow for changes in strength reduction factor.

Table 4 lists the ratio of the equivalent static base shear corrected for the factors listed above for each group divided by the equivalent base shear required by current standards (Group A). As noted previously the Group A values are based on a structural ductility factor of 5.

The relative high strengths obtained with the Group F (1955 code) are a reflection that the lateral force coefficient did not change with either seismic zone or fundamental period of building. In later loadings Standards there has been in general

Table 4: Ratio of equivalent static base shear strengths for each group divided by value corresponding to Group A.

Period for (A)	A	B	C	D	E	F
			Wellington			
0.5	1.00	0.94	0.52	0.65	0.80	0.53
1	1.00	1.02	0.75	0.96	1.27	0.99
1.5	1.00	0.91	0.66	0.89	1.27	1.34
2	1.00	0.86	0.81	1.01	1.24	1.65
2.5	1.00	0.75	0.81	1.01	1.24	1.65
3	1.00	0.75	0.81	1.01	1.24	1.65
			Christchurch			
Period for (A)	**A**	**B**	**C**	**D**	**E**	**F**
0.5	1.00	1.14	0.79	0.99	1.21	0.96
1	1.00	1.24	1.13	1.46	1.92	1.80
1.5	1.00	0.97	0.87	1.18	1.69	2.13
2	1.00	0.97	0.87	1.09	1.33	2.13
2.5	1.00	0.97	0.87	1.09	1.33	2.13
3	1.00	0.97	0.87	1.09	1.33	2.13
			Auckland			
Period for (A)	**A**	**B**	**C**	**D**	**E**	**F**
0.5	1.00	1.44	1.07	1.33	1.63	1.63
1	1.00	1.28	1.25	1.62	2.12	2.49
1.5	1.00	1.13	1.02	1.10	1.58	2.49
2	1.00	1.13	1.02	1.02	1.25	2.49
2.5	1.00	1.13	1.02	1.02	1.25	2.49
3	1.00	1.13	1.02	1.02	1.25	2.49

Table 5: Structural ductility factors for Group A standards for buildings designed to minimum requirements of earlier groups.

Period for (A)	A	B	C	D	E	F
			Wellington structural ductility factors			
0.5	5	5.37	9.98	7.90	6.39	9.78
1	5	5.34	9.61	7.69	6.28	9.42
1.5	5	4.90	6.71	5.19	3.94	5.04
2	5	5.50	7.61	5.64	3.92	3.73
2.5	5	5.81	6.18	4.94	4.04	3.03
3	5	6.67	6.18	4.94	4.04	3.03
			Christchurch structural ductility factors			
Period for (A)	**A**	**B**	**C**	**D**	**E**	**F**
0.5	5	4.36	6.45	5.08	4.08	5.20
1	5	4.40	6.34	5.07	4.15	5.18
1.5	5	4.04	4.43	3.42	2.60	2.77
2	5	5.17	5.74	4.26	2.96	2.35
2.5	5	5.17	5.74	4.60	3.76	2.35
3	5	5.17	5.74	4.60	3.76	2.35
			Auckland structural ductility factors			
Period for (A)	**A**	**B**	**C**	**D**	**E**	**F**
0.5	5	3.35	4.66	3.65	2.91	2.91
1	5	3.47	4.68	3.75	3.06	3.06
1.5	5	3.90	4.00	3.09	2.35	2.01
2	5	4.42	4.91	4.55	3.16	2.01
2.5	5	4.42	4.91	4.91	4.01	2.01
3	5	4.42	4.91	4.91	4.01	2.01

a reduction in lateral force coefficient (response spectra) for the lower seismic zones compared to the more earthquake prone regions.

The required minimum design strengths since 1976 (Group D) have been relatively consistent even though the ultimate limit state design earthquake loading was changed from a return period of 150 years to 500 years in 1992. This relative consistency was the result of changes in response spectra and the introduction of the structural performance factor, S_p, in 1992. This factor results in the peak building ductility increasing from 4/SM, which was 6.25 by NZS 4203:1984, to the structural ductility factor divided by the structural performance factor, μ/S_p, which was 9 (6/0.67) in NZS 4203: 1992 and 7.1 (5/0.7) in NZS 1170.5: 2004.

Having found the corresponding equivalent static base shear in terms of current standards (Group A) the associated structural ductility factor for NZS 1170.5 can be found by comparing the equivalent static base shear values. These values are shown in Table 5.

5.3 Comparative stiffness

Assuming that buildings have been designed to just satisfy the maximum inter-storey drift limit in each group of standards and by finding the equivalent base shear strength the relative stiffness can be assessed. The ratios of these relative stiffness values to the corresponding values for the Group A standards are listed in Table 6.

In assessing the maximum permissible inter-storey drift under the equivalent static forces at the centre of the building the maximum permissible design drift for the ultimate limit state has been modified for the following factors by;

- Dividing by the structural ductility factor;
- Adjusting to allow for the assumed effective section stiffness values;
- Decreasing to allow for deflection associated with accidental torsion;
- Modifying to allow for the drift modification factor or its equivalent;
- Allowing for the lateral deflection stiffness modification factor, which reduced the lateral deflection due to the over-estimate associated with the equivalent static method;
- Allowing for the influence of P-delta actions on lateral displacements;
- Allowing for changes in calculating inelastic deformation associated with the S_p values or the *2/SM* factors.

From Table 6: it can be seen that for the high seismic zones the Group B standards led to the stiffest buildings in the high seismic zone of all groups. For the intermediate and low seismic zones there has been some reduction in required stiffness in the Group A standards compared with values required previously from 1984 (Group C). The stiffness values for Group D (1976) are generally comparable with those of Group A. The group E values show very much lower stiffness values. Prior to 1965 no inter-storey drift limits were specified for seismic actions and consequently no relative stiffness values have been given for group F codes.

Table 6: Relative stiffness of buildings to corresponding Group A values.

Period	**Wellington relative stiffness**				
for (A)	**A**	**B**	**C**	**D**	**E**
0.5	1	1.17	0.54	0.49	0.24
1	1	1.18	0.58	0.45	0.25
1.5	1	1.28	0.74	0.59	0.35
2	1	1.14	0.65	0.54	0.35
2.5	1	1.08	0.80	0.61	0.34
3	1	0.94	0.80	0.61	0.34
Period	**Christchurch relative stiffness**				
for (A)	**A**	**B**	**C**	**D**	**E**
0.5	1	1.45	1.01	0.76	0.37
1	1	1.43	1.06	0.68	0.37
1.5	1	1.56	1.34	0.89	0.53
2	1	1.22	1.04	0.71	0.46
2.5	1	1.22	1.04	0.66	0.37
3	1	1.22	1.04	0.66	0.37
Period	**Auckland relative stiffness**				
for (A)	**A**	**B**	**C**	**D**	**E**
0.5	1	1.88	1.75	1.06	0.52
1	1	1.81	1.80	0.92	0.51
1.5	1	1.61	1.86	0.98	0.58
2	1	1.42	1.52	0.67	0.43
2.5	1	1.42	1.52	0.62	0.34
3	1	1.42	1.52	0.62	0.34

6.0 DISCUSSION AND CONCLUSIONS

In the introduction four questions concerning comparison of previous design standards with current practice were posed. The conclusions to these questions arising from this study are given below.

1. ***What factors should be considered when comparing codes?***

   To assess how a structure designed in an earlier decade matches up to current design criteria four different aspects need to be considered, namely;

   - The strength of the structure;
   - The stiffness of the structure;
   - The detailing used in the different structural elements;
   - The hierarchy of failure that will occur to ensure that in the event of a major earthquake a ductile failure mechanism will form in preference to non-ductile failure modes.

   This paper outlines different factors which should be considered in assessing the strength and stiffness provisions given in different structural design standards. In addition major changes are noted in detailing requirements through the decades together with an overview of the extent of capacity design in those decades.

   It is shown that many different factors in a structural standard contribute to the minimum specified strength and stiffness values. For a realistic comparison to be made between different structural codes/standards over the decades allowance must be made for differences in;

   - Design spectra;
   - Inter-storey drift limits;
   - Allowances for accidental torsion;
   - Allowances for P-delta actions;
   - Assumptions made regarding section properties and in particular allowance for the influence of flexural cracking on effective stiffness;
   - Allowance made for inelastic deformation on storey drift;
   - Allowance made for over-estimate of inter-storey deflections in elastic analysis with the equivalent static method compared with other methods of analysis.

2. ***How much effect do these factors have?***

   The effect that the different factors in the codes/standards have on the minimum strength and stiffness values for buildings in shown in Tables 2, 3, 4, 5 and 6. Tables 2 and 3 show how different assumptions that have been made over the decades with regard to the influence of flexural cracking on stiffness of sections can have a significant influence on the design strength and calculated inter-storey deflection.

   Table 4, which shows that the minimum lateral strength required for buildings with ductile moment resisting frames has, with a few exceptions, been reasonably constant from the mid 1970s to the present day. In earlier decades prior to the 1970s the required strength was generally higher, particularly in the low seismic zones. It is likely that in the earlier decades the strength requirements largely determined the size of the structural members while in later decades the size was largely determined by stiffness requirements.

   Table 6 highlights one potential problem. In one paper minimum required strength and stiffness values required by NZS 4203: 1992 and NZS 3101: 1995 were compared with a number of major overseas codes of practice [21]. This comparison indicated that the New Zealand codes in general required lower strength and stiffness levels for ductile moment resisting frame structures than the overseas codes of practice. Table 5 indicates that little has changed in terms of required strength with the introduction of NZS 1170.5: 2004 but there has been an appreciable decrease in required stiffness in the medium to high seismic zones. To the authors' knowledge no justification has been advanced for the reduction in minimum stiffness associated with the introduction of NZS 1170.5: 2004.

   Table 1 outlines the major short comings of capacity design and detailing provisions of the groups of standards used from the 1950s to 2009 with particular reference to buildings where the lateral force resistance is provided by ductile moment resisting frames. For many buildings a major area of concern is the detailing used for the support of precast floor components. Prior to 1995 no minimum width of support ledge was specified. The values given in NZS 3101: 1995 improved the situation, but these values are now considered inadequate to cope with the elongation that may arise in a major earthquake. In addition recent research has shown that brittle failure of the precast units can occur unless careful detailing is used. These requirements are given in NZS 3101: 2006 plus amendment 2 but not in previous standards. Consequently the details used in and near the support zones of precast floor units should be given close attention in assessing the need for seismic retrofit of buildings.

   As indicated in Table 1 capacity design provisions were introduced and modified throughout the period. In all the standards prior to Group A there was some level of under-estimate on the contribution that floor slabs can make to the over-strength of plastic hinges in beams. This under-estimate may in some cases erode the margin against the formation of a column sway mechanism in the event of a major earthquake.

3. ***Do previous code comparisons reported in the literature make sense?***

   Comparisons have been made in the past. One of these was in a paper presented by the DBH paper [20], in which the influence of different design standards on the proportion of hollow-core floors that were at risk in a major earthquake was assessed. In the DBH paper it was indicated that structures designed to NZS 4203: 1984 would be stiffer and have smaller inter-storey drifts in an earthquake than those designed to meet the requirements of earlier or later standards. This erroneous conclusion arose as all but the first 2 items in the list in section 6 (1) were ignored in reports on which the paper was based. The calculations reported in Tables 5 and 6 show different trends. When all the appropriate factors are considered it is found that the required minimum stiffness of buildings with ductile moment resisting frames designed to meet NZS 4203: 1992 for Wellington and Christchurch, are generally appreciably greater than for previous or later standards. It is only in the lowest seismic zone, as represented by Auckland, that the required stiffness of buildings is greater for NZS 4203; 1984.

4. Are there any surprises found while making these comparisons?

There are two major surprises which are highlighted by this study;

- The equal displacement concept, which is widely accepted in books and papers and has been extensively used in developing design rules, states that in-elastic structures that have the ability to dissipate an appreciable amount of energy through hysteretic damping deflect approximately as far as an elastically deforming structure provided both structures have the same initial stiffness. It is apparent that this concept has not been applied in any of the previous or current seismic loading standards in New Zealand. It should also be noted it is not applied in many codes of practice overseas.

- The stiffness requirements of the current seismic loading standard, NZS 1170.5 [1] are for moderate to high seismic zones appreciably less than the corresponding values in NZS 4203: 1992. If the first named author had been aware of this stiffness reduction he would have asked for some justification in this reduction as it appears to be out of line with overseas practice.

REFERENCES

1. Standards New Zealand, "NZS 1170.5:2004, Structural Design Actions, Part 5, Earthquake Actions New Zealand".
2. Standards New Zealand, "NZS 3101:2006, Concrete Structures Standard".
3. New Zealand Standards Institute, "NZSS 95, Pt. IV, Basic Loads to be Used in Design and Their Methods of Application", Mar. 1955.
4. Andrews A.L., "Before Beginning", *Published NZSEE*, 2008.
5. British Standards Institution, CP 114:1957, "The Structural Use of Reinforced Concrete in Buildings", 1957.
6. New Zealand Standards Institute, NZSS 1900, Chapter 8: "Basic Design Loads", 1965.
7. New Zealand Standards Institute, "NZSS 1900, Chapter 9.3, Design and Construction, Concrete", 1964.
8. Ministry of Works, "Code of Practice, Design of Public Buildings", *Office of Chief Structural Engineer*, May 1970.
9. American Concrete Institute, "ACI Standard, Building Code Requirements for Reinforced Concrete, (ACI 318-71), Detroit Michigan, 1971.
10. Standards Association New Zealand, "NZS 4203:1976, Code of Practice for General Structural Design and Design Loadings for Buildings".
11. Blakeley, R.W.G. "A review of the new code provisions for separation of elements and buildings", *Bulletin of NZNSEE*, Vol. **7**, No. 3, September 1974.
12. Standards Association New Zealand, "NZS 4203:1984, Code of Practice for General Structural Design and Design Loadings for Buildings".
13. Standards Association of New Zealand, "NZS 3101:1982, Code of Practice for Design of Concrete Structures".
14. Paulay, T., "Column, evaluation of actions", *Bulletin NZNSEE*, Vol. 10, No. 2, June 1977, pp 85-94.
15. Standards New Zealand "NZS 4203:1992, Code of Practice for General Structural Design and Design Loadings for Buildings".
16. Fenwick, R.C., Davidson, B.J. and Chung, B.T., "P-delta actions in seismic resistant structures", *Bulletin NZ National Society for Earthquake Engineering*, Vol. 25, No.1, Mar. 1992. pp. 56-69.
17. Ruiz, J.A.F., "Performance of ductile reinforced concrete moment resisting frames subjected to earthquake actions", *University of Canterbury,* Civil Engineering Report No. 2006/1, pp239.
18. Standards New Zealand, "NZS 3101: 1995, Concrete Structures Standard".
19. ICBO, 1997, "Uniform Building Code, 97", Vol. 2, *International Conference of Buildings Officials*, Whittier, CA.
20. Stannard, M., Bialostocki, R., Hopkins, D.C., Jury, R. and Saunders, D., "Hollow core floors – a regulator's perspective", paper no. 07, *NZSEE Conference*, March 2007.
21. Fenwick, R., Lau D. and Davidson B.J.,"A comparison of the seismic design requirements in the New Zealand loadings standard with other major design codes", *NZSEE Bulletin*, Vol.**35**, No.4, Sept. 2002, pp190-203.