From brittle to ductile: 75 years of seismic design in New Zealand

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"Earthquake-resistant design does not rest on any clearly defined basis; many fundamentals are obscure."

"As it is generally held that destructive seismic vibrations have, as a rule, periods of from 1 to 2 seconds, it is advisable for the natural period of a building to be less than 1 second. If it were above the 2 second limit, we should have a decidedly flexible structure, large movements, likelihood of damage to various rigid elements, risk of panic and other ill effects"
From *"Structural Design of Earthquake Resistant Buildings" by Irwin Crookes, 1940.*

ABSTRACT: This address traces the development of seismic structural design in New Zealand since the 1931 Hawke's Bay Earthquake, with emphasis on reinforced concrete buildings. From the mainly rigid and brittle unreinforced masonry structures which behaved so poorly in the 1931 'quake through the development of flexible ductile seismic design and base (seismic) isolation of the 60's to 80's to today where the structural engineer is expected to design and construct a building which will not only remain standing with little damage but will be operational a short time after the major earthquake. In some ways the structural design aims and objectives have turned full circle in the intervening 75 years. We have gone from brittle rigid structures through a period where flexibility was paramount to now where flexibility is limited and greater lateral stiffnesses are required but with ductile elements in the structure. This paper traces the efforts of the New Zealand's pre-eminent structural engineers and scientists to make seismic design techniques world leading. In most facets they have been successful (in my view) but as I will say more than once, only time will tell!

1. INTRODUCTION

1.1 In the beginning: after the earthquake

Before the 1931 Hawke's Bay earthquake there was nothing in NZ in the way of seismic provisions for the design of structures. Some provisions were drafted after the 1929 Murchison earthquake but a draft code was not published till after the major damaging Magnitude 7.8 Hawke's Bay earthquake we are celebrating (if that's the right word) at this conference (Davenport 2004). This process of seismic regulations closely following a major shake had occurred in Japan after the Great Kwanto Earthquake of 1923 (Magnitude 8.3) and in the USA after the San Francisco (1906) and the Long Beach (1933) 'quakes. It wasn't that earthquakes were a new phenomenon to NZ in 1931. The early settlers had built many of buildings of stone and brick masonry (like their home country counterparts) which were destroyed in Wellington by the earthquakes of 1848 Marlborough and 1855 Wairarapa 'quakes. Many of the masonry buildings were replaced with timber structures, some of which still exist (Wellington's Governmental building, the second biggest wooden building in the World) but many later burnt down to be replaced by unreinforced brick masonry structures. The first known book (in English) on the subject of earthquake resistant design and construction was written by C. Reginald Ford, an architect, and published in NZ, in 1926. This book covered earthquake damage in past NZ events as well as summarising earthquake damage reports from the USA and Japan. Ford's recommendations for designing seismic resistant structures were well ahead of their time.

The NZ Government of 1931 set up a Building Regulations Committee which presented a report to Parliament in June 1931 entitled "Draft General Earthquake Building By-law" (Cull 1931). This followed a literature survey completed by the British Building Research Station with various architectural, engineering and scientific groups contributing towards the specification of "adequate and acceptable seismic design criteria" (Shepherd 1968). The draft regulations required a horizontal acceleration of 0.1g, which was reduced to 0.08 times the weight above for general buildings and 0.1 of the weight for public buildings when the regulations were finally ratified. This rectangular distribution of lateral force with building height could be increased by the Local Authority, but not beyond a maximum acceleration of 0.15g (Davenport).

The newly constituted Standards Association published NZSS no. 95, New Zealand Standard Model Building By-Law in October 1935. The Code required that parts of buildings be tied together and that bracing was to be symmetrical, torsional effects should be taken into account and buildings to be used for public gatherings should have frames constructed of reinforced concrete or structural steel. These attributres obviously were not apparent in most of the damaged and destroyed buildings after the Hawkes Bay earthquake, although most of the framed structures survived the earthquake with little structural damage (van de Vorstenbosch *et al* 2002). The code restricted the use of solid round steel and cast iron as beams and columns. There was an increase in the material allowable stresses when the seismic force was being considered with gravity and partial live loads (25% in reinforced concrete) and mention of differing site conditions was made but there was little data to be able to give reliable coefficients for soft site conditions, for example. The 1935 By-Law did not apply over the whole country and it was left to local bodies to accept and use it as they wished. It is believed that local authorities in Auckland, Napier, Wellington, Christchurch and Dunedin used the By-Law extensively.

2. 1935-1965 THE DARK AGES

The only NZ authored and published book (post 1931) that I have found on earthquake engineering is "Structural Design of Earthquake-Resistant Buildings" by S. Irwin Crookes, published in 1940. Crookes was a Senior lecturer in Building Construction at the School of Architecture, Auckland University College and a consulting engineer. The book hardly mentions NZ seismic design practise of the period but draws on US and Japanese practise, which was more advanced than the 1935 By-Law at that time. Crookes makes special play of the lack of knowledge pertaining to seismic design and analysis. He points out that the 0.1g lateral acceleration had been widely accepted in building codes after first appearing in the 1924 Japanese City Building Law but that the actual maximum acceleration reached in the Kwanto earthquake was likely to have been greater than 0.3g. Buildings designed to the 0.1g force level prior to the 1923 event survived without major damage. He assumed that the buildings survived because their actual factors of safety were 3 to 4 times on the allowable design stresses.

The common method of frame analysis for seismic forces was to assume rigid floors and points of contraflexure at mid-length. Some understanding of important aspects of seismic design detailing is evident in the book, for example under reinforced concrete framing, "In columns the vertical bars should be spliced away from positions of maximum moment. If the bars are hooked the laps should be staggered to avoid destroying the homogeneity of the column core. Hoops and spirals should be ample and preferably *closely spaced*, particularly near ends of columns. In beams and girders some negative moment steel must often be carried further towards mid-span than is usual. The use of doubly-reinforced sections, particularly for rectangular beams is often necessary. Haunches are more extensively used than in ordinary (non-seismic) practice: substantial haunch reinforcement is often required". Typical Japanese reinforcement details for seismic resistant design are shown in Figure 1 taken from Crookes. *Not bad for 1940.* Note the presence of beam-column joint ties and top beam bars anchored *down* into the joint.

NZSS 95, The NZ Standard Model Building By-Law was published in 1955, replacing the 1939 edition (revised version of the 1935 By-Law). The major change was that the horizontal force could be distributed up the structure as an inverted triangular distribution, which approximated the first mode deflected shape of the building (Park 1987).

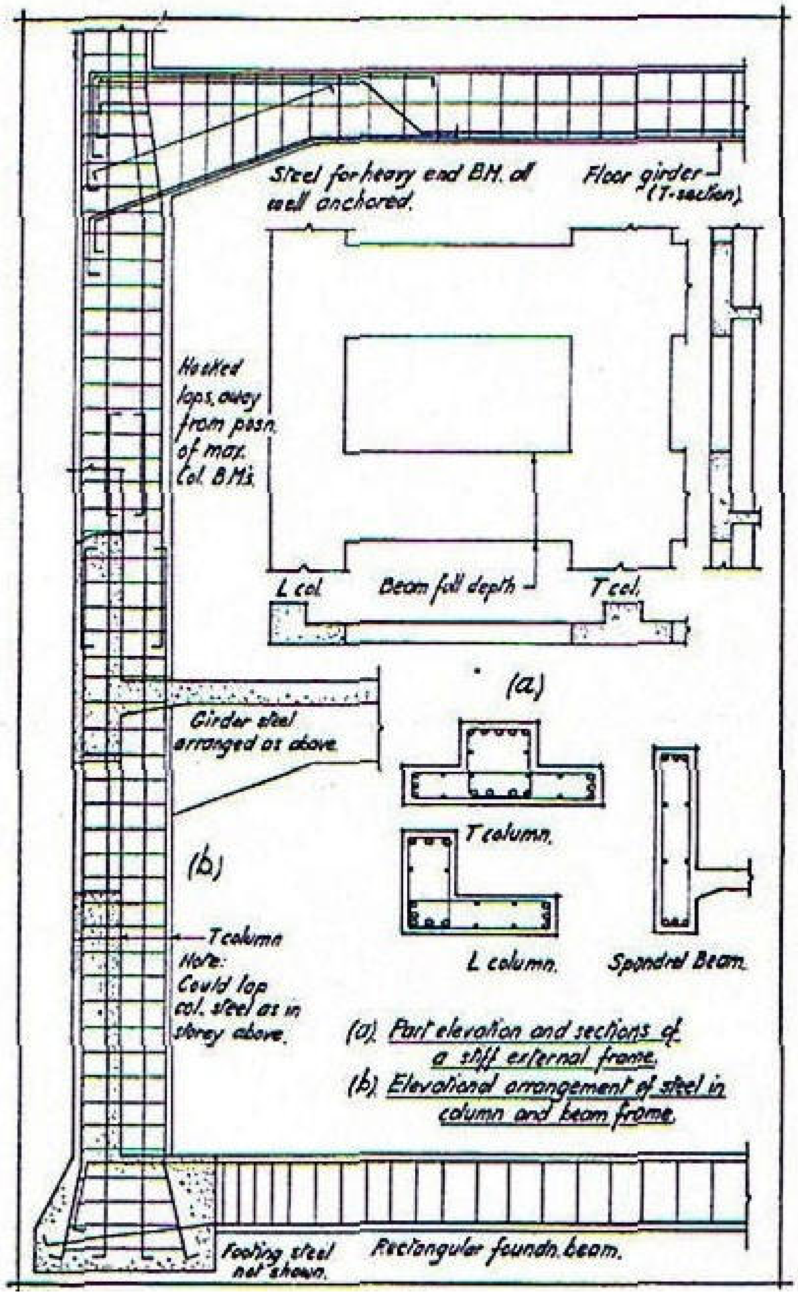


Figure 1. 1930's Japanese detailing of a reinforced concrete frame. From Crookes (1940).

Due to the Depression and the Second World War not many large buildings were constructed in the period between 1935 and 1955. As expressed by Jonnie Johnson (Ministry of Works Chief Structural Engineer) in a *NZ Engineering* Leading Article in September 1963: "Owing to the paucity of building since the inception of seismic provisions in the by-laws these clauses (of 1935) have remained largely unchanged despite considerable advances in the more active overseas countries. The concern of various professional groups in New Zealand at the lack of progress is shown by a number of recent developments A demand for by-law revision stimulated by a request from the Otago Branch of the NZ

Institution of Engineers for consideration of regional seismic zoning, and Auckland's interest in taller buildings, have resulted in a complete redrafting of the seismic provisions in the Model Building By-Laws by the loading committee of the Standards Institute". (J.A.R.J. 1963). His concluding comments make interesting reading today, "All the committees concerned with earthquakes found that their work was hampered by a deplorable lack of essential data. Although the Dominion Physical Laboratory (DSIR) has shown the way in the design and installation of strong-motion instruments and in the development of an electrical analogue to indicate the responses of a wide range of buildings to known earthquake ground motions, much local research work remains to be done. Until designers are able to predict with reasonable certainty the intensities and nature of the ground motions in all regions of importance to the community, and until they know more of the properties of NZ buildings, earthquake resistant design is unlikely to reach a high standard." The same issue contained 2 very topical papers on the subject, "Dynamic Response of Multi-story Buildings by R. O'Driscoll and Robin Shepherd (Life Member NZSEE) and the second by another well known Life Member Ivan Skinner entitled "Earthquake –resistant Design of Buildings RESEARCH PROBLEMS". In 1963 there were only six accelerometers installed throughout the country each able to record 2 horizontal components of the ground acceleration along with about 36 "unsatisfactory" peak-reading accelerometers. Only one 30 m high block of flats in Wellington was instrumented with 7 accelerometers and 7 strain-measuring cells (Skinner 1963).

In the 1960's the only strong motion record useful to structural engineers was from the 1940 El-Centro earthquake in Southern California. This strong motion accelergraph, with a peak ground acceleration of 0.33g, was used by almost every seismic loadings code worldwide from 1940 till about 1980. Ivan Skinner used the El-Centro record and other international records to develop the seismic coefficients published in the new 1965 NZ loadings Code (NZSS 1900, Chapter 8, Basic Design Loads). The horizontal seismic force was found by multiplying the seismic coefficient by the structure's seismic weight. For the first time the seismic coefficient depended on a zone factor, the type of occupancy and the structure's natural period of vibration. The country was subdivided into 3 zones, the highest risk zone (A) covering the lower half of the North Island and the upper half and full western half of the South Island (Figure 2). For medium rise buildings (up to about 4 storeys) the seismic coefficients for Wellington, Christchurch and Auckland (and Dunedin) were 0.12, 0.10 and 0.08 respectively (Fig 3). A structural ductility factor of 4 with a damping of 10% of critical were assumed for reinforced concrete buildings. With regard to this new term "ductility" the Code commentary commented "When a large recorded earthquake is applied to a building and the resultant forces calculated on the assumption that the building deforms elastically with 5 or 10% damping, very large forces are obtained. These calculated forces are usually several times larger than the static forces, which are applied during design under existing building codes. Despite the size of the calculated forces, well constructed buildings have performed surprisingly well during past earthquakes. This reserve of earthquake resistance has been attributed to the ductility of the buildings – the plastic deformation of the structural components and foundations, which absorb energy from the building motion. Hence, buildings in which such plastic deformation is acceptable have a considerable reserve or earthquake resistance beyond their capacity when stressed only to the elastic limit".

The provision of 3 seismic zones was controversial at the time and Park (1984) comments that many of staff at the Seismological Observatory were opposed to any form of zoning due to a lack statistical data from NZ earthquakes. However the Code Commentary says that the zones "were defined taking into account historical records of past damaging earthquakes, the scatter of recorded epicentres, evidence of ground disturbance in reasonably recent times and general geological considerations".

The 1964 Concrete Code (Chapter 9.3 of NZSS 1900) contained very little detail as to how a designer could comply with the requirement that buildings should have a ductility factor of 4 under seismic conditions. Soon after this the development of ductile design and detailing began to emerge. The Third World Conference on Earthquake Engineering was held in Auckland and Wellington in 1965 and this helped stimulate interest in the subject in NZ.

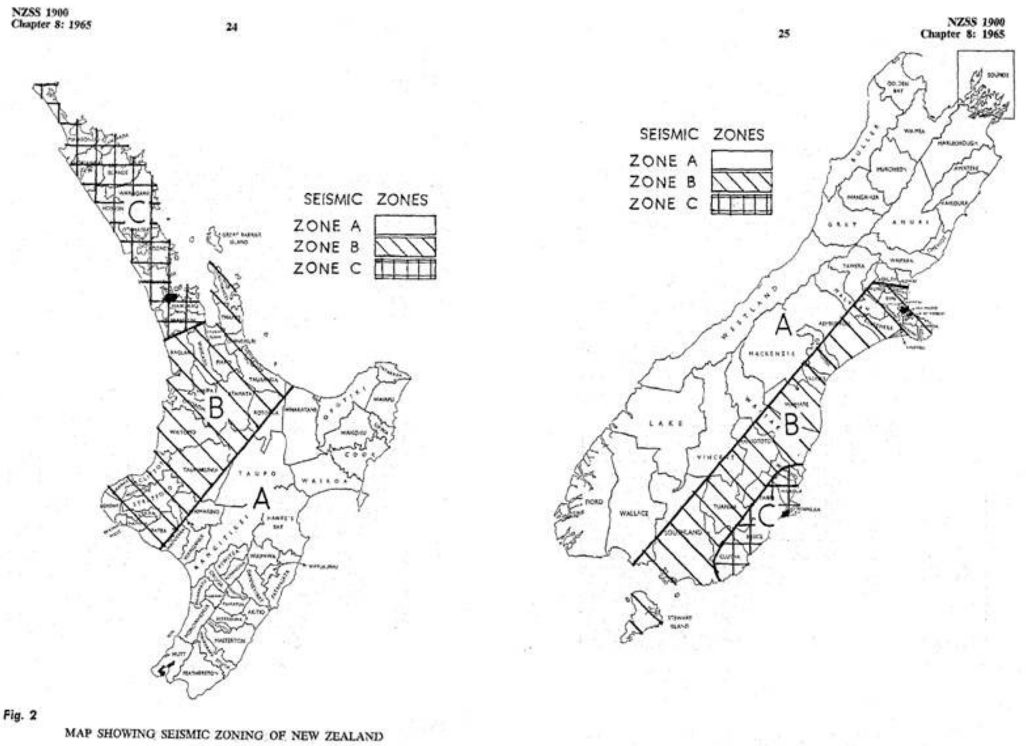


Figure 2. Seismic Zoning map from NZSS 1900 Chapter 8, 1965.

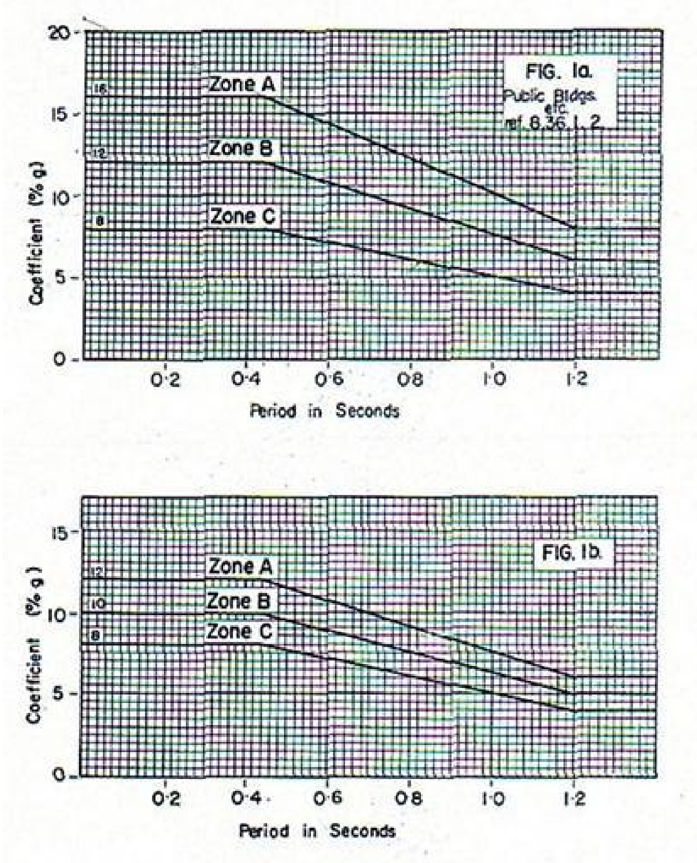


Figure 3. Basic Seismic Coefficients from NZSS 1900, Chapter 8, 1965.

3. STRENGTH AND CAPACITY DESIGN, AND ALL THAT

The "father of ductile design" in NZ was, in my view, John Hollings *(NZSEE Life Member)* of Beca Carter Hollings & Ferner in Wellington who published two inspirational papers in the new *Bulletin of the NZ Society for Earthquake Engineering* in 1969 (Hollings 1969a and 1969b). The first paper was simply titled "Reinforced Concrete Seismic Design" and described a ductile reinforced concrete structure as a "glass structure with lead like hinges". Hollings asked the question as to where these plastic hinges should form and described column hinge mechanisms, but was quick to not recommend their use in buildings greater than a few storeys. He concluded that beam-hinging mechanisms were far better with the appropriate reasons and he suggested a step by step design method to design ductile multi-storey structures, which was a fore runner of the now accepted capacity design procedure. Figure 4 shows a sketch of one of the first beam-hinging mechanisms published. The notion that a plastic hinge zone (PHZ) "must sustain imposed rotations through several reversals (during an earthquake) without loss of structural integrity" was indeed enlightening. Suggestions how parts of the glass-like reinforced concrete members could be detailed to act as lead like plastic hinges were explained, including compression zones in beams being confined with closely spaced ties and the advantages of equal amounts of top and bottom reinforcement at beam ends. The idea that a shear wall was not really a shear wall but a vertical beam with a plastic hinge forming at its base, before shear failure occurred, was described.

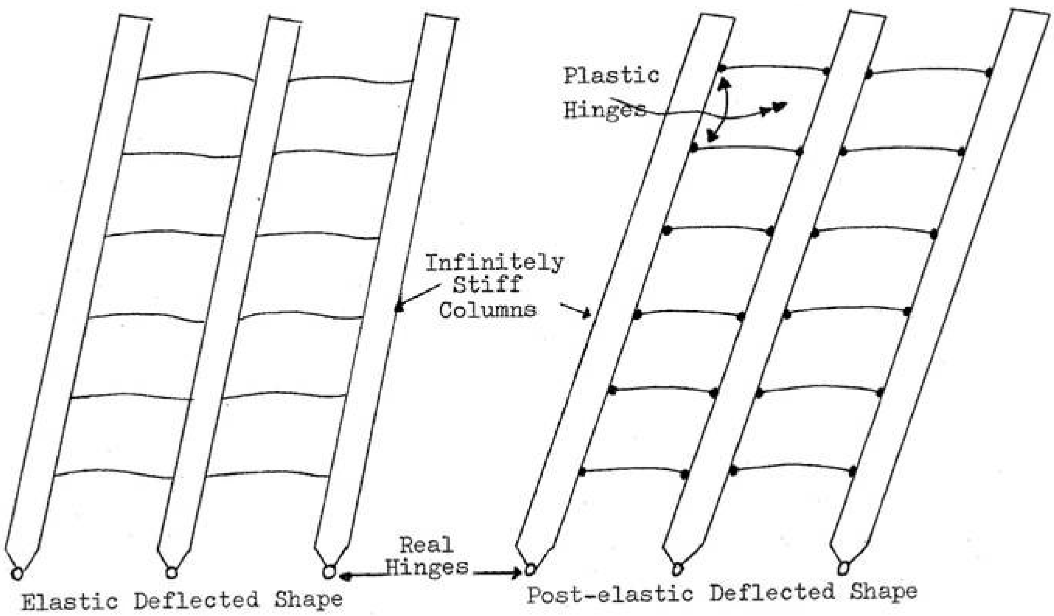


Figure 4. Elastic deflected shape & Plastic hinge positions in a ductile frame. From Hollings 1968a.

In the second paper Hollings described in detail the proposed method for ductile design of concrete frames as applied to the 16-storey Jerningham flats built on Wellington's Oriental Parade. The initial shear core structure was rejected for peripheral ductile frames with beamless flat plate floors. The building was actually designed in 1965 before NZSS 1900 Chap 8 had been published and the then current SEAOC Code (1963) was used to obtain the seismic coefficient. When Chapter 8 was finally released the building was reanalysed with the Code's recommended seismic loading, which was twice that of the SEAOC Code, see Figure 5. The paper describes how to proportion the seismic forces between frames, checking the elastic period from the deformations, allowing for torsion and how to find the design actions.

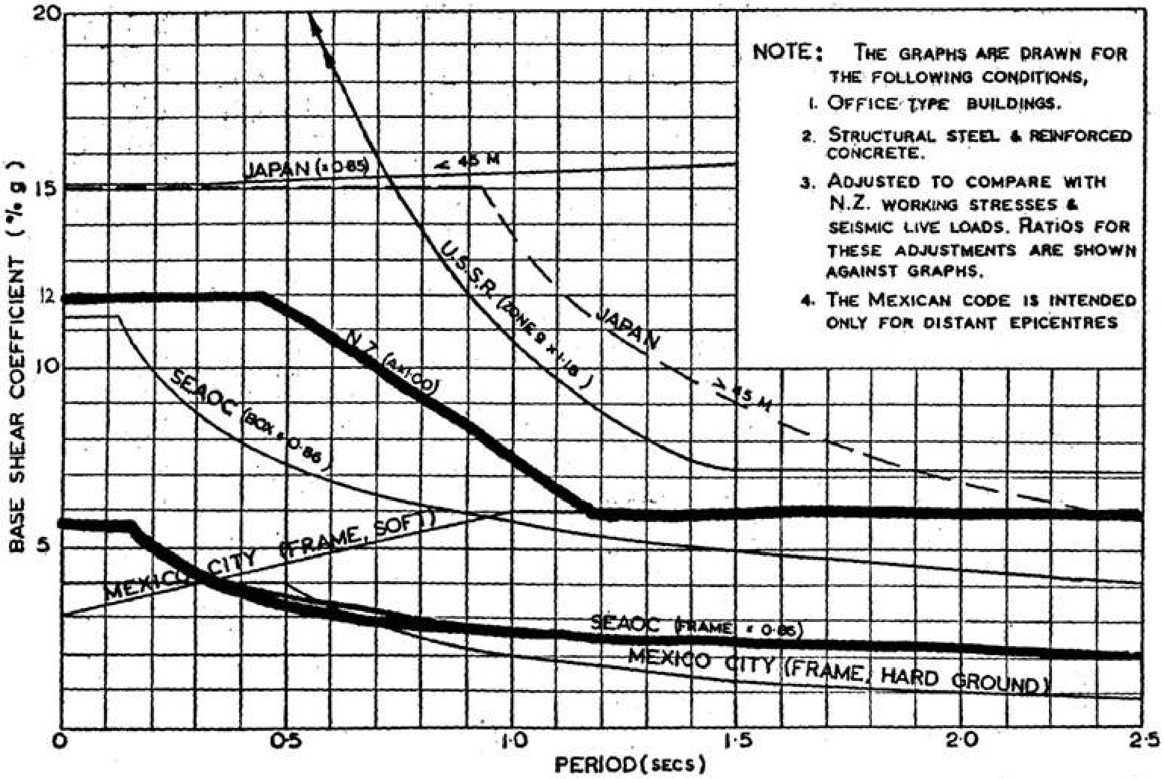


Figure 5. Comparison of Basic Seismic Coefficients 1965. From Hollings 1968b.

The hinge zones were detailed to occur at the beam ends with a uniform load factor of 1.45 and by matching the beam strengths to the elastic analysis actions as closely as possible, there being no elasto-plastic dynamic analyses available (later checked by Walpole and Shepherd). The columns were designed for overstrength by using a 1.25 minimum margin above the beam strengths, thus the columns were 1.25 x 1.45 = 1.81 times stronger than the code strength levels. This was certainly the beginnings of capacity design as we now know it. The problem of the corner columns going into tension under seismic conditions was described and the author admitted that not enough main column steel was detailed for that case. However Hollings felt that this was satisfactory as columns yielding in tension didn't mean a loss of ductility. Within the PHZ a "load factor" of 1.25 was used to preclude rupture of the HY60 reinforcing and the maximum steel strains were set at 2.5%, which produced a maximum "safe" hinge rotation of 0.01 radians (about a quarter of the current recommended reversing beam hinge plastic rotation in the revised NZS3101:2006). Generous amounts of binding (confining) and shear steel were provided in the hinge zones under full moment reversing conditions.

How to design the column shear ties created a problem and Hollings decided not to design for reversing plastic hinges at the top and bottom of each column as he felt this condition was unlikely to occur, even though higher mode effects were considered. He used the only good source of detailing recommendations at that time (Blume Newmark and Corning 1961) to find the proportion of column ties specified. Some of the detailing of the stirrup anchorages might make us shudder today, but in 1965 the design and detailing was years ahead of its time.

About this time (1965 in fact) Bob Park returned to New Zealand from Bristol University to a senior lecturing position at Canterbury University. The author was lucky to have Bob as his reinforced concrete lecturer in 1967 in 2nd Professional and ME project supervisor in 1970. Bob spent several lectures describing the ultimate strength design method for reinforced concrete based on the ACI (1963) concrete Code. This was all relatively new as reinforced concrete has been designed up to that time by the allowable stress method (like the other building materials). Together with Tom Paulay they spent the rest of their research careers (almost 40 years each) working on the behaviour, design and detailing of structures designed for earthquake loading. Many of the unknowns described by Skinner, Hollings and others were given the full research treatment, both experimentally and theoretically. These included the performance of plastic hinge zones in beams, columns and walls. Park wrote a leading paper for the NZIE Journal, *NZ Engineering* in 1968 where he clearly described the differences between the structural ductility factor and the required curvature ductilities at each plastic hinge for both beam-sway and column sway mechanisms. Usefully, how to estimate these curvatures was described in detail. Park & Paulay's pivotal work "Reinforced Concrete Structures" was published in 1975 and has been used internationally by designers and students for decades since.

In the late 1960's and early 70's the then Ministry of Works was designing many public buildings around NZ and due to the lack of guidance in the 1965 concrete code on how to obtain the required "adequate ductility" the Structural Design Office produced the booklet PW 81/10/1 :1970 the Code of Practice: Design of Public Buildings". This Code was written by the inspirational Chief Structural Engineer of the time, Otto Glogau with the help of his assistant Gordon McKenzie (*NZSEE Life Member).* Otto Glogau became the chairman of the drafting committee for the next version of the loadings code, NZS 4203, which was first published in 1976 and reprinted with amendments in 1984. Unfortunately Mr Glogau died when overseas after attending the 1980 World Conference on Earthquake Engineering in Turkey, at the peak of his career.

NZS4203 was a major advance on the 1965 Code allowing the strength method of design as well as the "alternative" allowable stress method. It included for the first time a structural type factor and a material factor, both to be incorporated into the estimation of the base shear coefficient. The structural type factor (S) was given as 0.8 for ductile frames and ductile walls with an adequate number of possible PHZs. Higher values of S were given for less ductile structural types. The material factor, M was 0.8 for structural steel and reinforced concrete and 1.0 for prestressed concrete and reinforced masonry. The inverted triangular distribution of seismic forces was continued with the proviso that 10% of the base shear should be added to the top floor for buildings with a height/width ratio greater than 3. This was to include some contribution of possible higher mode behaviour in slender buildings.

The concrete ductile moment-resisting space frame appendix contained many of the detailing clauses found in the current Concrete Standard NZS3101:1995 and the revised Concrete Code which was published in 1982. The concrete confinement "special transverse reinforcement" equations are very similar to the current equations, which were developed by Park *et al* (1980) and the design column shear was obtained from the moment capacity of the column at its top and bottom divided by the clear height. Near identical to the current limits on the amount of beam and column reinforcing (minimum and maximum) were specified in PW 81/10/1, splices were prohibited in PHZs and beam maximum shears were found from the sum of the PHZ beam capacities together with a factored gravity load component. At this time some shear was able to be carried by the concrete in the PHZ. It was recognised that beam and column ties did three jobs, confine the concrete core, resist the maximum (overstrength) shear and restrict the main bars from buckling, although the maximum spacings (d/4, 16 bar diameters or 300 mm) weren't as restrictive as currently (6 bar diameters for anti-buckling).

During the mid-70's a study group of the NZNSEE was set up to produce recommendations for the design and detailing of ductile structures culminating in a series of papers published on frames in the *Bulletin* in 1977 and on ductile walls in 1980. These recommendations covered much of the research completed in NZ and overseas over the previous decade, especially research carried out at Canterbury and Auckland Universities and at the Ministry of Works Central Laboratory by a large number of postgraduate students and staff under the supervision of Professors Park and Paulay and Drs Nigel Priestley and Richard Fenwick. As a result the entirely new concrete code and commentary (NZS 3101) published in 1982 was a state-of-the art code of practise. As Tom Paulay commented at the 1987 Pacific Conference on Earthquake Engineering (NZ Engineering 1987) ductile capacity design was a method where the designer "tells the structure what to do" and "In spite of its simplicity, this design approach should ensure excellent inelastic structural response, provided that, as a complementary task, all critical regions are judiciously detailed". This is indeed the nub of the matter.

Of particular interest to the author was the performance of reinforced concrete beam-column joints under seismic conditions beginning with testing small exterior joints at Canterbury University, Fig 6 (Megget & Park 1971) and interior joints at MoW Central Labs (Blakeley *et al* 1977). The 1965 Code had no requirements for transverse shear ties in beam-column joints and the first tests confirmed that large amounts of transverse ties were indeed required in this highly stressed zone. PW81/10/1 required transverse confining ties in the joints as well as ties designed to carry the shear forces from (all) the beam reinforcement at a stress assumed at yield stress. Note that biaxial beam yielding was to be assumed, a tougher requirement than currently. The joke going around the MoW Structural Design office in the early 70's was that if a canary could fly out of a beam-column joint reinforcing cage then the designer had made a mistake in his calculations! Today the canary has mutated to a slightly larger variety, maybe a blackbird, but definitely not a pigeon? Over the years the design equations for joints have been amended as more research has been completed but really the only refinements have been how much joint shear can be carried by the joint core concrete and the provision of vertical joint shear reinforcing, which was totally ignored initially until Roger Blakeley, Bob Park and others studied the first strut and tie models of the joint and realised that something had to carry the vertical joint shears. This job is now assumed to be done by the intermediate column bars because they aren't usually fully stressed under flexure but some designers find that hard to believe! Again time will tell, I guess.

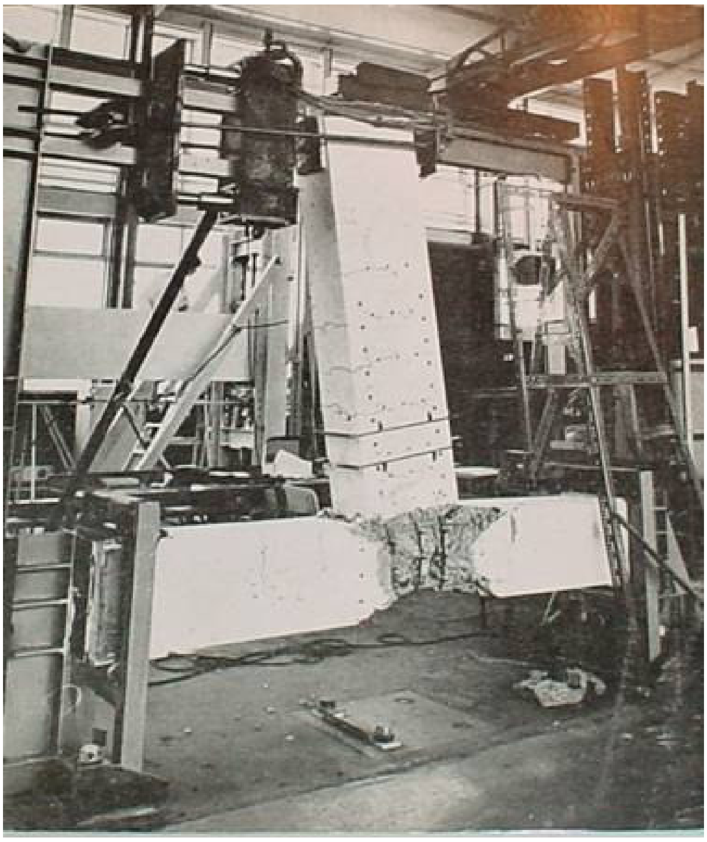


Figure 6. Exterior Beam-Column Joint testing at Canterbury University.

From NZ Engineering, Vol. 26, No. 11, November 1971.

The problem of what to do with the existing earthquake risk buildings was addressed by a change in the Municipal Corporations Act in 1968 allowing boroughs and cities to categorise themselves as earthquake risk areas and thus take to themselves powers requiring the upgrading of buildings. Wellington City surveyed the cities older buildings documenting 405 earthquake risk buildings in the CBD. The Council notified owners that these buildings should be strengthened (to two thirds of the current code requirements) or replaced by 2000. The owners of the 187 risk buildings in the central city “Golden mile” were asked to rectify their structures by 1982. Fowler (1983) reported that the 187 buildings had been reduced to 93 by 1983 with the total CBP risk buildings being reduced by 33%. Much of the strengthening was accomplished with extra shear walls, diagonal bracing or buttressing and the tying of structural floors and walls together. Many brittle hazards such as parapets, clock towers, etc have been removed after the two damaging 1942 South Wairarapa earthquakes (M7 & 7.1) felt strongly in Wellington.

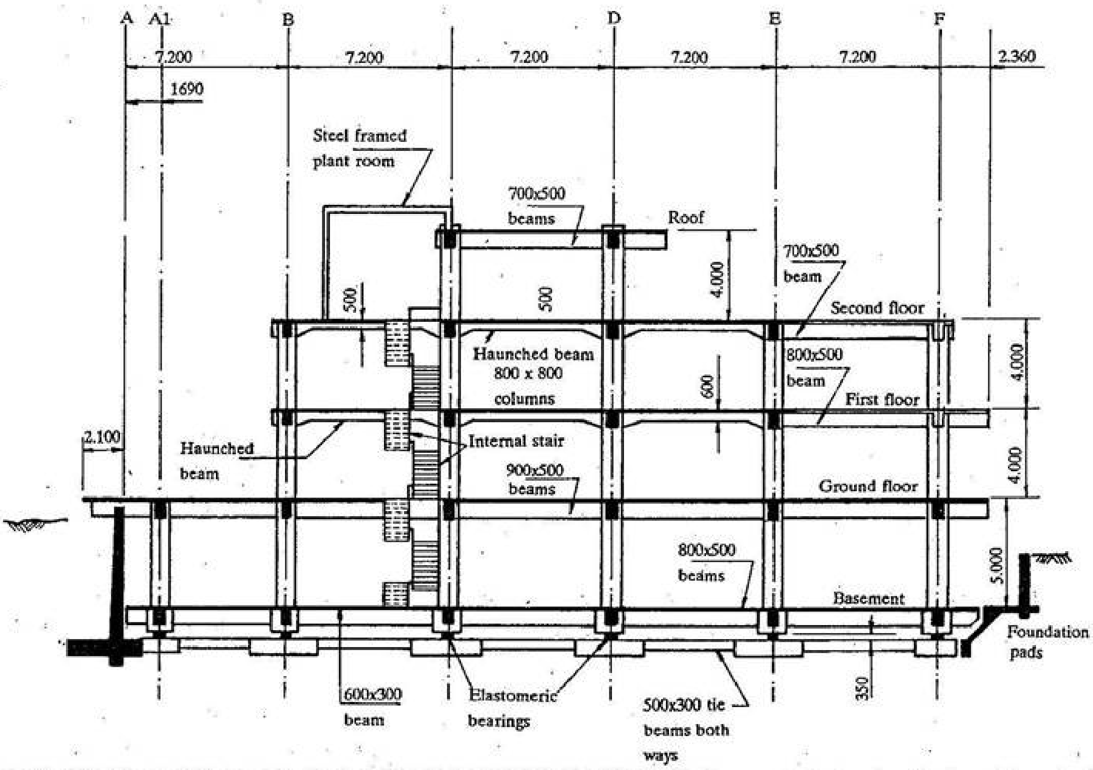
The 1992 version of the Loadings Code (NZS4203:1992) was a generation more sophisticated than the previous version. The Zone Factor map now comprised contours ranging from 0.6 up to 1.2 for central NZ and the base seismic hazard acceleration coefficient (response spectra) was plotted with different levels of ductility factor (1 to 10) for 3 different site soil conditions. Also included was a section on calculating the effects of P-delta deflections by adding extra lateral forces (strength) to the basic earthquake forces to allow for this effect, especially in tall slender structures. The extra analysis required was slow to be accepted but the requirements remain intact in the current seismic Loadings Standard (NZS1170.5). Much of this work was done by Richard Fenwick and Barry Davidson at Auckland University (Fenwick *et al* 1992).

4. INNOVATIVE SOLUTIONS

Base isolation is not a theoretically new technique used to reduce the lateral accelerations and thus forces in buildings; one of the first documented suggestions was a patent taken out by a New Zealander in 1929, which described a bed of soft shock reducing material placed and retained between the building’s base slab and its foundations. A US patent was granted in 1932. (de Montalk 1932).

From the early 1970’s the Physics & Engineering Laboratory of the NZ Dept of Scientific and Industrial Research (DSIR) developed and tested many forms of energy absorber to be used in the seismic or base isolation of structures. The aim of base isolation is to isolate the structure from its foundations by supporting the structure on (usually) horizontally flexible rubber bearings and absorbing much of the seismic energy by the use of lead dampers or yielding steel devices. The isolation increases the period of the structure above the isolators protecting it from the predominant ground motion frequencies. The most innovative energy absorber/isolator developed was the lead-rubber bearing (LRB), invented by Bill Robinson (Skinner *et al* 1980), where the elastomeric rubber bearing surrounds the lead core absorber. The first use of the LRB in a building was under the 4-storey William Clayton Building built within shouting distance of the Wellington Fault, see the building’s elevation and LRB elevation in Figure 7. This reinforced concrete ductile frame was designed with a “belt and braces” approach with near full ductile detailing in the PHZs and beam-column joints, even though dynamic inelastic analyses had shown that plastic hinging was unlikely even when the building experienced the then assumed “maximum credible earthquake”, explained by Megget (1978). Although the building was built with a 150 mm isolation gap around it, the latest predicted near fault peak velocity pulse likely may require a seismic gap nearer three times that provided. Thank goodness for the belt and braces approach used by the Structural Design office of the Ministry of Works & Development in the mid-1970’s!

Base isolation has been used in several major NZ bridges and major public buildings (Wellington Central Police Station and Te Papa) and in several instances it has been used as a strengthening technique (NZ Parliament Building). Unfortunately the technique hasn’t really taken off in NZ, as it has in Japan and the USA, and it is not commonly used here. The author is not certain whether this is due to the resistance to something new by structural engineers and architects or the inherent cost of the bearings and isolators. Also base isolation has not been codified in NZ, although recommendations were published in the *Bulletin* as far back as 1979 (Blakeley *et al* 1979). It is high time that these recommendations were updated and republished. Maybe we need a large damaging earthquake in a large city to persuade the professionals and owners that seismic isolation indeed reduces the damage both structurally and to the contents and that isolated structures are likely to be operational as soon as the electricity power supply is reconnected.



Section through the William Clayton Building, Wellington, New Zealand. The lead–rubber bearings are shown beneath the basement.

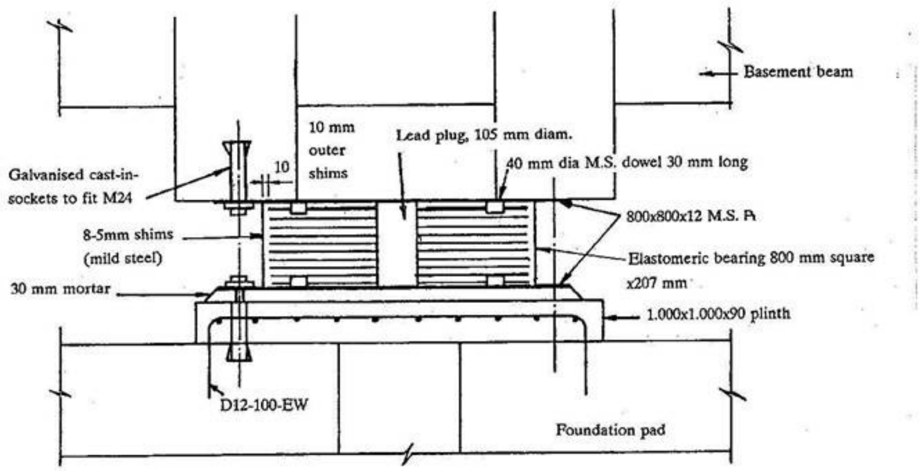


Figure 7. William Clayton Building elevation and elevation of Lead Rubber Bearing and fixings. From "Concrete Structures in Earthquake Regions", Edmund Booth.

5. THE 1990'S : CONSOLIDATION AND REFINEMENT

The current version of the Concrete Standard & Commentary (NZS3101) was published in 1995 with many refinements made to the earlier document. The Commentary (Part 2) was expanded to over 250 pages with much needed extra description. This volume became almost as much used as the Part 1 of the Standard. Figure 8 shows the commentary figure showing the localities of plastic hinges in a beam, which was only described in Part 1.

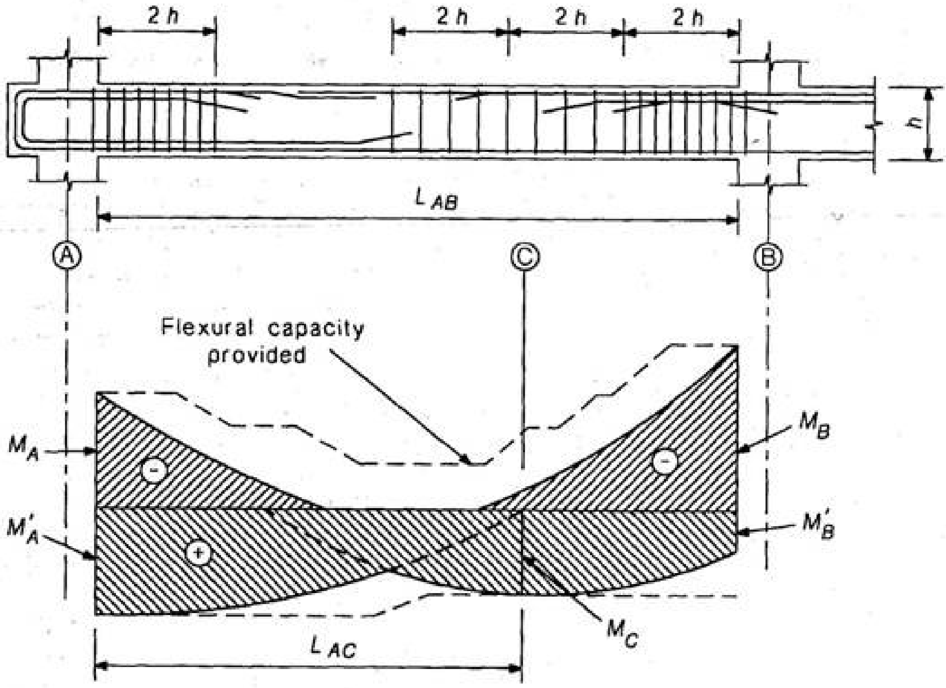


Figure 8. PHZ lengths for reversing and non-reversing beam hinge.
From NZS 3101:1995, Part 2, Commentary.

The major new problem of the 90's investigated was the effects of elongation caused by the lengthening of plastic hinge zones during an earthquake (Fenwick & Megget 1993). The authors concluded that this elongation could have dire effects on the performance of prestressed precast concrete floor slab units during a major shake, with the possibility that flooring units could fall off their seats. This proved to be the case in a full-scale test completed at Canterbury University (Matthews 2003) and amendments to the Standard were processed in an attempt to negate the problem. However there are hundreds of multi-storey buildings out there, many built quickly and cheaply(?) during the boom years of the late 80's, with small seating lengths for their slab units. The author tries not to have sleepless nights over how these buildings might behave during "the big one". Amendments were also made to the column design due to the proposition that PHZs could form either just below or above first floor level when beam elongation pushed the columns outwards relative to the usually fully fixed ground floor beams restrained by the foundation system.

Another recent problem has been the availability of 500 MPa reinforcing with little prior testing as to its effect on bond stresses through joints and whether it could be rebent or welded on site. As a result of recent tests the use of 500E grade reinforcing has been restricted in ductile beams when the reduced inter-storey drift limits are exceeded (Brooke *et al* 2003).

Displacement-based design received a lot of attention through this decade, as an alternative to the familiar force-based design, particularly by Nigel Priestley. The arguments still seem to be raging but as we enter the second half of the first decade of the new millennium both methods are being used, hopefully in situations where the particular method is best suited.

As is usual Standards NZ practise, in 2003 a committee was formed to consider a full rewrite of the 1995 Concrete Standard. This work is almost complete as I write this in February 2006 and the new Standard is likely to be published by mid-year. Although the number of totally new clauses is not as great as the previous edition there are new chapters on prestressed concrete and the strut and tie method, as well as a refinement of the capacity design approach, as a result of research work over the past decade.

6. THE FUTURE

The biggest question facing us is do all these seismic ductile design and detailing techniques actually work in the real life experiment (viz. the major earthquake) for the Concrete Standard is about 20 times larger than it was in 1965? We can only wait and see but there has been plenty of evidence that in medium magnitude 'quakes, particularly in Japan and California, modern ductile design and detailing can save the structure and the people in them. I think we still have work ahead on design levels of interstorey drift for the major earthquake, separation and fixing of architectural features (glazing, panels, equipment, etc) and maybe the actual risk (rather than perceived) in lessor seismic zones, namely Auckland. The 2004 (NZS 1170.5) basic seismic coefficient has dropped for Auckland to about 70% of the 1965 Code level for private buildings (assumed ductility factor of 4) with a period of about 0.5 sec while for Wellington the coefficient has increased by about 40%. Has Ruamoko (the Maori God of earthquakes and volcanoes) taken that on board? Are we really sure about the accuracy of these seismic coefficients, or are we relying on more ductile capability in our modern structures? It should be noted that the required strength levels for Wellington in NZS1170.5 "are appreciably less than those found using UBC and IBC codes of practice and very much less than the corresponding values in Eurocode 8" (Fenwick, Lau & Davidson 2002). What about the damping of modern structures during a major earthquake? From my perspective there seems to have been very little advancement in this aspect in the last 40 years. We need to spend more time helping developing countries with their seismic design techniques, where the chance of death due to structural failure is many times of that in NZ. We have probably solved all the "simpler" problems in the design of reinforced concrete structures to resist earthquakes but there are still many unanswered questions, for example the torsional analysis and design for irregular inelastic multi-storey buildings, the performance of corner columns under bi-axial seismic attack and the influence of precast flooring systems on the seismic strength and behaviour of space frame and shear wall buildings.

7. CONCLUSIONS

Phenomenal progress has been made in earthquake Loading Standards and the seismic design and detailing of structures since the Hawke's Bay earthquake of 75 years ago. This manifests itself in the complexity and volume of the current Standards and the time and effort required by structural engineers, architects and contractors to comply with these requirements. Highly significant research work has been completed on many aspects of structural performance in earthquakes (both analytical and experimental) but we are still waiting for the "big one" in NZ to confirm that we have done it correctly with our capacity design approach, ductility requirements in plastic hinge zones and the detailing required to obtain these ductilities (be they global or for each PHZ). Due to the NZ approach being somewhat different to that employed in the US and Japan recent large earthquakes in those countries haven't been able to answer all our questions. NZ structural engineers believe that their design and detailing is ahead of that practised overseas but only time will tell if we are correct.

In conclusion I would like to thank and acknowledge the New Zealand engineers and scientists who have taken us to where we are today in the field of seismic structural design particularly John Hollings, Ivan Skinner, Robin Shepherd, Otto Glogau, Bob Park, Tom Paulay, Nigel Priestley, Bill Robinson and Richard Fenwick. Without you gentlemen our knowledge and practise would be very thin indeed. My small part in the last 40 years has been to follow Wellington College's motto (my *alma mater*): "To receive the light and pass it on". It has been a great privilege for me to have worked with most of the inspiring men above.

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