REVIEW OF CODE DEVELOPMENTS FOR EARTHQUAKE RESISTANT DESIGN OF CONCRETE STRUCTURES IN NEW ZEALAND

R. Park*

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SUMMARY

The development of codes for the earthquake resistant design of concrete structures in New Zealand since the 1931 Hawke's Bay earthquake is traced. The background to the developments in the design procedures through the years is discussed. Californian seismic design codes, lessons from past earthquakes, and the results of analytical and experimental research work, much of it conducted in New Zealand, have led to the current philosophy for seismic design in New Zealand as expressed by the 1976 SANZ loadings cods and the SANZ concrete design code about to be published. These codes state requirements for both adequate strength and ductility, and emphasize the importance of structural detailing to achieve satisfactory performance of structures during severe earthquake loading. This New Zealand seismic design philosophy for concrete building and bridge structures is reviewed. A summary of the seismic design provisions of the new SANZ concrete design code (NZS 3101) is given in an Appendix.

1. INTRODUCTION

The circum-Pacific seismic belt is responsible for about 80% of the world's earthquakes. New Zealand is situated in part of that seismically active belt. The early settlers in New Zealand, coming from non-earthquake countries, introduced few measures for earthquake resistance into their buildings. Codes for earthquake resistant design in New Zealand have gradually evolved since the Hawkes Bay earthquake of 3 February 1931. The Hawkes Bay earthquake had a magnitude of 7.9 on the Richter scale and caused 256 deaths. During the last two decades, particularly, much attention has been given to earthquake engineering, and seismic provisions now dominate most structural design procedures in New Zealand.

New Zealand has been fortunate in that since the Hawkes Bay earthquake major earthquakes have not occurred close to large population centres and therefore damage from earthquakes has not affected a great proportion of the population. For example, the Inangahua earthquake of 24 May 1968 had a magnitude of 7.0 on the Richter scale but occurred in a sparsely populated part of New Zealand and there were 3 deaths. However, other countries have suffered significant damage and loss of life. For example, the great Kwanto earthquake in Japan on 1 September 1923 measured 8.3 on the Richter scale and killed almost 140,000 people. The massive earthquake which struck the city of Tangshan in China on 27 July 1976 registered a magnitude of 7.8 on the Richter scale and killed an estimated 655,000 people.

Californian seismic design codes, lessons from past earthquakes, and the results of experimental and analytical research work, much of it conducted in New Zealand, have led to the development of the current philosophy and procedures for seismic design in New Zealand. The current design philosophy and procedures are expressed by the 1976 loadings code and the concrete design code which is about to be published. These codes state requirements for both adequate strength and ductility and emphasize the importance of structural detailing to achieve satisfactory performance of structures during severe earthquake.

This presentation will review the development of seismic design codes for concrete structures in New Zealand. The current design philosophy for seismic resistant concrete buildings and bridges which has emerged in New Zealand in recent years will also be summarised.

2. SEISMIC ACTIVITY

Earthquakes generally occur due to release of strain energy from highly stressed regions of the earth's crust. The build-up of strain energy is due to shear deformations caused by relative movement of crustal plates. The deformations accumulate until the shear stress exceeds the rock strength and strain energy is released when the rocks rupture and slide past one another along a fault line. The sudden movement along the fault line caused the complex set of shock waves known as earthquakes. The Richter magnitude of an earthquake is a quantitative measurement of the size of an earthquake at its source, and is calculated from the maximum amplitude measured by a Wood-Anderson seismograph at a distance of 100 km from the epicentre. Earthquakes with a magnitude 5 or greater generate ground motions sufficiently severe to be potentially damaging to structures.

Figure 1 shows the distribution of large shallow earthquakes measured in New Zealand. The earthquakes shown are

*Professor and Head Of Department of Civil Engineering, University of Canterbury, New Zealand.

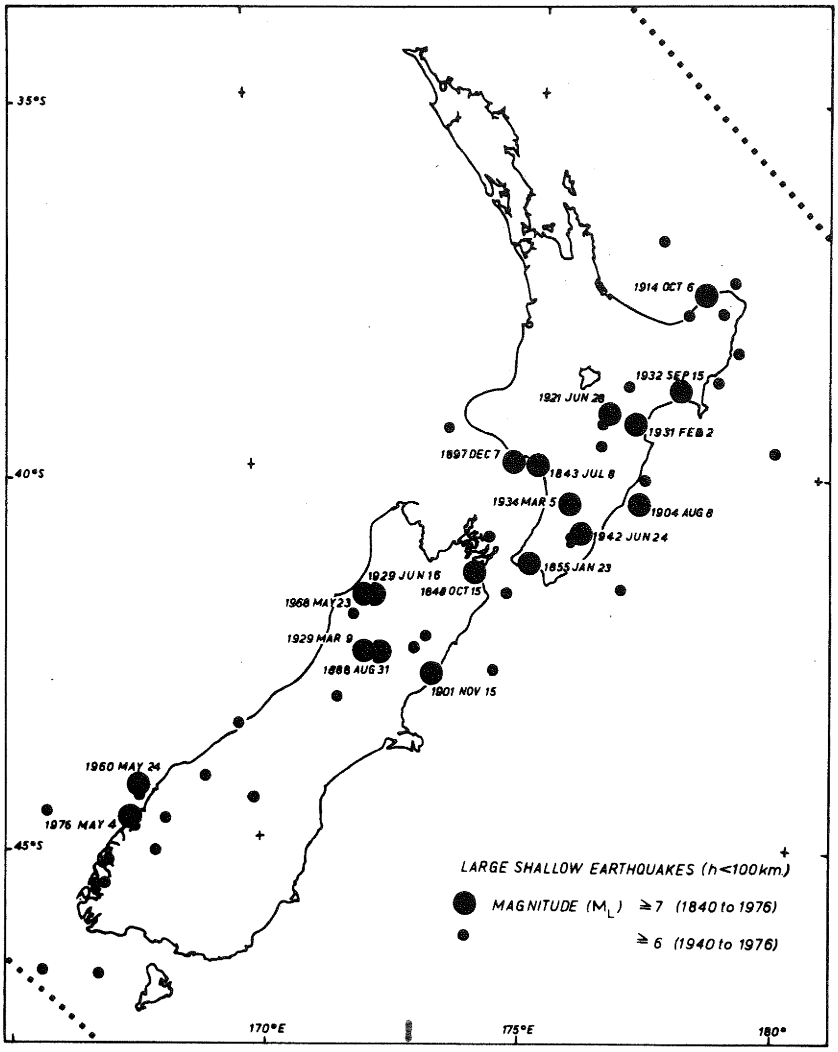


Figure 1 Large Shallow Earthquakes in New Zealand (1)

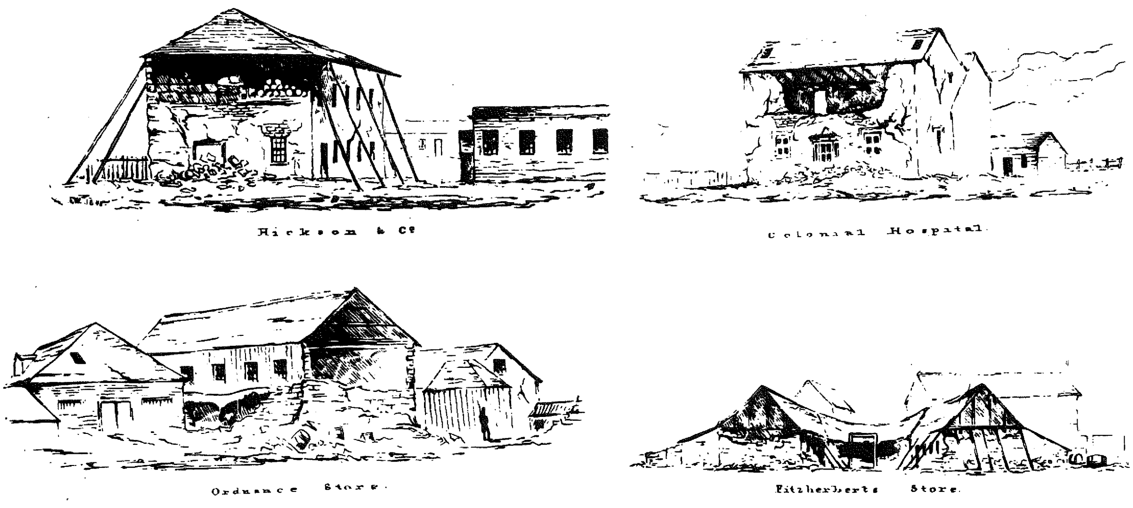


Figure 2 Sketches of Some Damaged Buildings in Wellington as a Result of the 1848 Marlborough Earthquake (Alexander Turnbull Library, Wellington)

those with Richter magnitude greater than 7 for the period 1840 to 1976 and with magnitude 6 to 7 for the period 1940 to 1976. Note that New Zealand has recorded earthquake data for about 140 years only, which is a relatively brief period (1).

Ground accelerations occurring during earthquakes have been measured by accelerographs and integrated to give the associated ground velocities and displacements. However most of these records have been obtained for earthquakes recorded overseas and strong motion records for New Zealand earthquakes have yet to be measured.

3. BEFORE THE 1931 HAWKES BAY EARTHQUAKE

New Zealand was subjected to a number of major earthquakes in the early years of European settlement, as is evident from Figure 1. Figure 2 shows some typical damage to buildings in Wellington as a result of the 1848 Marlborough earthquake which had a Richter magnitude of 7.1. These early warnings that special building precautions were needed for earthquake resistance went largely unheeded by settlers who had come from non-earthquake countries and had brought traditional European buildings procedures with them.

However there developed some interest in earthquake resistant design in the first quarter of the century. A notable New Zealand book published in 1926 was "Earthquakes and Building Construction" by C.R. Ford (2), who was principally an architect. This was probably the first book in the English language dealing directly with earthquakes in their relation to building structures. As well as presenting a good deal of New Zealand experience of earthquakes, Ford also summarised a number of Japanese and United States reports available at the time. Ford comments that Mr G. Hogben, Government Seismologist, had stated that "...earthquakes in New Zealand are a matter of scientific interest rather than a subject for alarm". Ford himself was clearly not of that opinion and he states "unless they are regarded as of more than academic interest, unless they are seriously studied in relation to their effects upon building structures and the lessons learnt be given practical application in their design, then earthquakes may indeed be regarded with dread". Ford observed that in many New Zealand municipalities by-laws giving requirements for earthquake resistant building construction were commonly defective.

The main recommendations for seismic resistant construction made by Ford were:

(a) Buildings should be designed to resist a horizontal force equal to 0.1 of the weight of the structure. (This followed the suggestion of an American, H.M. Hadley, and was based on Japanese experience in the great Kwanto earthquake of 1923 in which buildings designed to this horizontal force with low maximum stresses generally withstood the earthquake well).

(b) In reinforced concrete buildings care was needed at construction joints, reinforcement should be adequately anchored, haunches in beams are desirable, stirrups in columns should be closely spaced for some distance from the top and bottom during construction, and good workmanship and constant supervision during construction are vital.

(c) Buildings with complex structural systems should be avoided and simple symmetrical structural systems were preferred. In buildings with an L or U shaped plan, the re-entrant corners require particular consideration being the junction points of sections of the building with different periods of vibration.

(d) Foundations are of vital importance and should be designed for the horizontal forces imposed by the earthquake. Foundations should be well tied together with strong beams.

Ford also observed that in other countries building regulation committees had been set up after disastrous earthquakes. He urged that a committee composed of representatives of NZ Institute of Architects, NZ Society of Civil Engineers and NZ Federation of Building Employers, be set up to consider building regulations necessary to provide resistance to earthquakes before an earthquake created the need in New Zealand.

4. THE 1931 HAWKES BAY EARTHQUAKE AND ITS EFFECT ON EARLY CODES

The Hawkes Bay earthquake of 3 February 1931, measuring 7.9 on the Richter scale, caused extensive damage to buildings and was responsible for the loss of 256 lives. Fire followed the earthquake rapidly, completing the devastation caused by the earthquake. Figures 3 and 4 show close-up views of some of the business centres of Napier after the earthquake and fire and give an impression of the resulting devastation. Figure 5 shows the Nurses Home of Napier Hospital before and after the earthquake. Whereas load bearing masonry structures performed badly in the Hawkes Bay earthquake, buildings with reinforced concrete frames on the whole suffered very little structural damage and withstood the earthquake with remarkable success. In New Zealand this led to a shift in emphasis of building type from load bearing brick to framed buildings.

As a result of the Hawkes Bay earthquake a Buildings Regulation Committee, under the Chairmanship of Professor J.E.L. Cull of Canterbury College, was set up by Government with instructions "to prepare a report embodying such recommendations as it thought fit, with a view to improving the standard of building construction in the Dominion in relation to earthquake resistance". The Committee had its first formal meeting on 21 February 1931, which was eighteen days after the earthquake. Thirteen full days of meetings, and many other meetings of subcommittees of mainly Wellington members, produced a report in two main parts: Appendix 1 - draft general earthquake building by-law, and Appendix 2 - draft clauses for incorporation in a uniform building code for the Dominion.

Figure 3 Damage at Napier as a Result of the 1931 Hawkes Bay Earthquake (Alexander Turnbull Library, Wellington)

Figure 4 Damage at Napier as a Result of the 1931 Hawkes Bay Earthquake (Alexander Turnbull Library, Wellington)

Appendix 1 came into effect immediately as an emergency measure to ensure that buildings in the course of design in New Zealand should be built to a better standard than prevailing in the past. It essentially covered the ground already outlined by Ford (2). It endorsed design for a horizontal force equal to at least 0.1 of the weight carried by the building. The weight carried by the building was defined as the dead load plus a specified proportion of the live load. Stresses found by elastic (straight line) theory due to this earthquake loading plus vertical gravity loading were not permitted to exceed the working stresses allowed for vertical load alone by more than 25% in the case of reinforced concrete. It emphasized the importance of having brick and other types of walls securely tied together at the level of each floor, and also the importance of inter-connecting all foundation footings. It required that the structural system resisting horizontal loading by symmetrically located about the centre of mass of the building or else proper provision made for torsional moment on the building. It required that any building for public meetings should be constructed with a structural frame of steel or reinforced concrete.

Appendix 2 of the report was the initial move to provide a uniform building code throughout New Zealand. It could only be introduced after consultation with the local bodies which would be required to administer it, and whose existing individual codes it would replace. A further appendix to the Report contained reports on damage observed at Napier and Hastings which emphasized the extent of damage resulting solely from the absence of supervision during construction.

The recommendations of the Committee's Report led to the 1935 Standard Model Building By-Law, followed by the 1939 NZSS 95 New Zealand Standard Code of Building By-Laws. It is of interest to note that Part 5 of this 1939 By-Law dealing with reinforced and plain concrete construction was very much influenced by British concrete practice at the time. Working stress design was recommended.

In 1940 a book by S.I. Crookes (3) entitled "Structural Design of Earthquake-Resistant Buildings" was published in New Zealand. Crookes was on the staff of the School of Architecture, University of Auckland. His book is a milestone text for structural engineers involved in earth-quake resistant design. The book was particularly helpful in outlining design procedures for multistorey frames, and drew a great deal on Californian practice at the time. Crookes recommended for seismic design the use of horizontal loading found from the building weight multiplied by an assumed seismic coefficient. Crookes recognises the following assumptions in this procedure, some of which are more acceptable than others: the choice of an arbitrary seismic design acceleration, the assumption of weights concentrated at floor levels and roof, the neglect of vertical acceleration, the assumption of uniform acceleration up the height of the building, and the assumption that destructiveness is dependent on the acceleration only and resonance is ignored. However he defends this procedure as being a practical method for practising engineers, whereas the more sophisticated dynamic procedures available at the time, although in theory sounder, were still being hotly contested and could not be used with complete confidence. He discusses design methods at length, particularly with respect to deflection, relative rigidities of beams and columns of frames, distribution of shear loads between columns, frame analysis, torsion and overturning. He presents the 1933 Californian code (revised 1937 version) in full. He makes the interesting observation that "it is underestimating to say that most earthquake-resistant buildings of moderate dimensions involve at least twice as much structural design work as those of ordinary construction", and he agrees that "In fairness, the present scale of remuneration .... should be increased where earthquake-resistant designs are required". This is an outstanding book for its time and although published in 1940 it could still be of interest to earthquake engineers today.

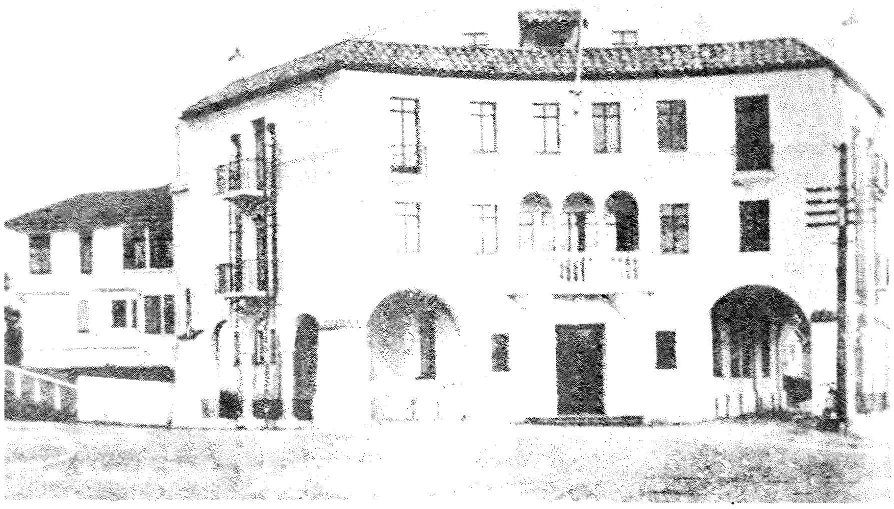
The NZSS 95 New Zealand Standard Model Building By-Law published in 1955 superceded the 1939 edition. In this 1955 By-Law, the horizontal loading on a public building was recommended to be either that given by a uniform seismic coefficient of 0.1 up the height of the building (0.08 was recommended for private buildings), or that given by a seismic coefficient which varied linearly from zero at the base to 0.12 at the top of the building (same for private buildings). The second option of a horizontal load distribution in the shape of an inverted triangle recognised approximately the deflected shape of the building in its first mode of dynamic response. Working stress design was recommended.

5. THE EARLY NZSS 1900 CODES

In 1964 and 1965 the New Zealand Standard Model Building By-Law relevant to concrete structures was first published as part of the NZSS 1900 series.

(a) Seismic Design Loads

Chapter 8, Basic Design Loads (1965), recommended horizontal seismic loading of $V = CW_t$, where W_t is the dead load plus the seismic live load and C is the seismic coefficient which depended on the seismic zone, the type of occupancy, and the period of the building. At the time there was only one strong motion record available of engineering significance, namely the record obtained at El Centro, California, in 1940 which had a maximum ground acceleration of 0.33g (see Figure 6). In the absence of local data the "standard" earthquake for regions of maximum seismicity in New Zealand was assumed to be similar to that of the El Centro earthquake. Figure 7 shows the acceleration response spectra for a single degree of freedom elastic system, with different amounts of viscous damping, responding to the 1940 El Centro N-S earthquake. The



Before Earthquake

After Earthquake

Figure 5 Nurses Home at Napier Hospital Before and After the 1931 Hawkes Bay Earthquake (Alexander Turnbull Library, Wellington)

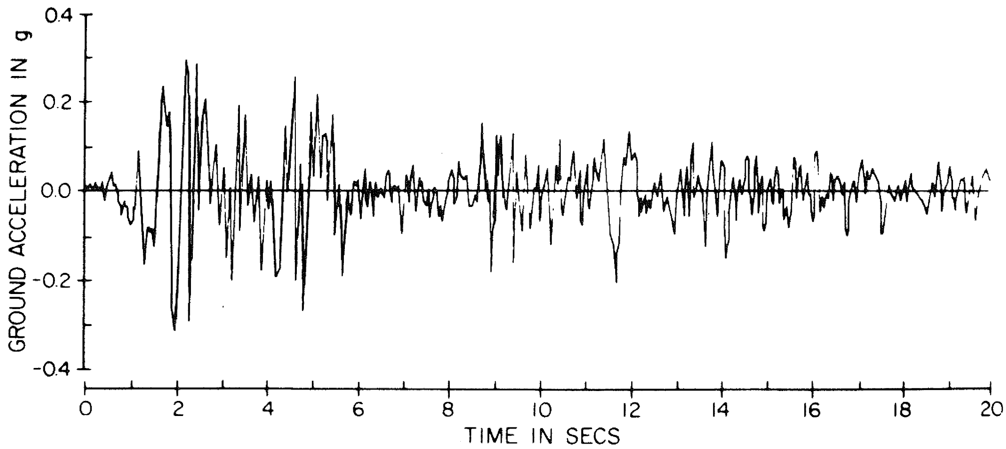


Figure 6 North-South Component of Ground Acceleration Measured at El Centro, California, at Approximately 6 km from the Causative Fault of the Magnitude 7.1, 18 May 1940, Earthquake. Recorded on Very Deep (+ 1500 m) Alluvium (7).

spectra in Figure 7 are plotted for no damping and 2, 10 and 20% critical damping, and give the maximum acceleration induced by the ground motion into the mass for different periods of natural vibration of the elastic system. Pioneering work was performed in New Zealand by Skinner (5) on the El Centro and other earthquake records. This led to the basic seismic coefficients in the code shown in Figure 8 which assumed for reinforced concrete structures a viscous damping of 10% of critical and a ductility factor of 4, but according to the commentary on the code "modified in the long period range to make it more suitable for a reasonably broad range of intermediate subsoils". Comparison of Figures 7 and 8 indicates the level of reduction of the horizontal loading as a result of the assumed viscous damping and ductility. These reductions brought the seismic coefficients down to a level which had been used successfully in many countries for a number of years, ranging from 0.16 to 0.08 for short period structures.

With regard to this recommended level of seismic loading the commentary of the code stated: "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 percent or 10 percent 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 building - 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 of earthquake resistance beyond their capacity when stressed only to the elastic limit".

Hence the code acknowledged the importance of ductility. However its requirements for ductility were stated only in the following general form: "All elements within the structure which resist seismic forces or movements and the building as a whole shall be designed with consideration for adequate ductility". No guidelines were given as to how "adequate ductility" was to be achieved. The commentary to the code stated that a safeguard is to limit "the use of reinforced masonry buildings to low structures of minor importance and by building in reinforced concrete in the intermediate field and in structural steel of adequate ductility for taller structures and for those of importance to the community".

The boundaries for the three seismic zones shown in Figure 9 were according to the commentary of the code, "defined taking into account historical records of past damaging earthquakes, the scatter of recorded epicentres, evidence of ground disturbances in reasonably recent times and general geological considerations". The probability of earthquakes of all intensities was considered to lessen as one proceeded from Zone A to Zone C. It should be noted that the decision concerning seismic zoning was extremely controversial. In particular, the majority of staff of the Seismological Observatory were strongly opposed to any form of seismic zoning since they maintained that statistical analyses of available data from instruments in New Zealand did not support zoning. Also, it is apparent that some account

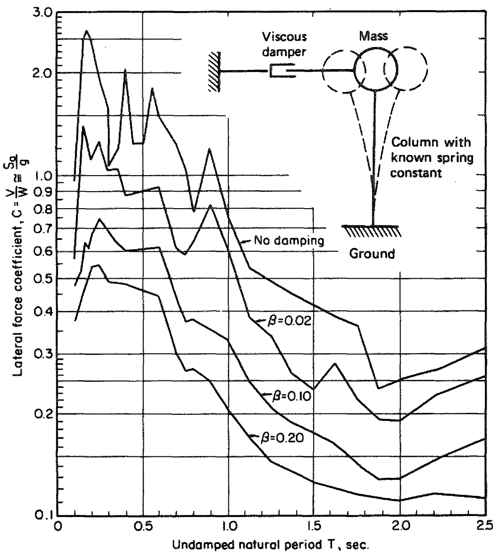


Figure 7 Acceleration Response Spectra for Elastic Single Degree of Freedom Systems, 1940 El Centro Earthquake N-S Component [4]

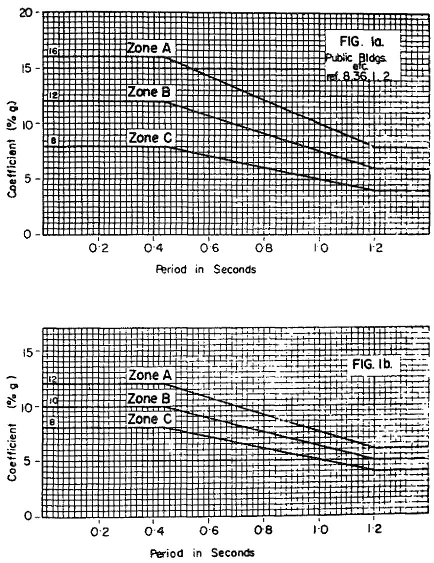


Figure 8 Seismic Coefficients from NZSS 1900 Chapter 8, Basic Design Loads, 1965. (Fig. 1a is for Public Buildings, Fig. 1b is for Other Buildings)

was taken of other than tectonic pressures in deciding the zone provisions. For example, it appears that the Zone C region around Dunedin was possible because that part of Otago saw an advantage in being in an "earthquake quiet" zone.

The distribution of the total horizontal seismic force up the height of a typical building was recommended to be the inverted triangular shape (with the greatest force at the top, except that for slender buildings (height/width > 3) 10% of the total force was concentrated at the top and the remaining 90% was distributed in the usual way.

The use of dynanic analysis to determine design actions for structures was permitted providing that the design actions so found were not less than 80% of the values computed using the seismic coefficients of the static analysis procedure.

Note that the design actions found from structural analysis using the horizontal forces given by the seismic coefficients or dynamic analysis were what today would be referred to as "unfactored actions". That is, they were for use in working stress design.

The reason for the reduction in the elastic response inertia loads as a result of "ductility" in code designed structures was not explained very well in the commentary to the code. The explanation can be made quite simply with reference to Figure 10 and is given elsewhere, for example (4, 6, 7). A single degree of freedom oscillator when responding elastically will have a load-deflection relationship as presented in Figure 10a, where point b is the maximum response. The area abc under the curve represents the potential energy stored at the maximum deflection, and as the mass returns to the zero position the energy is converted into kinetic energy. If the oscillator is not strong enough to carry the full elastic response inertia load and develops a plastic hinge with elastoplastic characteristics, the load-deflection curve will be as in Figure 10b. When the plastic hinge capacity is reached, the deflection response proceeds along line de, and point e represents the maximum response. The potential energy stored at maximum deflection in this case is represented by the area adef: note that the forces acting on the structure have been limited by the plastic hinge capacity. When the mass returns to the zero position, the energy converted to kinetic energy is represented by the small triangular area efg, because the energy represented by the area adeg is dissipated by the plastic hinge by being converted into heat and other irrecoverable forms of energy. Thus it is evident that in the elastic structure the full stored energy is returned as velocity energy in each cycle, whereas in the elastoplastic structure only part of the energy is returned. Hence the potential energy stored in the elastoplastic structure in each cycle is not required to be so great as in the elastic structure, and the maximum deflection of the elastoplastic structure is not necessarily much greater than that of the elastic structure. In fact, a number of dynamic analyses (4,7) have indicated that the maximum deflection reached by the two structures may be approximately the same, except for structures with a small period where the maximum deflection reached by the elastoplastic structure will be greater. Behaviour based on the assumption of equal maximum deflections is illustrated in Figure 10c. A measure of the ductility of a structure is the displacement ductility factor μ defined as

$$\mu = \frac{\Delta_u}{\Delta_y} \qquad (1)$$

where Δ_u is the lateral deflection at the end of the postelastic range and Δ_y is the lateral deflection when yield is first reached. When a number of load cycles are involved, Δ_y is taken as the lateral deflection when yield is first reached in the first load excursion into the postelastic range. Hence, if the equal displacement concept is valid and a displacement ductility factor $\mu = 4$ can be attained by the structure, the design seismic horizontal load need only be $^1/4$ of the elastic response inertia load, as is illustrated in Figure 10c. Note that both the displacement ductility factor μ and the energy dissipation (that is, the area within the load-deflection hysteresis loop) are important. That is, the structure should be tough enough not to undergo a significant reduction in strength, stiffness and hysteretic damping during the reversed loading cycles in the inelastic range.

(b) Concrete Design

Chapter 9, Design and Construction, Division 9.3 Concrete (1964) recommended a working stress (elastic or straight line theory) approach for section design. This code was again based mainly on British practice. It gave no recommendations for detailing reinforced concrete structures for ductility, although the loadings code required structures to have "adequate ductility". Designers therefore tended to turn to United States design procedures for ductility, such as the recommendations of the Seismology Committee of the Structural Engineers Association of California and the book by Blume, Newmark and Corning (4). This meant that designers tended to use associated US reinforced concrete design procedures and the building code of the American Concrete Institute (ACI 318-63) became widely used in New Zealand in lieu of the NZSS 1900 chapter on concrete design. This ACI code permitted section design by either the working stress method or by the ultimate strength method. The ultimate strength approach has definite advantages when assessing strength and ductility under seismic loading and gained favour in New Zealand.

In 1970 the Standards Association of New Zealand published for use in conjunction with, and as a means of compliance with, NZS 1900 : Chapter 9.3A, a provisional New Zealand Standard Code of Practice for

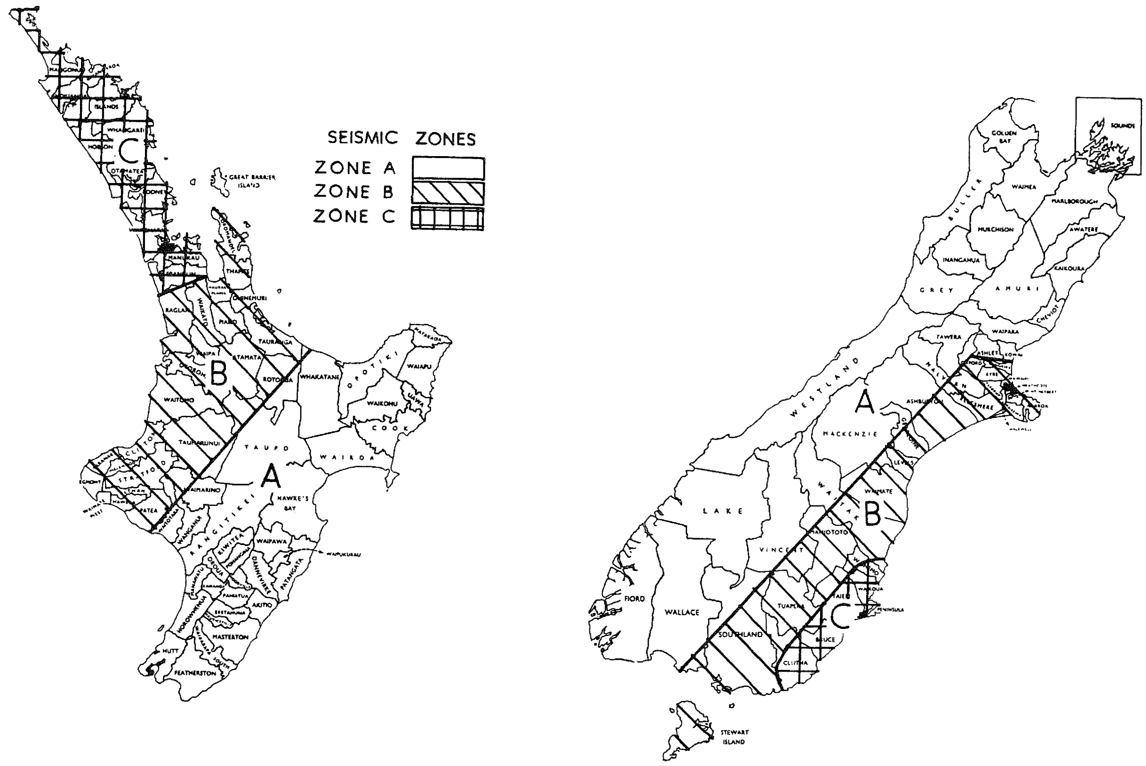
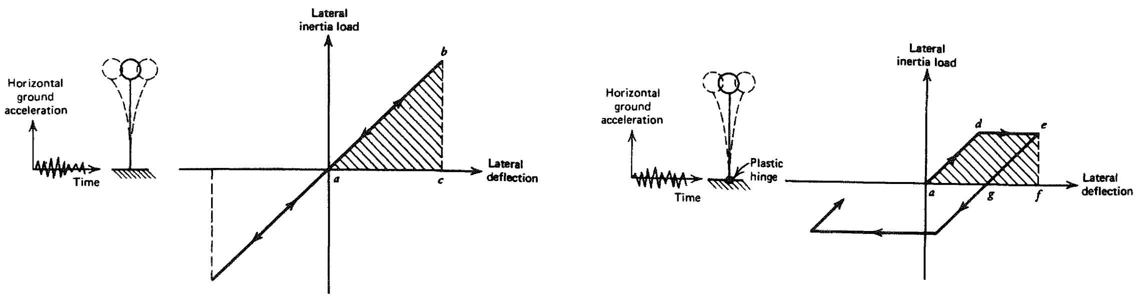
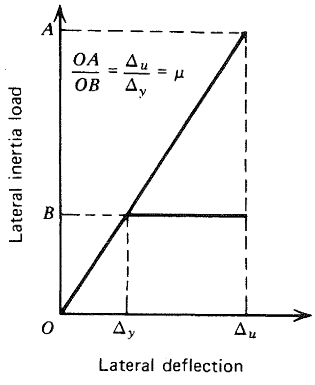


Figure 9 Seismic Zoning of New Zealand from NZSS 1900
Chapter 8, Basic Design Loads, 1965



(a)

(b)



(c)

Figure 10 Response of Single Degree of Freedom Systems to Earthquake Motions (6)
(a) Elastic Response
(b) Elasto-Plastic Response
(c) Assumed Equal Maximum Displacement Response of Elastic and Elasto-plastic Systems

Reinforced Concrete Design, NZS 3101P:1970. The main body of this code was based on ACI 318-63. For ductility provisions for seismic design, the code contained an appendix which recommended that a reinforced concrete frame should conform to the detailing requirements of the 1968 Revision of "Recommended Lateral Force Requirements" published by the Seismology Committee of the Structural Engineers Association of California (SEAOC), plus a few stated additional detailing requirements. Unfortunately NZS 3101P:1970 did not gain wide acceptance and use by the design profession in New Zealand. This was because by the time NZS 3101P:1970 became available in New Zealand the 1971 revision of the building code of the American Concrete Institute (ACI 318-71) had come to hand which made ACI 318-63 (on which NZS 3101P:1970 was based) out of date. ACI 318-71 emphasized the strength design method (strength design was previously known as ultimate strength design), relegated working stress design to a very small part of the code and referred to it as the "alternative design method", and included an appendix which gave special provisions for seismic design. For most of the nineteen seventies it was the practice of most designers in New Zealand to use ACI 318-71 along with seismic provisions from either its appendix or from a more recent issue of the SEAOC provisions.

It is also of interest that in 1968 the Standards Association of New Zealand published a New Zealand Standard Recommendation "Prestressed Concrete", NZR 32:1968, which was based mainly on the 1963 ACI building code and contained no recommendations for seismic design. However in 1966 the New Zealand Prestressed Concrete Institute had published its own Seismic Design Recommendations for Prestressed Concrete in which the ultimate strength design method is recommended and load factors for seismic loading are suggested which are greater than those used for reinforced concrete in recognition of the expected greater response of prestressed concrete structures to earthquakes. (This greater response is due to the greater elastic recovery and hence smaller dissipation of prestressed concrete structures). Other recommendations dealt with practical details, such as that all prestressing tendons should be fully grouted throughout their length, and that where structures incorporate mortar joints suitable binding or enclosures of the joint itself should be provided to prevent loss of material under seismic moment reversals.

6. THE CURRENT 1976 LOADINGS CODE FOR BUILDINGS

In 1976 the Standards Association of New Zealand published a Code of Practice for the General Structural Design and Design Loadings for Buildings NZS 4203:1976. This code was a revision, in the means of compliance format, of NZS 1900:Chapter 8 : 1965 and is the current loadings code.

(a) General Features

The code allows design by the strength method or by the working stress method. The working stress method is referred to as the "alternative method" in the code to emphasize that the strength method is to be preferred.

The code does not specifically state its aims with regard to the level of seismic loads. However in New Zealand, as in other seismic countries, it is generally accepted that in almost all cases it is uneconomic to design a structure to withstand the greatest likely earthquake without damage. In the event of a really major earthquake the aim is to prevent collapse, and thus save lives, but the structure may be damaged beyond repair and be a complete ecomomic loss. For the severe earthquake that could reasonably be expected to occur in the life of the building there should be little or no damage.

The code is extremely detailed in its provisions and it was not greeted with enthusiasm by the profession when it was first introduced because of its complexity.

(b) Design Loading

For the strength method the load factors have been derived from ACI 318-71. The design loads U involving combinations of service dead load D, reduced service live load L_R , and earthquake load E, should not be less than which ever of the following combinations gives the greatest effect:

$$U = 1.4D + 1.7L_R \qquad (2)$$

$$U = 1.0D + 1.3L_R + E \qquad (3)$$

$$U = 0.9D + E \qquad (4)$$

The reduced service live load L_R is found by multiplying the service live load by a reduction factor which depends on the use of the building and the tributary area of floor or roof supported by the structural member. In equivalent static force analysis the total horizontal seismic force V on the building is given by:

$$V = C_d W_t \qquad (5)$$

where W_t is the total reduced gravity load equal to service dead load plus a proportion of the service live load (typically one-third of the service live load), and

$$C_d = CISMR \qquad (6)$$

where C is the basic seismic coefficient which varies between 0.15 and 0.05 (see Figure 11) depending on the seismic zone (there are three seismic zones), the period of the structure, and the subsoil condition; I is an importance factor which varies between 1.6 and 1.0, depending on how essential it is that the building should be functional after a seismic disaster; S is a structural type factor equal to 0.8 for ductile frames with an adequate number of possible beam plastic hinges and ductile coupled shear walls, and having a higher value for less ductile structural types; M is a material factor

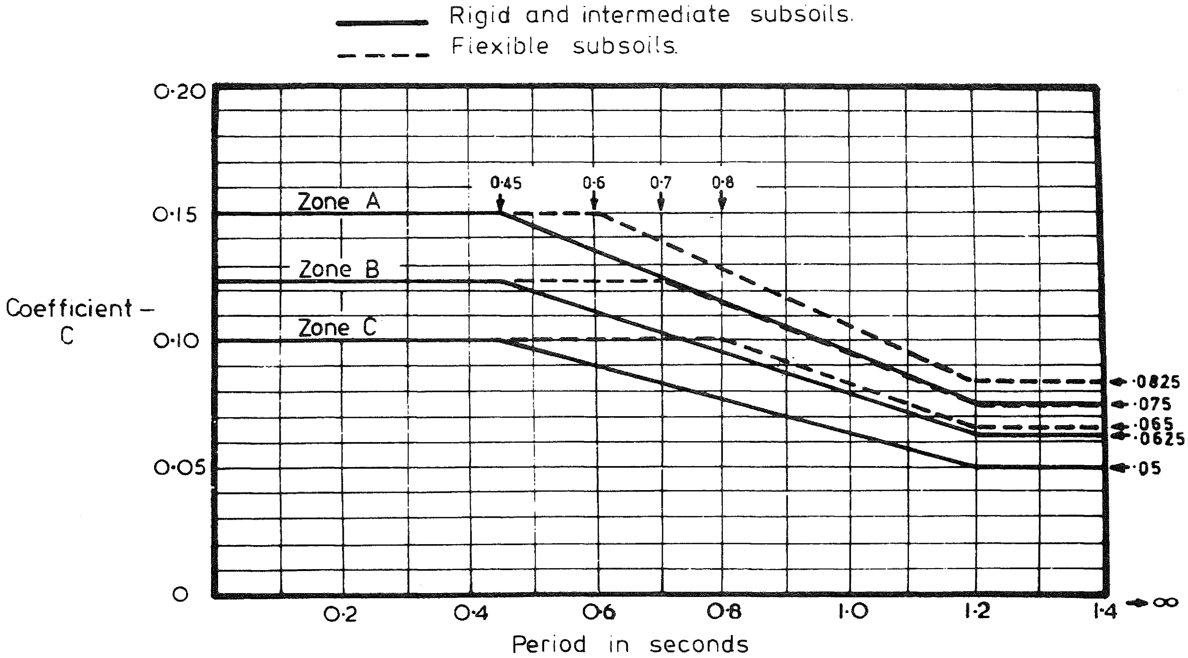


Figure 11 Basic Seismic Coefficients C from Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203:1976

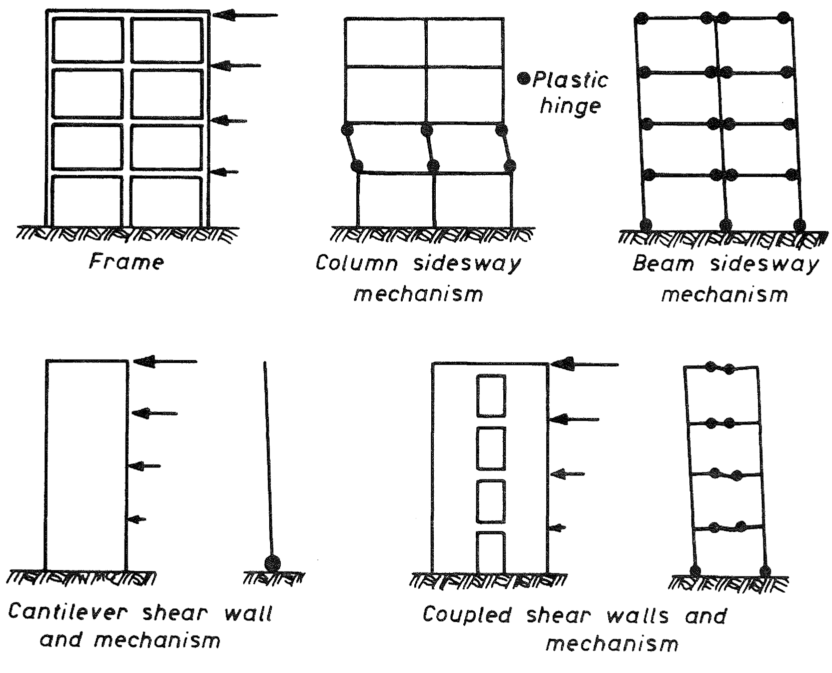


Figure 12 Building Structures With Seismic Loading and Possible Mechanisms

equal to 1.0 for reinforced concrete and 1.2 for prestressed concrete; and R is a risk factor equal to 1.0, unless the building accommodates large numbers of people or contains high risk chemicals or other materials when a greater value is used. For buildings with equal floor loads the distribution of V up the height of the structure has the shape of an inverted triangle, with the greatest horizontal load at the top, except that for buildings with a height to width ratio greater than 3, 0.1V is considered concentrated at the top storey and the remaining 0.9V is distributed as before up the height. To provide for shear resulting from torsional motions the applied horizontal force at each level is applied eccentrically with respect to the centre of rigidity at that level. Two equations for the eccentricity of the horizontal load are given, each a function of the horizontal dimension of the building and the distance from the centre of rigidity to the centre of mass, and the most unfavourable condition is used.

Dynamic analysis by spectral modal analysis may also be used to determine design actions. At any level the design shear so found should not be taken as less than 80% of the shear found from the equivalent static force analysis approach, and the base shear shall not be taken as less than 90% of C_dW_t.

(c) Ductile Frames and Walls

By way of general seismic design principles, the code requires that ductile buildings should be designed to be capable of dissipating significant amounts of energy inelastically under earthquake attack. Buildings designed for ductile flexural yielding should be the subject of "capacity design" and should have "adequate ductility". In the "capacity design" of earthquake resistant structures "energy dissipating elements or mechanisms are chosen and suitably designed and detailed and all other structural elements are then provided with sufficient reserve strength capacity to ensure that the chosen energy dissipating mechanisms are maintained throughout the deformations that may occur." An approximate criterion for "adequate ductility" given in the commentary of the code is that "the building as a whole should be capable of deflecting laterally in at least eight load reversals so that the total horizontal deflection at the top of the main portion of the building under the loading of Equations 3 and 4, calculated on the assumption of appropriate plastic hinges, is at least four times that at first yield without the horizontal load carrying capacity of the building being reduced by more than 20%. The horizontal deflection at the top of the building at first yield should be taken as that when yield first occurs in any main structural elements or that at the earthquake load E calculated on the assumption of elastic behaviour, which ever is the greater."

The code recognizes that it is reasonable to design the beams of two-way frame systems for seismic loading considered separately along each of the two principal axes of the structure. However it requires that columns or walls, including their foundations and joints, which are part of a two-way system, should be designed for the concurrent effects resulting from a general direction of seismic loading which causes the simultaneous yielding of all beams framing into the column or wall in the two directions.

Ductile frames, according to the code, should be capable of dissipating seismic energy in a flexural mode at a significant number of plastic hinges in the beams (ideally, in the beam sidesway mechanism of the frame of Figure 12). However, energy dissipation by column plastic hinge mechanisms (the column sidesway mechanism of the frame of Figure 12) is permitted in single or two-storey structures and in the top storey of multistorey structures. Column sidesway mechanisms are not permitted in tall buildings because such a mechanism can make very high curvature ductility demands on the column plastic hinges in the critical storey of such buildings, whereas for a beam-sidesway mechanism the curvature ductility demands at the plastic hinges in the beams and at the column bases is more moderate and can more easily be provided.

Ductile coupled shear walls, according to the code, should be designed so that the coupling beams yield before the walls, and the coupling beams should be detailed so as to be capable of dissipating significant seismic energy. Only when the yield capacity of the coupling beams, associated with the major portion of the overturning moment on the structure, is exhausted should the wall elements yield. Ductile cantilever shear walls should be designed to ensure that energy dissipation is by flexural yielding and that the wall will not fail prematurely in a non-ductile manner. Mechanisms for wall behaviour are illustrated in Figure 12.

The code does not expect designers to calculate the curvature ductility required at the potential plastic hinge sections and then to match the demand. Instead it states that "Adequate ductility shall be considered to have been provided if all primary elements resisting seismic forces are detailed in accordance with the requirements for ductile detailing in the appropriate material code."

(d) Buildings of Limited Ductility

If buildings have sufficient horizontal load strength to be capable of responding elastically to very severe earthquakes the need for special detailing is eliminated. The code permits small buildings of not more than 2 or 3 storeys height to be designed without any special detailing for ductility provided that the design seismic horizontal load is calculated using a structural type factor of S = 6. This amounts to using a design seismic load which is 7.5 times that used for the design of ductile frames with an adequate number of possible plastic beam hinges or ductile coupled wall structures (which have a structural type factor of S = 0.8). This is referred to as the "elastic response design procedure" in the code. Small buildings can sometimes be economically designed to have this high horizontal load strength and the permitted design procedure is then much less complex than for ductile structures.

A further approach allowed by the code for small structures detailed for only limited ductility, or capable of only limited ductility (rather than "full" ductility), is to use a design seismic horizontal load which is part way between the level for fully elastic response and that for ductile structures. This is referred to as the "low inelastic demand design procedure" in the code. This procedure may be used for frames of small buildings of not more than 2 or 3 storeys in height that have relatively large member cross sections. The design seismic horizontal load is calculated using a structural type factor of up to S = 2.4. This amounts to using a design seismic load that is up to 3 times that used for the design of ductile frames with an adequate number of possible plastic beam hinges or ductile coupled wall structures (which have a structural type factor of S = 0.8). This procedure allows some relaxation of the detailing requirements and a capacity design procedure is unnecessary.

(e) Interstorey Deflections

The code places a limit on horizontal interstorey deflections in order to limit the damage to non-structural elements. The code requires that the ratio of interstorey deflections to storey height should not exceed 0.010, unless the non-structural elements are effectively separated from the structure in which case this ratio may be 0.0006. The interstorey deflection is calculated, assuming elastic behaviour of the structure, from the horizontal forces, used in the design increased by multiplying by $2.0CI/C_d$ for structures dissipating seismic energy by ductile flexural yielding, by 2.0 for shear walls, and by $2.8CI/C_d$ for prestressed concrete seismic resisting systems and small buildings designed by the elastic response design procedure. These multiplication factors recognise that the actual maximum building deflection will be greater than that given by elastic theory since energy dissipation by ductile response has been assumed.

7. THE CURRENT LOADING CODES FOR BRIDGES

The current loading and other general criteria used in the design of bridges on public roads is set out in the "Highway Bridge Design Brief" issued by the Ministry of Works and Development (Civil Division Publication, CDP 701/D September 1978). This code states that ideally bridge structures should be designed so that earthquake induced energy is dissipated by members acting in a ductile manner. These may be main members (usually in the piers rather than the superstructure) or by introducing ductile energy dissipating devices between the superstructure and its supports. A capacity design procedure is recommended to ensure that energy dissipation occurs at the chosen location. The minimum earthquake load is derived from

$$V = C\,F\,W \qquad (7)$$

where F is an importance factor which varies between 1.0 for important crossings and frequently used bridges to 0.7 for less frequently used bridges W is the bridge total dead load, and C is the seismic coefficient which depends on the seismic zone (A, B or C as for buildings) and period of the structure and ranges between 0.16 for a short period structure in Zone A to 0.04 for a long period structure in Zone C. The earthquake load so found is multiplied by a load factor of 1.35 in strength design when its greatest effect is considered with other loadings. The design earthquake loading is based on the assumption that the structure can attain a displacement ductility factor of at least 4 and probably 6. Higher strength structures are permitted with reduced ductility requirements. Structural integrity during earthquakes is emphasized, such as positive horizontal linkage between adjoining parts of the structure and hold down devices. The SANZ concrete design code is specified for use in design of members. Other useful background is given in the publication "Ductility of Bridges With Reinforced Concrete Piers" issued by the Ministry of Works and Development (Civil Division Publication, CDP 810/A April 1975), but it is now recognised that this publication is excessively conservative in its requirements for transverse reinforcement in bridge piers.

Railway bridges are designed for earthquake loads similar to those for highway bridges.

8. THE 1980 AND 1981 CONCRETE CODES

In the late 1970s a good deal of effort was put in by Committees of the Standards Association of New Zealand in revising the concrete codes. Such revision was badly needed, especially in seismic design in order to match the requirements of the exacting 1976 Loadings Code (NZS 4203). The concrete construction and design requirements were separated into two separate documents, to serve as the technical requirements of NZS 1900 : Chapter 9.3A and a means of compliance with general by-law requirements.

(a) The Concrete Construction Code

In 1980 the current New Zealand Standard Specification for Concrete Construction (NZS 3109:1980) was issued. This code covers construction requirements for both reinforced and prestressed concrete. It is of interest that construction specifications for unbonded prestressing tendons are included for the first time in New Zealand.

(b) The Concrete Design Code

The Committee set with the task of revising the concrete design code commenced its meetings in 1976. A draft code was issued for comment in 1978 and again in 1980. At the present time the Committee has completed its deliberations and it is expected that the resulting New Zealand Standard Code of Practice for the Design of Concrete Structures (NZS3101) should be published shortly.

The new code covers both the design of reinforced concrete structures, prestressed concrete structures, and structures containing both prestressing tendons and ordinary reinforcing steel. The new code is based mainly on the 1977 ACI building code but with additional seismic provisions based on recent research and experience in New Zealand and elsewhere. Both buildings and bridges are covered by the code. New Zealand concrete design codes in past years had tended to be more directed towards buildings, and the decision to cover design requirements for both buildings and bridges was encouraged by the Ministry of Works and Development.

The task of the SANZ committee in preparing the new concrete design code involved extensive meetings since a great deal of new information on seismic design had become available from overseas. Also, much experimental research into the seismic resistance of structural concrete had been conducted in New Zealand since the mid-nineteen sixties, mainly in the laboratories of the University of Canterbury, University of Auckland and Ministry of Works and Development. This work involved properties of steel and concrete under seismic type loading; shaking table tests on model structures; pseudo-static load tests on reinforced concrete model frames and shear walls, beam-column subassemblages, slab-column subassemblages and slab-wall subassemblages; and pseudo-static load tests on prestressed concrete beam-column subassemblages. The experimental work was accompanied by analytical studies involving dynamic response of various structural systems to severe seismic ground motions, and theoretical studies of moment-curvature behaviour and strength and deformation characteristics of the range of structures considered. Most of this work has been reported in the literature and that dealing with reinforced concrete up to the mid-nineteen seventies is summarised in a book (6) and also in paper (8).

The consideration of these New Zealand and overseas developments in seismic design provisions was made easier for the SANZ concrete design committee by a series of meetings of discussion groups organised by the New Zealand National Society for Earthquake Engineering. The New Zealand Society for Earthquake Engineering was formed in 1968. Since 1974 it has been the New Zealand National Society for Earthquake Engineering. The aim of the Society is the advancement of the science and practice of earthquake engineering. Engineers, scientists, architects, contractors, and all who have an interest in earthquakes and their effects, are eligible for membership. The Society publishes a quarterly Bulletin containing a wide range of papers on earthquake engineering, and has organised conferences on earthquake engineering which have been of great value to the profession in New Zealand and neighbouring countries. In 1976 the Management Committee of the Society decided to arrange a series of workshops for structural engineers in New Zealand to make them more familiar with the 1976 buildings loadings code (NZS 4203), and with recent developments in seismic design procedures for (i) ductile moment resisting reinforced concrete frames, (ii) reinforced concrete walls and diaphragms, and (iii) bridges. It was realised however that there were many "grey areas" and the first step was to set up a discussion group in each of the three areas to obtain consensus views on the range of issues. Background papers in the form of design recommendations and commentary, representing the views of the discussion groups, were written by individuals and published in the Bulletin of the New Zealand National Society for Earthquake Engineering (9, 10, 11). The views expressed at these series of meetings, and the resulting background papers, were invaluable to the SANZ concrete design committee in its preparation of the new concrete design code. Nevertheless it took 42 one-day meetings of the SANZ concrete design committee to produce the final version of the code.

It is to be noted that the emphasis of current New Zealand codes in on good structural concepts and detailing. It is recognised that uncertainty exists regarding the selection of the mathematical model representing the behaviour of the building and the form of the imposed ground shaking. Major damage observed in overseas earthquakes has been shown to be due mainly to poor structural concepts (for example: column sidesway mechanisms and considerable twisting, due to a soft storey, or lack of symmetry and uniformity) and poor ductile detailing (for example: brittle connections, inadequate anchorage of reinforcement or insufficient transverse reinforcement to prevent shear failure, buckling of compressed bars and crushing of concrete). The aim in seismic design should be to impart to the structure features which will result in the most desirable behaviour, which implies establishing a desirable hierarchy in the possible failure modes of the structure. This philosophy may be incorporated in a rational capacity design procedure as previously defined. Note that a proper assessment of the strength and ductility of a structure cannot be made using the working stress design method. Hence the new concrete design code does not permit the working stress design method to be used. Instead, design is required by the strength method.

An attached Appendix A gives an outline of the seismic design recommendations of the new concrete design code. The background of many of these detailing aspects have been summarised recently elsewhere (6, 8-13).

9. POSSIBLE FUTURE DEVELOPMENTS IN SEISMIC DESIGN PROCEDURES FOR CONCRETE STRUCTURES

(a) Design Seismic Loading

It is evident that the present design spectra and other factors involved in the determination of seismic loading will be revised as further research information becomes available. Berrill, et al (11, 14) have pointed the way for future developments. They have proposed seismic design loads for bridges based on the seismic zone, the return period of the earthquake, the fundamental natural period of the structure, and the design value of ductility. The seismic zoning scheme is essentially similar to that in the existing buildings loadings code (NZS 4203), except that a smooth transition is proposed between Zones A and C. The consideration of the earthquake return period gives the designer more feel for the risk of the design value being exceeded. The Zone A elastic response spectrum, for a 150 year return period, is similar to the 1966 Parkfield, California, accelerogram, which was recorded close to the source of that M = 5.6 earthquake. The design load spectra are plotted for a range of displacement ductility factors μ between 1 and 6, thus allowing the designer to match the adopted seismic design load to the estimated ductility available from the structure. Figure 13 shows the spectra for the basic seismic coefficient proposed for Zone A for 150 year return period and 5% critical damping for various values of the displacement ductility factor μ. The elastic response spectrum is given by the curve for $\mu = 1$. When the period $T \geqslant 0.7$ seconds, the spectra for the other values of μ were obtained by multiplying the elastic response spectrum by $1/\mu$. When $T = 0$ there is no reduction from the elastic spectrum, and for $0 < T < 0.7$ seconds a linear variation between these extremes was used. Note that the use of a reduction factor of $1/\mu$ (that is, use of the equal displacement concept) is unconservative for periods $T < 0.7$ seconds. Figure 14 shows the proposed Zone A spectra (14) compared with some recorded strong motions, and the Zone A Ministry of Works and Development design spectrum for bridges. The proposed elastic spectrum ($\mu = 1$) in Figure 14 should be compared with 6 times the MWD curve since the MWD design procedure is based on $\mu = 6$. Note that at small periods the proposed spectra results in much higher design loads for ductile structures than is required by the existing spectra.

(b) Concrete Design Code

The field of concrete design has moved repidly in the last two decades. It has been a large step to bring New Zealand designers from the 1964 code which was based on working stress design, specified no provisions for ductility, and was in Imperial units, to the new concrete design code which is based on strength design, specifies and emphasizes detailing for ductility, and is in SI units. An effective code in the intermediate years could have been of assistance in helping designers make the transition. It is evident that the new code should be given a significant period to settle in. Further improvements will obviously be made, but hopefully these will be accompanied by simplification in presentation and in detailing procedures, rather than by further complications.

10. CONCLUSIONS

1. Before the 1931 Hawkes Bay earthquake, earthquakes in New Zealand were regarded by many as a matter of scientific interest rather than a subject for alarm. Requirements for earthquake resistant building construction were commonly defective. However there was some early informed opinion on earthquake resistant design in New Zealand. For example, the book "Earthquake Resistant Construction" by C.R. Ford, published by Whitcombe and Tombs in Auckland in 1926, was probably the first book in the English language dealing directly with earthquakes in their relation to building structures.

2. The 1931 Hawkes Bay earthquake caused extensive damage and the loss of 256 lives. New Zealand has been fortunate in that since that Hawkes Bay event, major earthquakes have not occurred close to large population centres and therefore damage from earthquakes has not affected a great proportion of the population.

3. Codes for earthquake resistant design in New Zealand have gradually evolved from the time of the Hawkes Bay earthquake. The first codes endorsed design for a horizontal force of at least 0.1 of the weight carried by the building and recommended the working stress design approach. The concept of a horizontal force distribution up the height of a building in the shape of an inverted triangle, to more closely follow dynamic effects, was first introduced in 1955.

4. The 1965 New Zealand standard for basic design loads recommended horizontal seismic loading which depended on the seismic zone, the type of occupancy, and the period of vibration of the building. In the absence of local data the standard earthquake for regions of maximum seismicity in New Zealand was assumed to be similar to that of the 1940 El Centro earthquake which had a maximum ground acceleration of 0.33g. The seismic coefficients for reinforced concrete structures were obtained from elastic acceleration response spectra assuming viscous damping of 10% of critical and a ductility factor of 4, but modified in the short and long period ranges. This code acknowledged the importance of ductility and required structures to be designed for "adequate ductility". However the 1964 concrete design code was based on working stress design and British practice, and no design recommendations were given as to how adequate ductility was to be achieved. Designers tended to turn to United States design procedures for ductility, such as the recommendations of the Structural Engineers Association of California (SEAOC).

5. In 1970 the Standards Association of New Zealand (SANZ) published the concrete design code NZS 3101P which was based mainly on the 1963

building code of the American Concrete Institute (ACI). For ductility provisions for seismic design, this code recommended use of the 1968 SEAOC code with a few minor changes. Unfortunately this 1970 SANZ code did not gain wide acceptance in New Zealand because the 1971 ACI building code made it out of date and was widely used in lieu of the SANZ code. The 1971 ACI building code emphasized the strength design method rather than working stress design, and included an appendix which gave special provisions for seismic design.

6. In 1976 SANZ published the current loadings code for buildings NZS 4203. This code requires that ductile buildings should be capable of dissipating amounts of energy inelastically under seismic attack. Buildings designed for ductile flexural yielding are to be subject of "capacity design". In capacity design, elements of primary lateral load resisting systems are chosen and suitably designed and detailed for energy dissipation under severe deformations. All other structural elements are then provided with sufficient strength so that the chosen means of energy dissipation can be maintained. The code requires the concurrent effects resulting from a general direction of seismic loading to be considered. Desirable modes of energy dissipation are recommended and seismic coefficients are based on the expected ductility of systems. This code is extremely detailed in its provisions. The code leaves it to the materials code (for example, the concrete design code) to specify the procedures for ductile detailing.

7. In 1976 the concrete design committee of SANZ set about its task of producing a code which would match the requirements of the loadings code. The committee has completed its deliberations and it is expected that the new code will be published shortly. The new code covers both buildings and bridges, from reinforced concrete and prestressed concrete. It is based mainly on the 1977 ACI building code but with additional seismic provisions based on recent research and experience in New Zealand and elsewhere. It will result in a high standard of detailing for earthquake resistance.

8. A considerable amount of research and development into the design of earthquake resistant reinforced and prestressed concrete frames, shear walls and bridges has been conducted in New Zealand in recent years. The design profession has taken a lively interest in this research and development. The New Zealand National Society for Earthquake Engineering and the National Roads Board have organised discussion group meetings which have helped maintain excellent communication between research workers and designers. An excellent interchange of views has taken place and agreement has been reached on many seismic design procedures. The task of the SANZ concrete design committee in preparing the new code has been made lighter by this excellent communication.

9. The emphasis of current New Zealand loadings and concrete design codes is on good structural concepts and detailing. It is recognised that uncertainty exists regarding the selection of the mathematical model representing the behaviour of the structure and the form of the imposed ground shaking. Major damage observed in overseas earthquakes has been shown to be due mainly to poor structural concepts (for example: column sidesway mechanisms and/or considerable twisting, due to soft storey, or lack of symmetry and uniformity), and poor ductile detailing (for example: brittle connections, inadequate anchorage of reinforcement or insufficient transverse reinforcement to prevent shear failure, buckling of compressed bars and crushing of concrete). The aim in seismic design is to impart to the structure features which will result in the most desirable behaviour, which implies establishing a desirable hierarchy in the possible failure modes for the structure. This philosophy may be incorporated in a rational capacity design procedure. A proper assessment of the strength and ductility of a structure cannot be made using the working stress design method. Hence the new concrete design code does not permit the working stress design method to be used; instead, design is required by the strength method.

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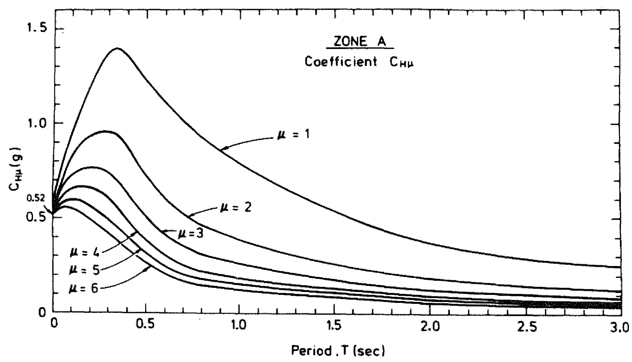


Figure 13 Basic Seismic Coefficient Proposed for Zone A for a 150 year Return Period and 5% Critical Damping [11]

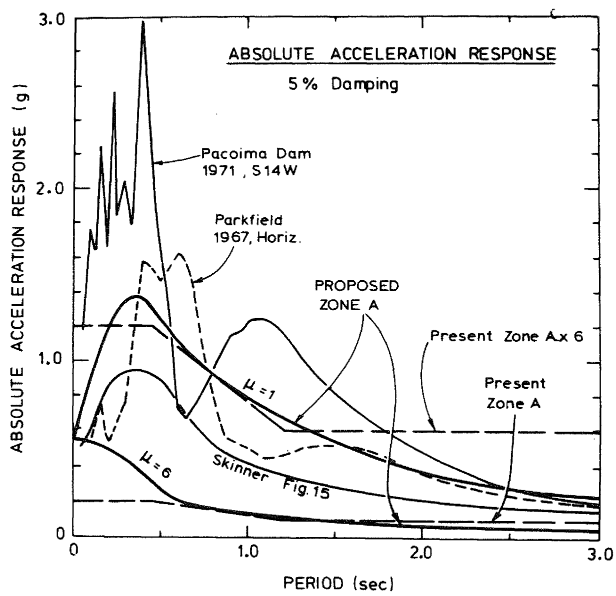


Figure 14 Elastic Acceleration Response Spectra for Various Earthquakes Compared With Proposed Zone A (150 year Return Period) Spectra and Existing MWD Bridge Design Spectra [14]

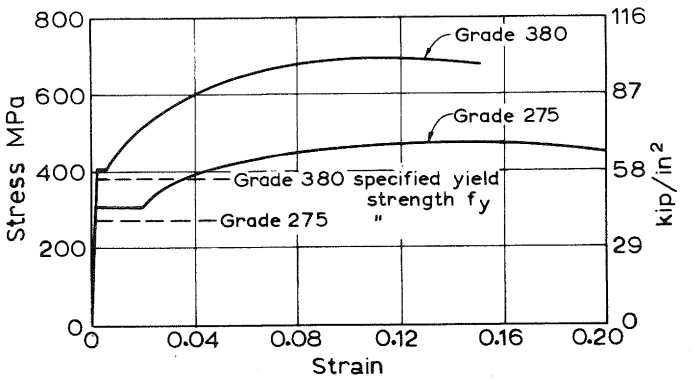


Figure A.1 Stress-Strain Curve for Grade 275 and 380 Steel Reinforcement With Monotonic Loading [12]

APPENDIX A

SUMMARY OF SEISMIC DESIGN PROVISIONS OF THE NEW SANZ CONCRETE DESIGN CODE (NZS 3101)

A.1 General Approach

The general design approach is to design the chosen plastic hinge locations in the structure to have sufficient strength and ductility to satisfy the combinations of factored seismic and gravity loads recommended by the appropriate loadings code, and then to use the capacity design procedure to find the strength required elsewhere in the structure in order to ensure that failure cannot occur away from the plastic hinge locations during severe earthquake loading.

In the capacity design procedure it is assumed that the plastic hinge regions develop their maximum likely flexural strength, referred to as the "flexural overstrength". The flexural overstrength should be calculated assuming that the actual strength of the longitudinal reinforcing steel is $1.25f_y$ when f_y = 275 MPa and 1.40 f_y when f_y = 380 MPa, where f_y = specified yield strength of the reinforcing steel. The overstrength moment at plastic hinges takes into account the actual steel strength being greater than specified and the effect of strain hardening at high strains (see Figure A.1.)

A.2 Ductile Moment Resisting Reinforced Concrete Frames

(a) Design Actions

Design actions in beams - The design moments in beams should be calculated by elastic frame analysis for the code load combinations (Equations 2 to 4). Moment redistribution from the derived moment envelope may be used to gain a more advantageous design moment envelope and thus allow a more efficient design of the beams. The amount of moment redistribution permitted for any span of a beam forming part of a continuous frame should not exceed 30% of the maximum elastic moment derived for that span for any combination of design earthquake and gravity loading. The design shear force in beams should be determined by a capacity design procedure for when the flexural overstrength is reached at the most probable plastic hinge locations within the span and the factored gravity load is present. For example, for the beam in Figure A.2 the design shear force at B will be

$$V_{uB} = \frac{M'_{oA} + M_{oB}}{\ell_{AB}} + \frac{w\ell_{AB}}{2} \qquad (A.1)$$

where M'_{oA} and M_{oB} are the flexural overstrength capacities of the sections for positive moment at A and negative moment at B, and w is the factored uniform dead and live load considered to be present per unit length.

Design actions in columns - A strong column-weak beam is adopted for frames with more than two storeys, and the column design actions are found using the following capacity design approach.

The design bending moments in the columns are determined by multiplying the column moments found from elastic frame analysis for the code load combinations by factors greater than unity to take into account:

(i) The flexural overstrength at the beam plastic hinges.

(ii) The higher mode effects of dynamic loading which can cause much higher column moments than calculated from the static loading which is based on mainly a first mode response. For example, Figure A.3 shows the possible bending moment distribution in a column at an instant during an earthquake and it is evident that the point of contraflexure can move well away from midheight. Hence the total beam input moment $M_{b1} + M_{b2}$ may have to be resisted almost entirely by one column section, rather than shared almost equally between the column section above and below the joint.

(iii) The greater column moments in a two-way frame caused by the possible simultaneous yielding of beams in two directions due to seismic loading acting in a general (skew) direction and having components of load along both principal axes of the structure. For example, for the symmetrical building shown in Figure A.4, if a displacement ductility factor of 4 is reached in direction 2 it only requires $\Delta_1 = \Delta_2/4$ to cause yielding in direction 1 as well, and this occurs when θ is only 14^0. Thus yielding in the beams in both directions may occur simultaneously for much of the seismic loading, and for a structure with beams of equal strength in each direction, the resultant beam moment input applied biaxially to the columns is $\sqrt{2}$ times the uniaxial beam moment input. Biaxial bending will generally reduce the flexural strength of the column. Typically the flexural strength of a square column for bending about a diagonal may be 15% less than the flexural strength for uniaxial bending. Therefore concurrent earthquake loading may cause the columns to yield before the beams unless columns are strengthened to take this effect into account.

The detailed capacity design procedure, given in the commentary of the code, is due to Paulay (9). In this procedure the design uniaxial bending moment for the column, acting separately in each of the two principal directions of the frame, is given by

$$M_{col} = \phi_o \omega M_{code} \qquad (A.2)$$

where M_{code} is the column moment at the beam centre line derived from the code static loading and to be reduced as indicated by the moment gradient to give the moment at the beam face, ϕ_o is ratio of the overstrength flexural capacity of the beams as detailed to the dependable beam moment capacity required by the code, and ω allows for higher mode and concurrent loading effects and ranges between 1.2 and 1.8 for one-way frames and between 1.5 and 1.9 for two-way frames, depending on the period of the building. Note that in two-way frames the columns are designed for uniaxial bending only, since ω includes some moment enhancement to make allowance for the effect of biaxial bending. The values of ω are based on dynamic analyses and judgement.

The design axial loads in columns should be derived from the shear forces in the beams found considering the appropriate flexural overstrength of the beams. The maximum design axial load in columns is limited to $0.7f'_cA_g$ or $0.7P_o$, which ever is greater, where f'_c = concrete compressive cylinder strength, A_g = gross area of column section, and P_o = axial load strength of column, since the ductility of very heavily loaded columns may be small even with extensive confining steel.

To protect columns against possible brittle shear failure, the design shear forces for columns should be based on an adverse moment gradient consistent with the development of plastic hinges at flexural overstrength in the adjoining beams or columns where plastic hinges are expected.

Design actions in beam-column joint cores - The design shears which should be used are those associated with the attainment of the flexural overstrength of the beams.

(b) Reinforcement in Beams

Plastic hinge locations - The plastic hinge locations in beams are considered by the code to extend over a length equal to twice the beam overall depth (2h) in the vicinity of the maximum moment sections, as is shown in Figure A.2.

Longitudinal reinforcement - In order to ensure ductile flexural behaviour during reversed loading at plastic hinge regions, the code recommends that the area of compression reinforcement should not be less than one half of the area of tension reinforcement, in order to ensure that adequate compression steel is present to assist the concrete carry compression. Equations are also given for the allowable upper limit of tension steel area, which must not be exceeded if adequately ductile behaviour is to be achieved.

Transverse reinforcement - Stirrup-ties are necessary in plastic hinge regions to confine the concrete, to prevent buckling of the nonprestressed longitudinal steel, and to act as shear reinforcement. The maximum permitted centre to centre spacing of stirrup-ties in such regions is 150mm or d/4 or six longitudinal bar diameters, which ever is least, where d = effective depth of beam. The six bar diameter requirement is to ensure that buckling of compressed reinforcing bars does not occur when cycles of reversed loading cause a reduction in the tangent modulus of the steel due to the Bauschinger effect (see Figure A.5). It is required that the tie yield force should not be less than one-sixteenth of the longitudinal yield force in the bar or bars it is to restrain, except that when the tie spacing is less than 100 mm this requirement can be relaxed.

Since the design shear forces are found from the assumed flexural overstrengths the code allows design of sections for shear to be carried out using a strength reduction factor ϕ for shear of 1.0. However it has been found from tests (6) that reversed flexure in plastic hinge regions causes a degradation of the shear carried by the concrete shear resisting mechanisms of aggregate interlock, dowel action, and across the compression zone. Therefore the code requires that shear reinforcement, normally in the form of vertical stirrups, be provided to carry the total shear force in plastic hinge regions. Also, tests have demonstrated that full depth flexural cracks can exist in the plastic hinge regions, as well as inclined diagonal tension cracks, during much of the reversed loading range. This is because when longitudinal steel yields in tension for loading in one direction, open cracks will be present in the concrete "compression" zone when the load is applied in the opposite direction. These cracks will remain open until that steel yields

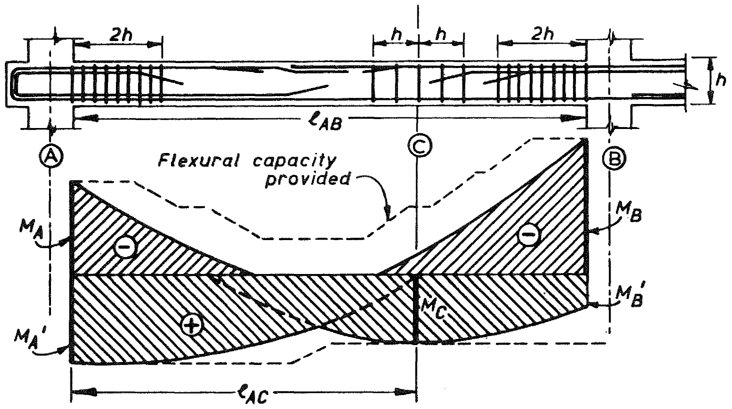


Figure A.2 Localities of Plastic Hinge Regions in a Typical Beam, and Bending Moment Envelope Due to Design Seismic Plus Gravity Loadings (from Commentary of NZS 3101)

$$M_{c1} = M_{b1} + M_{b2} - M_{c2}$$

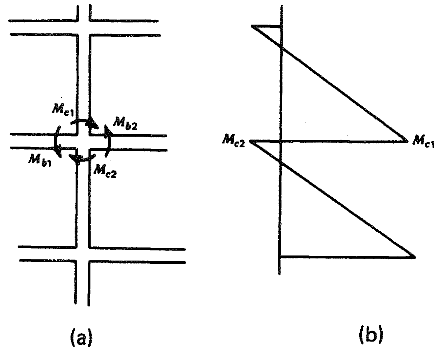


Figure A.3 Possible Column Bending Moments During Seismic Response,
(a) Part of Frame, and (b) Column Moments (6)

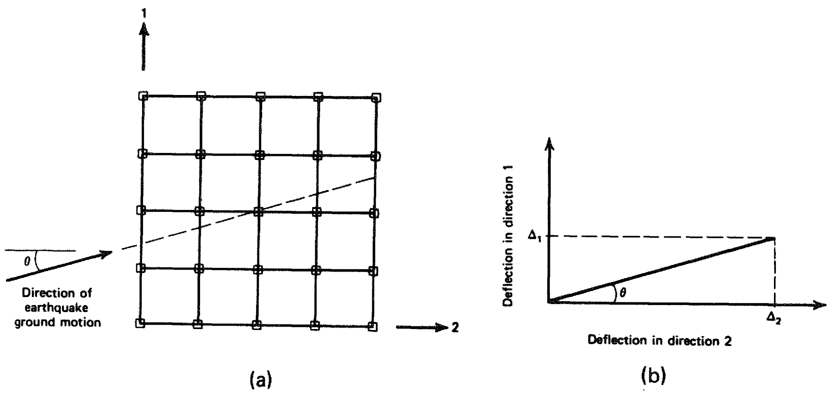


Figure A.4 General Direction of Earthquake Loading on Building (6)
(a) Plan of Building, and (b) Horizontal Deflection of a Floor

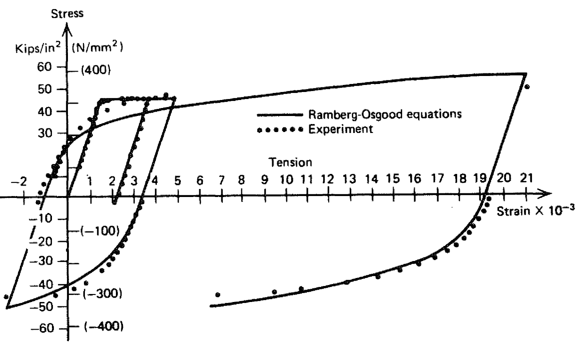


Figure A.5 Stress-Strain Curve for Grade 275 Steel Reinforcement with Reversed Loading (12)

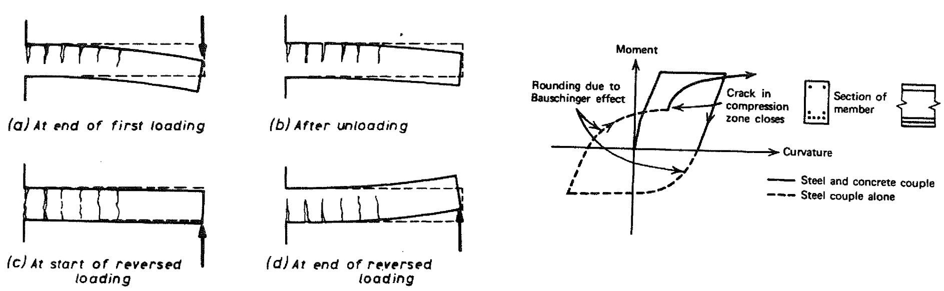


Figure A.6 Effect of Reversed Loading on a Reinforced Concrete Cantilever Beam (6) in compression and allows the cracks to close and the concrete to carry some compression (see Figure A.6). Thus for parts of the loading cycles the bending moment will be carried by a steel couple alone. If the shear stress at the section is high a sliding shear deformation can occur along a full depth vertical crack and the load-deflection hysteresis loop for the structure will become "pinched", resulting in reduced energy dissipation. To avoid a sliding shear failure and loss of energy dissipation the code requires the presence of inclined shear reinforcement (for example, bent up bars) in plastic hinge regions when the shear stresses to be carried are high. Outside plastic hinge regions shear may be considered to be carried by both transverse reinforcement and the concrete shear resisting mechanisms.

(c) Reinforcement in Columns

Plastic hinge locations - The potential plastic hinge locations in columns should be considered to be the end regions of length not less than the larger member section dimension or where the moment exceeds 0.8 of the end moment. Where the column load is high this potential plastic hinge length is increased by 50% because the greater content of confining steel at high axial loads enhances the flexural strength of the plastic hinge regions and could result in flexural failure occurring in the less heavily confined adjacent regions.

Longitudinal reinforcement - In potential plastic hinge regions the longitudinal bars should be spaced not further apart between centres than the larger of one-third of the section dimension in that direction or 200 mm. Limits are also set on the maximum and minimum steel ratios permitted.

Transverse reinforcement - The possibility of yielding occurring at the column ends due to the effects discussed previously makes it important to ensure that columns are capable of behaving in a ductile manner. Hence adequate transverse steel in the form of hoops or spirals should be present at the potential plastic hinge regions at the column ends, to confine the concrete and to prevent buckling of the longitudinal steel. Examples of an inadequately confined tied column and a well confined spiral column after earthquake damage are shown in Figure A.7. Although spiral steel has been shown to provide better confinement than an equal volume of perimeter hoops of rectangular shape, arrangements of rectangular hoops with several legs across the section, such as shown in Figure A.8, will result in very good confinement.

Overseas code requirements for confining steel in columns are fairly arbitrary. The concrete design code gives equations for confining steel content which are an attempt to give a more rational method based on the curvature ductility demands and on moment-curvature analyses of sections (9). In particular, as well as the usual variables, the required amount of confining steel is also stated as a function of the axial load level, since moment-curvature analyses have shown (6) that the available curvature ductility of the column decreases with increase in axial load level. The code equations will result in less transverse steel for columns with moderate to low axial load levels than recommended in ACI 318-77 or the SEAOC codes.

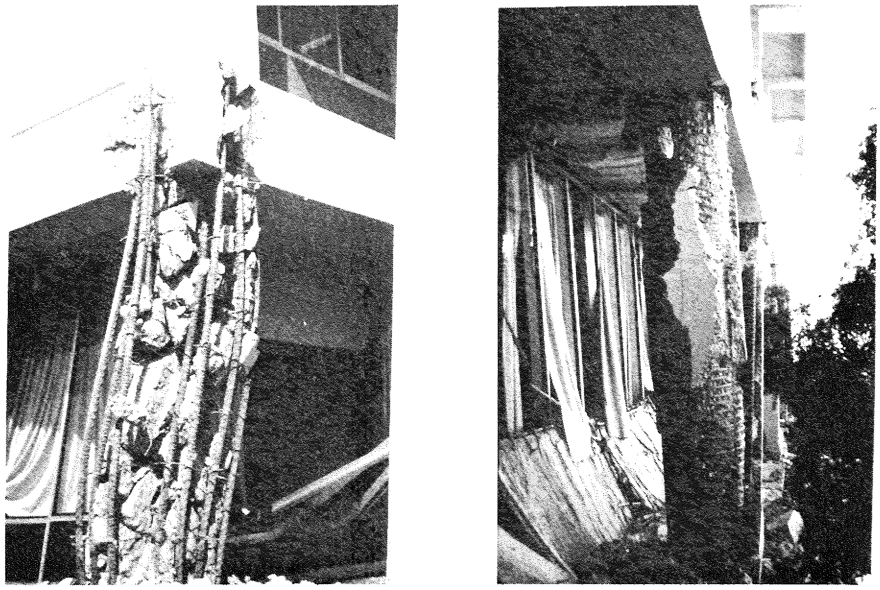
In the potential plastic hinge region the maximum centre to centre spacing of the transverse reinforcement should not exceed the smaller of one-fifth of the column diameter or one-fifth of the smaller cross section dimension, or six longitudinal bar diameters, or 200 mm. Also, where rectangular hoop reinforcement, including overlapping hoops and supplementary cross ties, is used the centre to centre spacing across the section between cross linked bars should not exceed the larger of one-third of the section dimension in the direction in which the spacing is measured, or 200 mm.

For columns in which a capacity design procedure is used to provide a high degree of protection against a column sidesway mechanism (with plastic hinges only in the columns of one storey) the area of special transverse steel may be reduced to one-half of that required by the code equations for confining steel.

Some shear can be carried by the concrete shear resisting mechanisms at high axial loads, since plastic hinging is less likely than in beams and axial compression aids the shear transfer.

(d) Reinforcement in Beam-Column Joints

The strength of a beam-column joint core should be greater than the strength of the members it joins, since the joint core strength may degrade rapidly with cyclic loading, the joint core is difficult to repair, and failure of the joint core could lead to collapse of the column. When reaching their strength under seismic loading, beam-column joint cores can become critical regions of the structure, as is illustrated in Figures A.9 and A.10b. It is to be noted that the book by Blume, Newmark and Corning (4) made no reference to joint core design. The performance of beam-column joints under pseudo-static seismic loading has been studied extensively in New Zealand in recent years (6,9). As a result of these tests design provisions for joint cores have been recommended which differ from those of ACI 318-77. It would appear to be erroneous to base a design procedure for joint cores on test results obtained from members, as the ACI Code has done. For example, Figure A.10a shows a reinforced concrete interior beam-column joint which had been designed using the method of ACI 318-77 (without intermediate column bars and with beam bar diameter/column depth ratio = 1/16). The joint failed in joint core shear and slip of beam bars after the first inelastic loading cycle (6), demonstrating the inadequacy of the ACI approach. Figure A.10b shows the joint after four inelastic loading cycles to a displacement ductility factor of four.



(a) (b)

Figure A.7 Lower Storey Columns of Olive View Hospital After the 1971 San Fernando Earthquake. (a) Inadequately Tied Column, and (b) A Well Confined Column

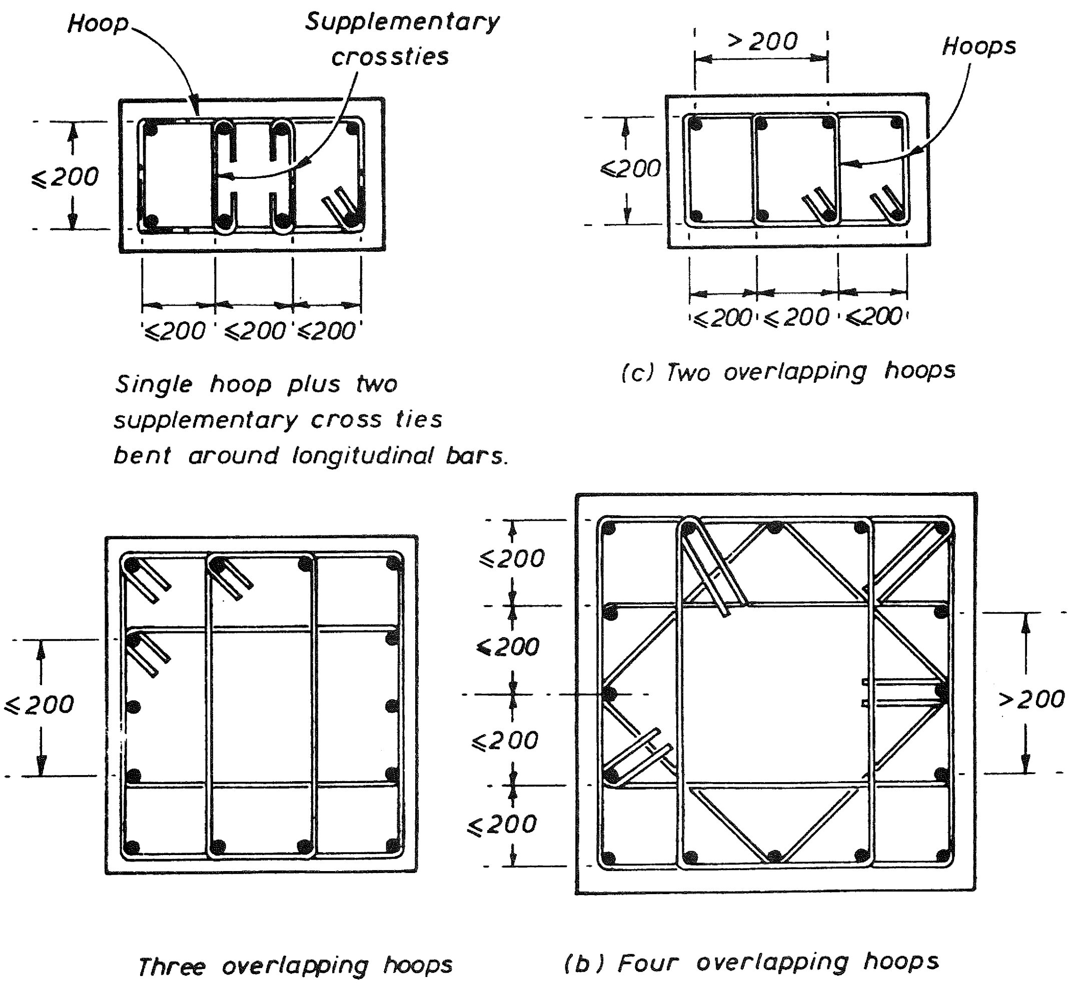


Figure A.8 Some Recommended Details of Transverse Steel in Potential Plastic Hinge Regions of Columns (from Commentary of NZS 3101)

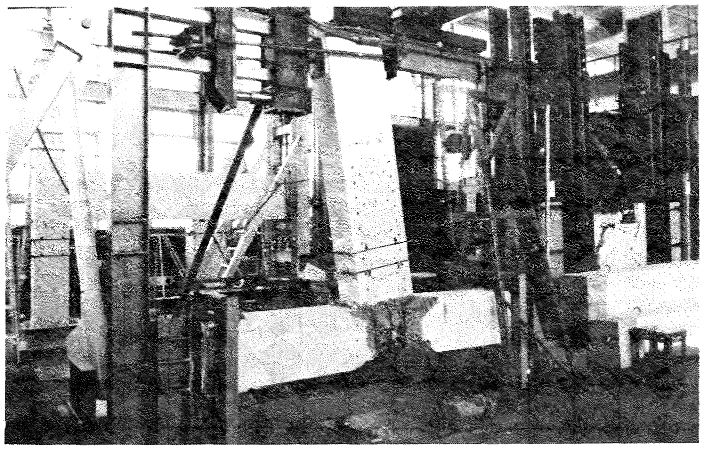
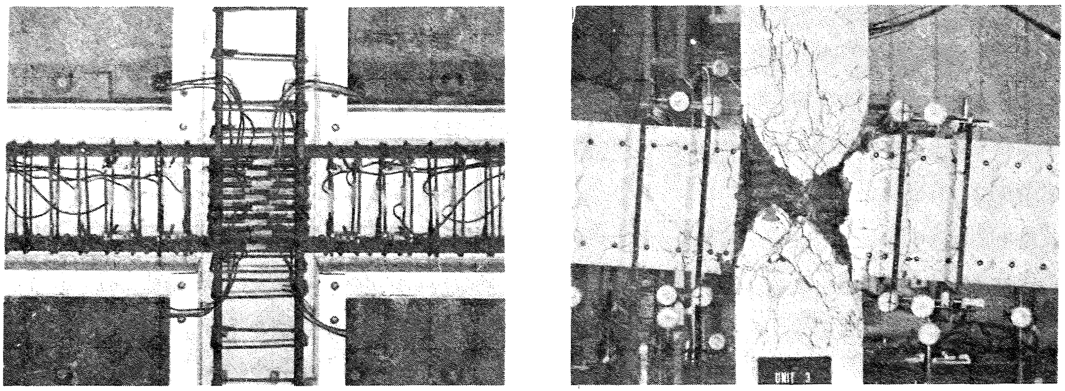
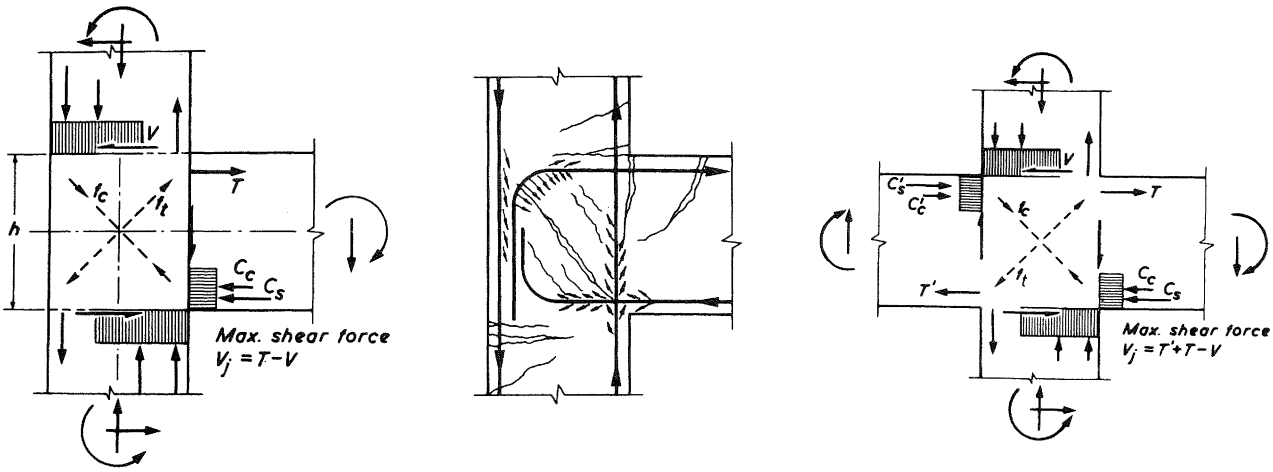


Figure A.9 Reinforced Concrete Exterior Beam-Column Joint Core at Failure



(a) (b)

Figure A.10 Reinforced Concrete Interior Beam-Column Joint:
(a) Reinforcement Cage Before Placing Concrete, and
(b) Shear Failure in Joint Core After Testing (6)



(a) (b) (c)

Figure A.11 Reinforced Concrete Beam-Column Joints of Frame with Seismic Loading: (a) Actions and Stress Resultants of Exterior Joint, (b) Crack Pattern and Bond Forces at Exterior Joint, and (c) Actions and Stress Resultants at Interior Joint.

Anchorage of bars - Figure A.11a and b show an exterior beam-column joint of a reinforced concrete frame and the associated forces and cracking due to a seismic load application to the frame. It is clear that the bond conditions for the longitudinal beam and column bars are unfavourable because: (1) large steel forces need to be transferred to the concrete over relatively short lengths of bar, (2) flexural and diagonal tension cracks are present which will alternate in direction during cyclic loading, and (3) bond deterioration will occur during cyclic loading (6). For example, for the outer column bars the high bond stresses and the anchorage forces from the beam bars can result in vertical splitting of the concrete along the outer column bars (see Figure A.12). Degradation of bond strength will also cause yielding of longitudinal bars to penetrate into the joint core, thus reducing the effective anchorage length and possibly resulting in loss of anchorage. Therefore, in the concrete design code it is recommended that at exterior beam-column joints in which plastic hinging occurs in the beam at the column face, the anchorage of beam steel should be considered to commence within the joint core at one-half the column depth or ten longitudinal bar diameters, which ever is less, from the face of the column where the steel enters (see Figure A.13). An anchorage block, in the form of a beam stub at the far face of the column in which the longitudinal beam bars can be anchored, has been shown to result in considerable improvement in joint performance (6) and are being used by some designers in New Zealand (see Figures A.13b and A.14).

Figures A.11c and A.15 show an interior beam-column joint of a reinforced concrete frame. If plastic hinges form in the beams at the column faces, a beam bar will be yielding in tension on one side of the joint core and in compression on the other side of the core, and hence twice the yield force of the bar will need to be transferred by bond to the joint core, which may require extremely high bond stresses. For this case the new concrete design code recommends that to avoid a bond failure the ratio of longitudinal beam bar diameter to column depth should not be greater than 1/25 for Grade 275 deformed steel bar or 1/35 for Grade 380 deformed steel bar.

Shear resistance - The mechanisms of shear resistance within a beam-column joint core involve:

(1) a diagonal compression strut carrying the concrete compressive forces across the joint, and

(2) a truss mechanism of joint core reinforcement carrying the longitudinal bar forces across the joint (6). These two mechanisms are illustrated in Figure A.15b and c. When full depth cracking occurs in the beams at the column faces the diagonal compression strut mechanism becomes much less effective, unless the axial compression on the column is significant. Diagonal tension cracking in alternating directions can also cause a degradation of the strength of the diagonal compression strut mechanism. Hence cyclic loading causes a transfer of shear force to the truss mechanism. It is to be noted that the truss mechanism requires both horizontal and vertical shear reinforcement, which can be provided by horizontal column hoops and vertical column bars between the corner vertical bars of the section. The concrete design code requires at least one intermediate column bar to be present between corner bars on each side of the column. That is, four bar columns should not be used. The design shear forces acting on the joint core are found from the beam and column actions at faces of the joint core. The shear strength is given by the sum of the shear carried by the concrete V_c (that is, the mechanism of Figure A.15b) and the shear carried by the shear reinforcement V_s (that is the mechanism of Figure A.15c). When plastic hinges form in the beams at the column faces, the code assumes $V_c = 0$ when the column load is small, but V_c is greater than zero at moderate axial loads and increases as the axial load level increases. The code gives equations for the design of both horizontal and vertical shear reinforcement. The horizontal shear reinforcement would normally take the form of column hoops continued through the joint core, and the vertical shear reinforcement would normally be in the form of intermediate longitudinal column bars.

Plastic hinges located away from faces of joint core - The degradation of joint core shear strength, and the bond problems associated with longitudinal beam and column steel passing through the joint core, can be greatly reduced if yielding of longitudinal steel is forced to occur away from the faces of the joint core. Thus an attractive design concept involves deliberately designing plastic hinges to form in the beams away from the columns (9). The code allows a reduction in the content of joint core shear reinforcement and the use of larger diameter longitudinal bars passing through the joint core for this design situation. Plastic hinging can be forced away from column faces by suitable reinforcing details or by haunching the beams, as shown in Figure A.16.

A.3 Ductile Reinforced Concrete Shear Wall Structures

(a) General

Research conducted in New Zealand in recent years on the behaviour of shear walls under pseudo-static seismic loading has shown that properly detailed shear walls will provide adequate strength and ductility in buildings (6,10,15). Reinforced concrete shear walls provide an attractive means of seismic resistance, helping to reduce problems such as column yielding, beam-column joint detailing, and the limitation of drift. Their stiffness also enables much non-structural damage during a severe earthquake to be minimized.

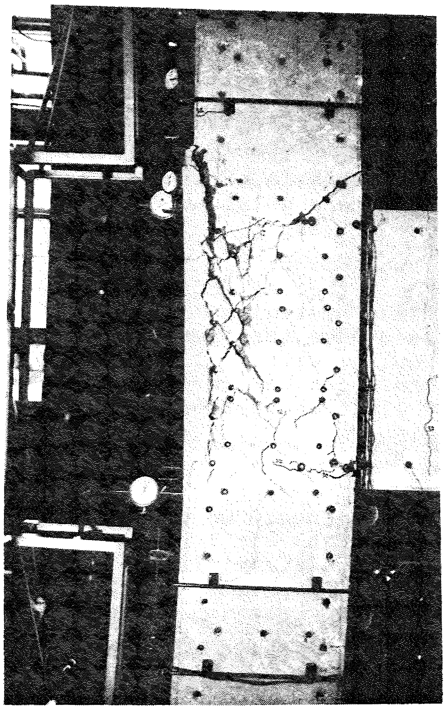


Figure A.12 Reinforced Concrete Exterior Beam-Column Joint Showing Splitting Cracks at Outer Longitudinal Column Bars

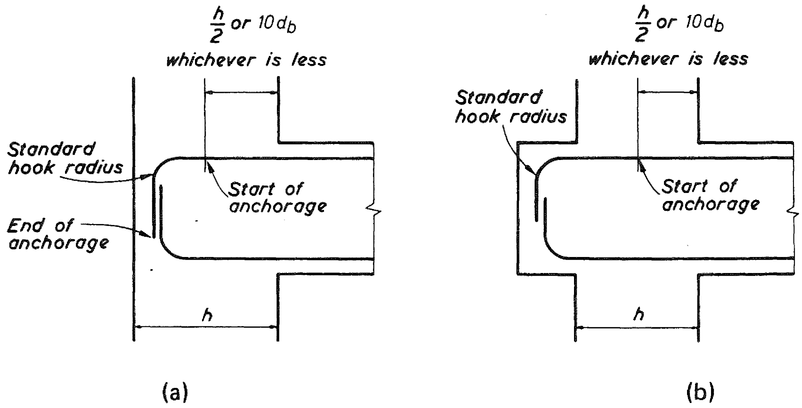


Figure A.13 Anchorage of Beam Bars at Exterior Column When Plastic Hinge Can Form in Beam at Column Face: (a) Anchorage in Column, (b) Anchorage in Beam Stub

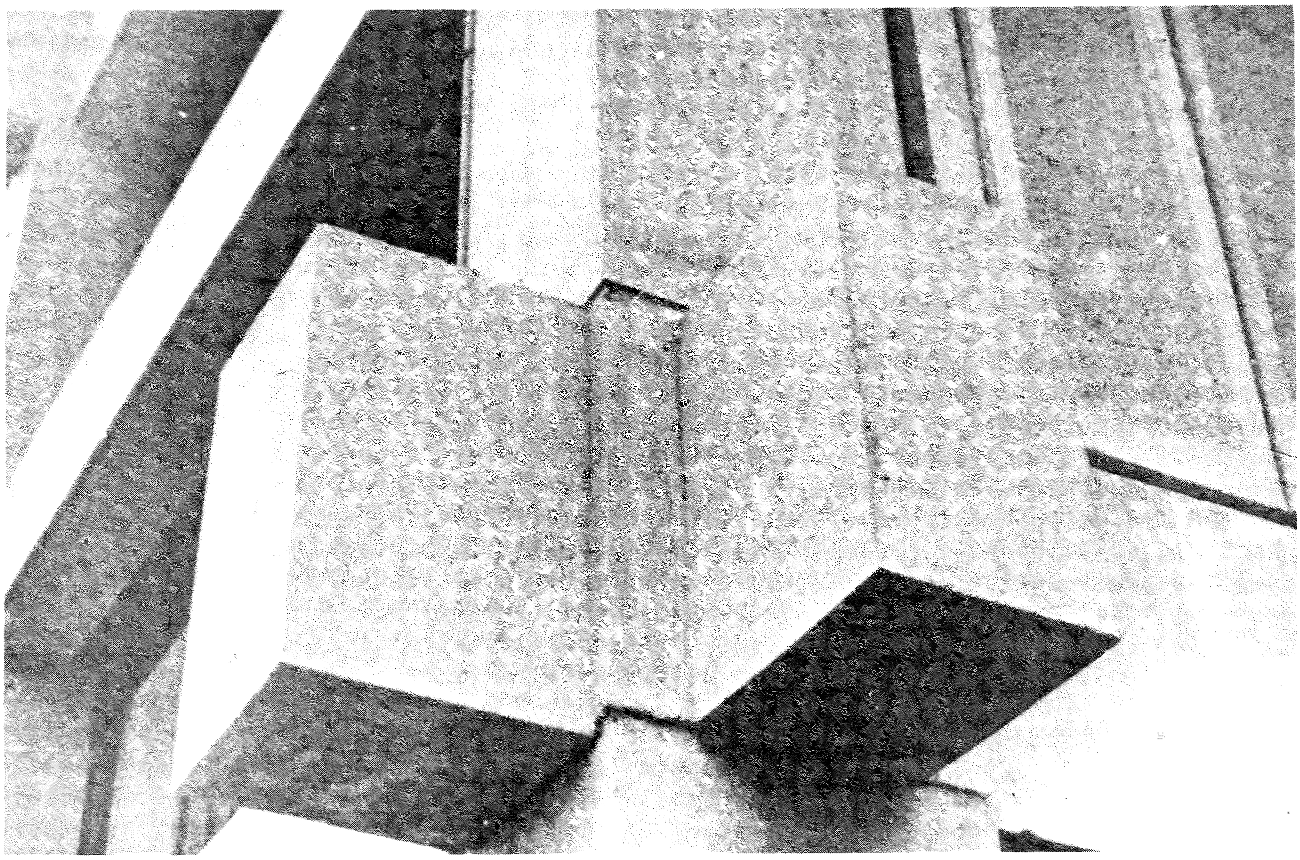


Figure A.14 Beam Stub to Improve Behaviour at Exterior Beam-Column Joint

(b) Design Actions

The aim of the concrete design code is to ensure that energy dissipation in all types of shear walls should be predominantly by ductile flexural yielding. A capacity design procedure should be adopted to ensure that the flexural and shear strengths of the walls in regions away from the plastic hinge regions will be in excess of the bending moments and shear forces induced when flexural overstrength is reached at the plastic hinges.

For a cantilever shear wall the plastic hinge region at the base of the wall is designed for the bending moment induced by the code load combinations. According to the commentary of the code the vertical reinforcement should be curtailed so that the moment of resistance reduces linearly from the end of the plastic hinge region to the value of the design moment at the top of the wall. This linear distribution of moment is shown in Figure A.17 and is to eliminate the possibility of yielding in the upper regions of the wall due to higher mode effects. The commentary of the code also recommends that the design shear force at all levels, indluding the plastic hinge regions, be obtained from

$$V_{wall} = \phi_o \omega_v V_{code} \qquad (A.3)$$

where v_{code} is the shear force derived from the code static loading, ϕ_o is the ratio of overstrength flexural capacity of the wall as detailed to the dependable wall capacity required by the code, and ω_v is the dynamic magnification factor which varies between 1.0 and 1.8 depending on the number of storeys and increases with the number of storeys.

For shear walls with coupling beams a significant part of the energy dissipation should occur due to flexural yielding of the coupling beams. Ductile coupled shear walls should have sufficiently stiff coupling beams to cause the axial forces in the walls, induced by the shears in the coupling beams and derived by elastic analysis for code loading, to resist at least two-thirds of the total overturning moment at the base of the wall. In such a system, during a severe earthquake, most of the coupling beams will yield before the wall bases. To maintain this energy dissipating system it is necessary that the walls sustain the axial loads introduced by the shears in the coupling beams at their flexural overstrengths together with the moments at the wall bases.

(c) Cantilever Shear Walls Without Openings

Cantilever shear walls are in effect cantilever columns with small axial load and narrow width. The longitudinal reinforcement content is small and therefore they can be expected to behave in a ductile manner, providing lateral instability of the compression flange does not occur, lateral buckling of the longitudinal compression steel is prevented, concrete is confined, and shear failure is prevented. The potential plastic hinge region is assumed to extend above the critical section at the base by the horizontal depth of the wall ℓ_w or one-sixth of the height of the wall, which ever is larger.

Shear wall sections are often thin and therefore under reversed cyclic yielding there is a danger of section instability by lateral buckling. The code recommends that in potential plastic hinge regions the thickness of rectangular multistorey walls should be at least $\ell_n/10$ where ℓ_n is the unsupported height of the wall between floors or other effective lateral supports. If the neutral axis is close to the compressed edge this requirement need not be complied with. If necessary the wall thickness can be increased to meet the $\ell_n/10$ limit where required, or a small flange can be provided (see Figure A.18).

Transverse ties should be placed at a spacing not exceeding six longitudinal bar diameters at the extremities of the wall section in the potential plastic hinge zone to prevent buckling of the longitudinal reinforcement. Also, where the concrete strain in the compression zone at the flexural strength is high, hoops should be placed to confine the compressed concrete. The transverse reinforcement provided also has to satisfy the shear strength requirements. Figure A.18 shows some details of transverse reinforcement.

(d) Coupled Shear Walls

Many shear walls contain vertical rows of openings, and the walls on each side of the openings are connected by short deep beams. The openings are typically for doors or windows. Extensive studies of the behaviour of coupling beams have been made in New Zealand (6,10,15). When the wall is subjected to seismic loading the coupling beams are subjected to flexure and shear and because of their small span/depth ratio shear deformations may become very significant. For coupling beams with clear span/depth ratios less than about 2, the diagonal tension cracking causes a redistribution of the tensile forces and the whole length of the longitudinal bars, top and bottom, in these beams may be in tension. In such a case no increase in ductility is available through compression steel, since the concrete carries all the compression. Diagonal tension cracking in alternating directions significantly reduces the capacity of the concrete to carry shear. Therefore shear reinforcement should be provided to carry all the shear force. The stiffness of the coupling beams can degrade significantly with cyclic loading and the failure for members adequately reinforced for shear may be due either to crushing of concrete, or to a sliding shear failure along a vertical crack. Vertical stirrups cannot effectively prevent this type of shear failure and hence if conventional reinforcing details are used the nominal shear stress should be limited to ensure that sliding shear failure does not occur.

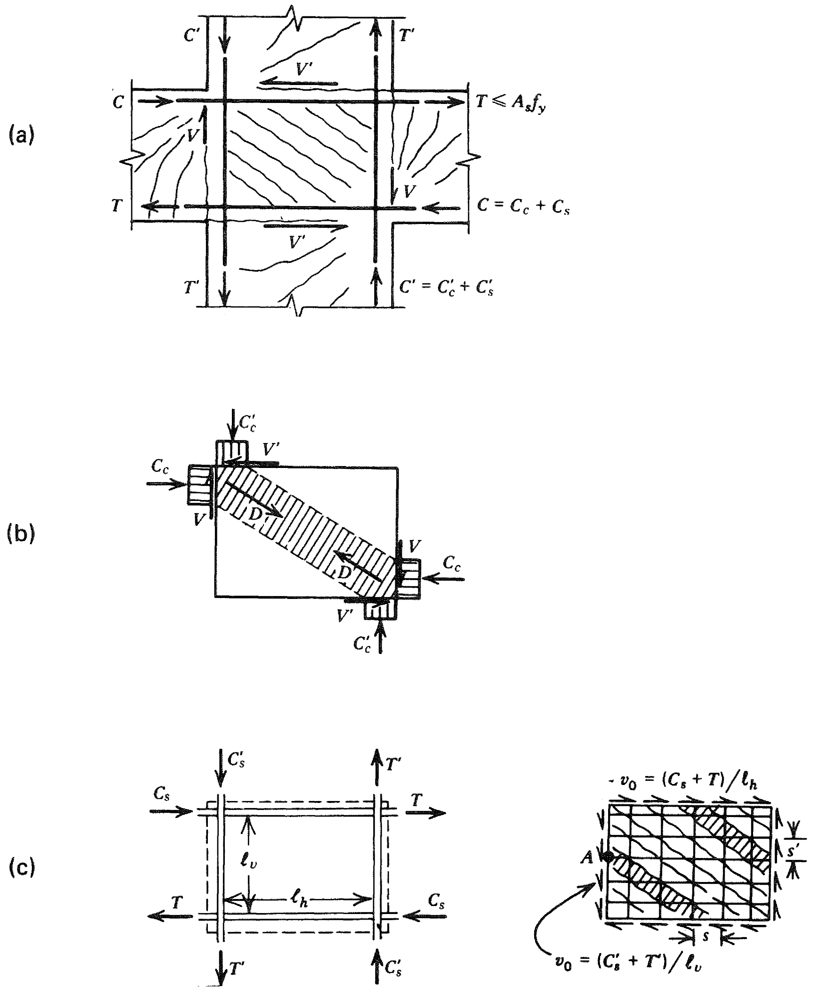


Figure A.15 Idealized Behaviour of Reinforced Concrete Interior Beam-Column Joint: (a) Internal Actions and Crack Pattern, (b) Shear Transfer of Concrete Compression Forces by Diagonal Compression Strut Mechanism, and (c) Shear Transfer of Reinforcement Forces by Truss Mechanism (6)

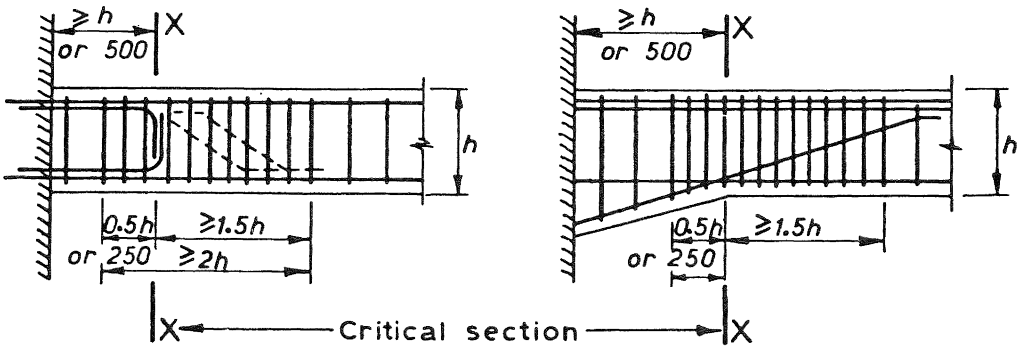


Figure A.16 Suggested Beam Reinforcement to Locate Plastic Hinge Away from Column Face (from Commentary of NZS 3101)

The ductility and useful strength of coupling beams can be considerably improved if, instead of the conventional arrangement of longitudinal flexural steel and vertical stirrups, the principal reinforcement is placed diagonally in the beam. For such a beam the applied moments and shear are resisted by the internal diagonal compression and diagonal tension forces. When a full depth open crack exists after cyclic loading the diagonal steel carries both the moment and flexure without assistance necessary from the concrete other than stabilizing the compression bars against buckling. Diagonally reinforced coupling beams have been shown to have excellent characteristics, and the hysteresis loops have almost the stability of a steel member (6,15). Strength degradation only occurs if buckling of compression bars commences. The code recommendations are based on the above considerations. Diagonally reinforced coupling beams are now becoming commonly used by designers in New Zealand (see Figures A.19 and A.20).

A.4 Ductile Moment Resisting Prestressed Concrete Frames

(a) General

The use of prestressed concrete has been widely accepted for many years for structures carrying mainly gravity loading. The application of prestressed concrete to such structures, for example precast floor systems, has been significant, due to economics, pleasing architectural forms, and the suitability of prestressed concrete for prefabricated construction. However the use of prestressed concrete in primary seismic resistant elements has not met with such ready acceptance, mainly because of the paucity of information on the behaviour of prestressed concrete structures during earthquakes. Recent research, however, has overcome much of this suspicion of prestressed concrete (16) and the concrete design code contains provisions for the design of prestressed concrete for seismic resistant framed structures.

(b) Non-linear Dynamic Response

Figure A.21 shows idealized moment-curvature (or load-deflection) curves for reinforced concrete (modelled by a Ramberg-Osgood relationship), prestressed concrete, and partially prestressed concrete. The prestressed concrete system shows a large capacity for recovery of deflections, even when loaded well into the inelastic range, and therefore the energy dissipated per cycle (area within the moment-curvature loop) is small. On the other hand, a reinforced concrete system will dissipate significant energy. Comparative studies of prestressed and reinforced concrete single degree of freedom systems, using realistic shapes for moment-curvature idealizations as in Figure A.21, have shown that for systems with the same design strength (from code seismic loading for reinforced concrete), initial stiffness, and viscous damping, responding to the El Centro 1940and other very severe earthquakes, the maximum horizontal displacement of the prestressed concrete system was on average 1.3 times that of the reinforced concrete system (16). In spite of the greater displacement response, the ductility requirements of the plastic hinges in prestressed concrete structures can be met with proper detailing. However the greater displacement response of a prestressed concrete structure means that a greater deformation capacity is necessary, and a greater level of non-structural damage will occur, than in an equivalent reinforced concrete structure. For this reason the loadings code, NZS 4203, recommends a 20% increase in the code seismic loading for prestressed concrete structures over that used for reinforced concrete structures. This is achieved by use of a materials factor M = 1.0 for reinforced concrete and M = 1.2 for prestressed concrete in Equation 6.

(c) Ductility of Members and Behaviour of Joints

Theoretical studies have indicated the distributions of prestressing steel which will result in ductile behaviour of members (6). The concrete design code gives detailing rules. For example, in plastic hinge regions in beams, longitudinal steel should exist in the top and the bottom of the member, closely spaced stirrup-ties should be present, and the ratio of depth of the concrete rectangular compressive stress block to member depth should not exceed 0.2 for reasonable ductility. Some nonprestressed steel should generally be present in plastic hinge regions. A good compromise is to use a partially prestressed concrete design. The prestressing steel (post-tensioned) enables the dead and live load to be balanced to a desired degree thus giving a design with good deflection control and also enables continuous frames to be assembled from precast elements. The nonprestressed steel helps to fatten the hysteresis loops and improves the ductility of plastic hinge regions. Tests have also indicated that the shear strength of beam-column joint cores is much improved by the presence of a tendon in the mid-depth region, since such a tendon is able to act as shear reinforcement across the joint core.

(d) Unbonded Tendons

The use of unbonded tendons in prestressed concrete structures in seismic zones has caused considerable controversy. In New Zealand it has been a requirement that prestressing tendons should be bonded to the concrete. That is, only grouted post-tensioned tendons or pretensioned tendons have been permitted. However the concrete design code will allow the use of unbonded tendons with proper precautions (that is: with proper crack control measures, corrosion protection measures, and anchorages of proven relaibility under cyclic loading) but only in primary seismic resisting members if a substantial quantity of bonded steel is also present. There is economy in using unbonded tendons because of the ease of placing them and the avoidance of ducts and grouting. However it is evident that they are best used in slabs.

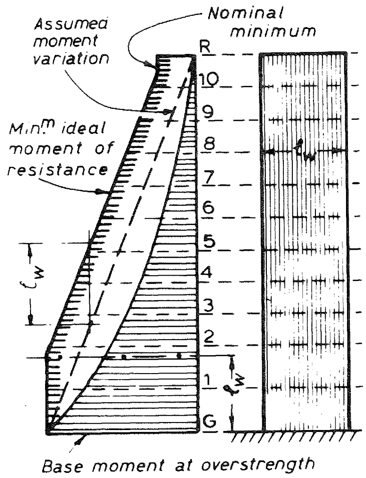


Figure A.17 Recommended Design Moment Envelope for Cantilever Shear Walls (from Commentary of NZS 3101)

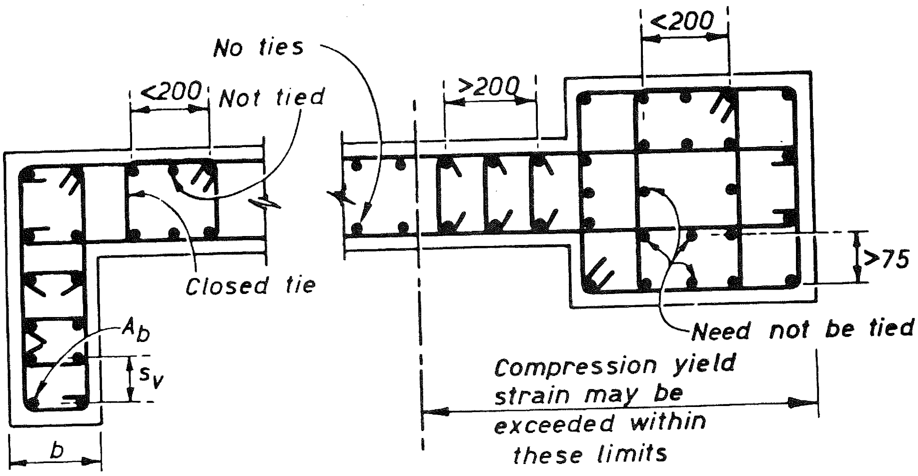


Figure A.18 Details of Transverse Reinforcement in Potential Plastic Hinge Region of Cantilever Shear Wall (from Commentary of NZS 3101)

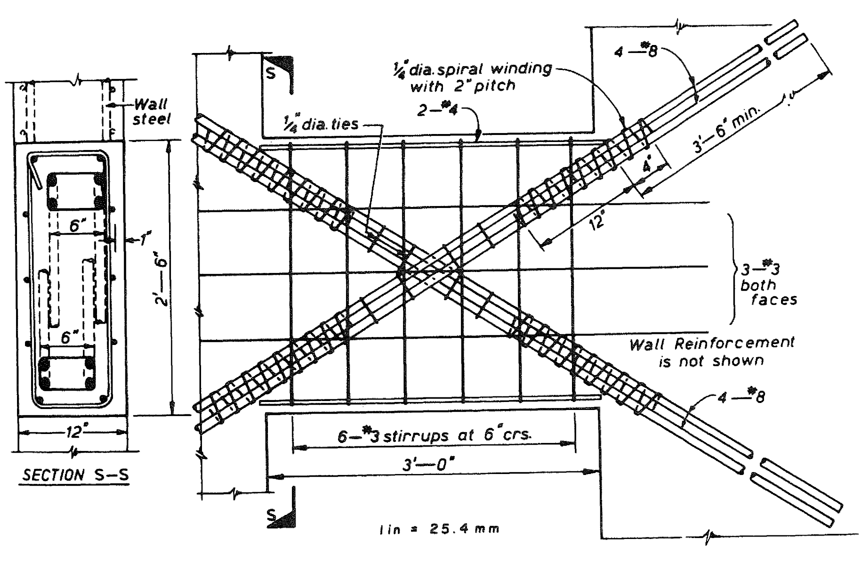


Figure A.19 Typical Details of a Diagonally Reinforced Concrete Coupling Beam (6)

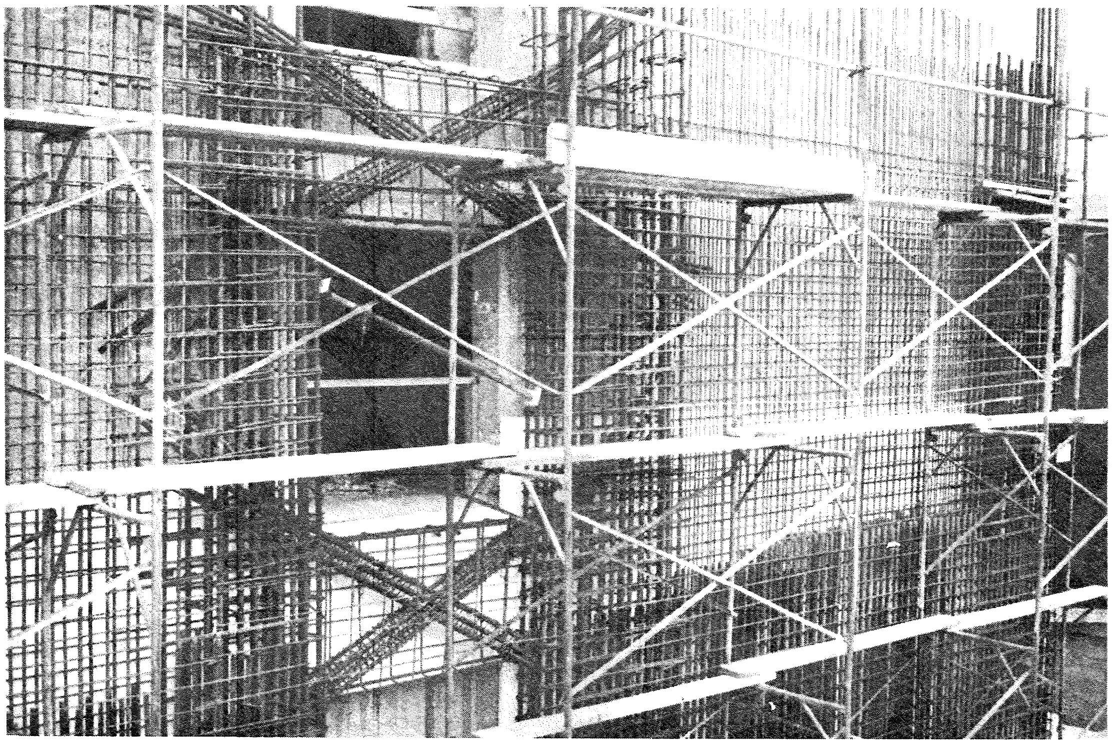


Figure A.20 Diagonal Reinforcement for Coupling Beams of Shear Walls During Construction

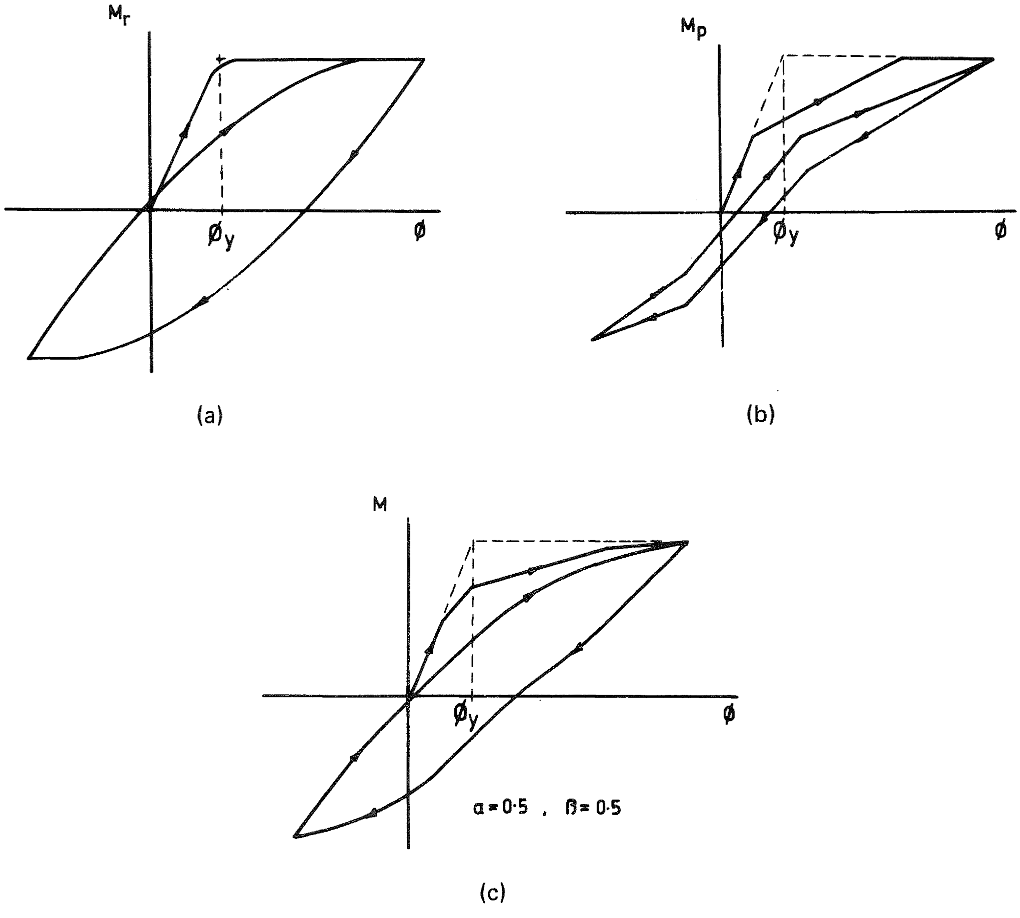


Figure A.21 Idealized Moment-Curvature Loops for (a) Reinforced Concrete, (b) Prestressed Concrete and (c) Partially Prestressed Concrete Systems

A.5 Ductile Bridge Structures

In the past, greater attention has been given to the development of seismic design procedures for buildings than for bridges. However, the spectacular collapse of several bridges during the San Fernando earthquake in 1971 (see, for example, Figure A.22a) highlighted the need for the development of improved seismic design methods for bridges. During recent years in New Zealand considerable efforts have been made to satisfy these needs by the Ministry of Works and Development, the Universities, DSIR and consultants (11,17). A good deal of this research has been sponsored by the National Roads Board.

The design of the deck of a bridge is dominated by dead and live load effects, temperature, etc., rather than by seismic load effects. However, the deck must be adequately supported at the pier caps so as not to become dislodged during the earthquake when significant displacements of the piers occur, and spans when not continuous should be linked together. The piers are the critical design regions for earthquake loading.

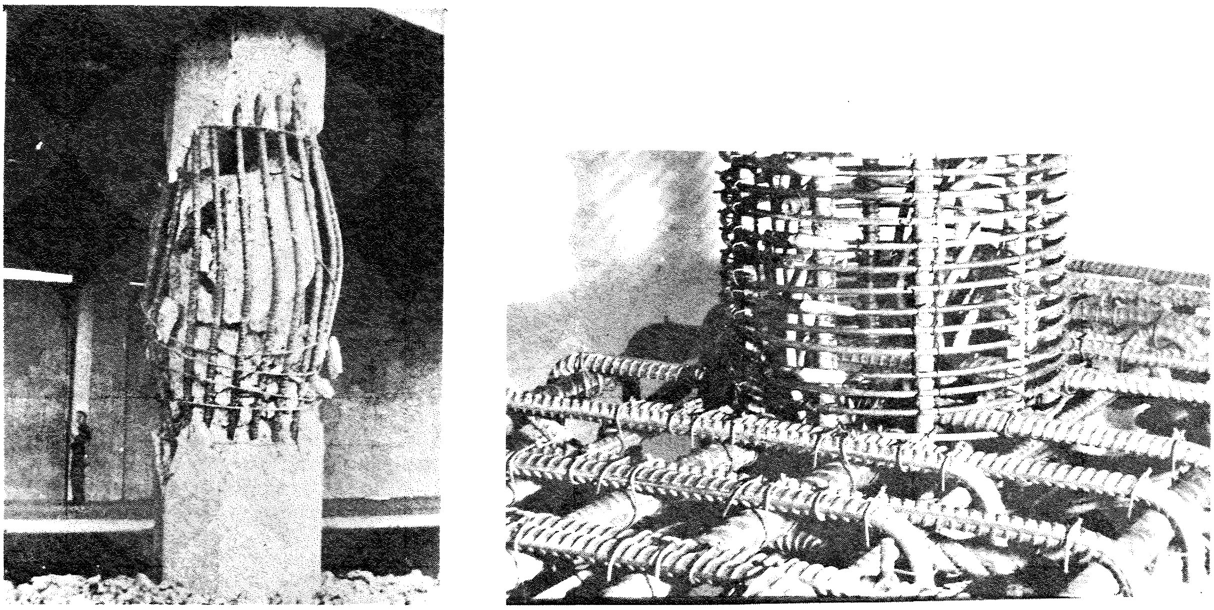
The piers may be designed so as to be capable of dissipating energy by the formation of ductile plastic hinges during a very severe earthquake. The chosen energy dissipating mechanism is usually the pier rather than the foundations because of the greater accessibility for inspection and repair of the piers. A very important aspect in design is to provide sufficient quantity of transverse steel to confine the concrete and to prevent buckling of the longitudinal bars in the plastic hinge regions, and to prevent shear failure. The seismic design provisions of the concrete design code are recommended for detailing for pier ductility. Spiral reinforcement placed in the potential plastic hinge region of a circular bridge pier is illustrated in Figure A.22b. Where there is doubt that "full" ductility can be achieved, greater seismic loads may be used.

Alternatively, the seismic forces, and hence the ductility demand, in bridge piers may be reduced in some cases by the incorporation of energy dissipating devices between the bridge deck and the piers (11,17). This procedure is referred to more generally below.

1.6 Structures Incorporating Mechanical Energy Dissipating Devices

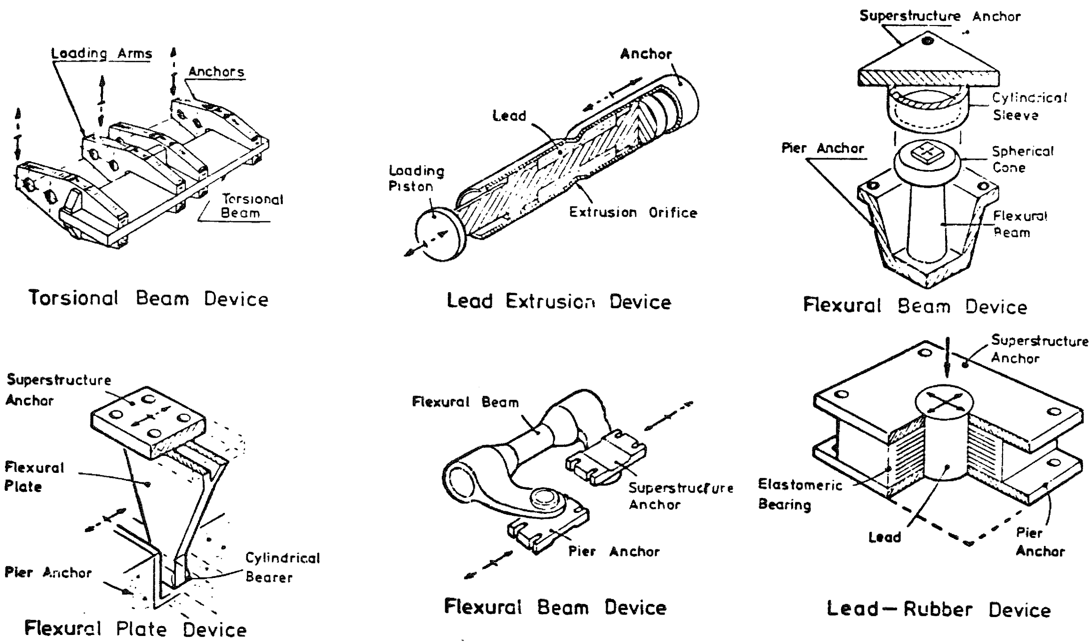
An alternative to the conventional ductile seismic design approach is to use a design approach based on two concepts: (1) The structure is supported on flexible bearings, usually elastomeric rubber bearings, so that the period of vibration of the combined structure and supporting system is sufficiently long so that the structure is isolated from the predominate earthquake ground motion frequencies, and (2) in addition, sufficient extra damping is introduced into the system by mechanical energy dissipating devices to reduce the response and keep the deflections within acceptable limits.

A range of mechanical devices which act as hysteretic dampers have been investigated at the Physics and Engineering Laboratory of the Department of Scientific and Industrial Research, New Zealand. These energy dissipation devices may take the form of steel elements which bend, or twist, lead extrusion or lead shear devices. Figure A.23 shows a range of possible energy dissipating devices which have been developed. Some of these devices are suitable for insertion between the piers and deck structure of bridges, or the foundations and the structure of buildings. A number of New Zealand studies, using nonlinear dynamic analysis, have demonstrated that base isolation is most efficiently employed in short to intermediate period structures. Seismic forces in the structure are decreased and hence ductility requirements are reduced. This method of protection against seismic loading holds much promise, particularly since structural damage is limited. It is a practical design approach now, having been used in several bridges in New Zealand (17) and for a major building in Wellington.



(a) (b)

Figure A.22 Reinforced Concrete Bridge Pier: (a) Damage in 1971 San Fernando Earthquake Due to Inadequate Confinement, (b) Spiral Steel Provided in Potential Plastic Hinge Regions by More Recent Detailing Procedures



(a) Uniaxial (b) Omnidirectional

Figure A.23 Mechanical Energy Dissipating Devices (17)