UNDER	**THE COMMISSIONS OF INQUIRY ACT 1908**
IN THE MATTER OF	**ROYAL COMMISSION OF INQUIRY INTO BUILDING FAILURE CAUSED BY CANTERBURY EARTHQUAKES** **KOMIHANA A TE KARAUNA HEI TIROTIRO I NGA WHARE I HORO I NGA RUWHENUA O WAITAHA**
AND IN THE MATTER OF	**THE CTV BUILDING COLLAPSE**

STATEMENT OF EVIDENCE OF MURRAY LIONEL JACOBS
IN RELATION TO THE CTV BUILDING

DATE OF HEARING: COMMENCING 25 JUNE 2012

I, **MURRAY LIONEL JACOBS**, Civil Engineer of Auckland, say as follows:

Qualifications and experience

1. I am a civil and structural engineer. I am a director of Murray Jacobs Limited, a civil and structural engineering consultancy practice based in Auckland.

2. My qualifications are a Bachelor of Engineering (Honours) and PhD in Engineering. I am a Member of the New Zealand Institute of Engineers, a Chartered Professional Engineer, and an International Professional Engineer.

3. I have over 35 years experience in the design of structures in Auckland. Many of these structures have been in the CBD. Some of the buildings that I have been involved in are:

   (a) Vero Centre, Shortland Street a 40 level office tower;

   (b) PwC Tower, Quay Street a 30 level office tower;

   (c) ASB Tower, Albert Street a 35 level office tower

   (d) Sylvia Park Shopping Centre.

   (e) Quay West, Customs Street apartment building.

   (f) BNZ tower lower Queen Street

4. I have read the Code of Conduct for Expert Witnesses. I agree to comply with the Code and I have prepared this statement in accordance with it.

Instructions

5. I have been asked by Counsel Assisting the Royal Commission to provide evidence to the Commission that addresses the following issue:

   Whether on 30 September 1986, being the date on which a building permit was issued by the Christchurch City Council for what is now referred to as the CTV Building, the Building complied with the Christchurch City Bylaw No 105 (1985) and the relevant Standards, Standard Specifications and Codes of Practice listed in the Second Schedule to that Bylaw.

Materials reviewed

6. In preparing this statement, I have reviewed and had regard to the following documents:

   (a) NZS 4203:1984 Code of Practice for General Structural Design and design loadings for Buildings. (**ENG.STA.0018**)

   (b) NZS 3101 Part 1 :1982 Code of Practice for The Design of Concrete Structures (**ENG.STA.0016**)

   (c) NZS 3101 Part 2:1982. Commentary on The Design of Concrete Structures (**ENG.STA.0017**)

   (d) Structural Drawings -Office Building – 249 Madras Street. Alan Reay Consultants S1-S39 (**the permit plans**) **(BUI.MAD249.0284)**

   (e) Christchurch City Council By Law No 105 (1985) (**ENG.CCC.0044**)

   (f) CTV Building Collapse Investigation for Department of Building and Housing 25 January 2012 by Clark Hyland and Ashley Smith (**BUI.MAD249.0189**)

   (g) Calculations seismic, Alan M. Reay Consulting Engineer (**BUI.MAD249.0272**)

Methodology

7. The requirement and intentions of the three relevant codes of practice applicable at the time of design will be examined and compared with the design of the CTV Building as shown on the permit plans. The questions asked will be:

   (a) Does the design comply with the codes and their intentions?

   (b) If not, what parts of the structure did not comply?

   (c) How significant were any areas of non-compliance to the ability of the Building to withstand an earthquake.

Summary of findings

Codes:

8. The building designed was required to comply with the Christchurch City Council By-Law No.105 **(ENG.CCC.0044**). They required the building to be designed to the current NZ Codes. The three significant Codes, all included in By Law No. 105, are:

NZS 4203:1984	Code of practice for general structural design and design loadings for buildings
NZS 3101:...	The design of concrete structures -
	Part 1:1982 Code of Practice for the design of concrete structures
	Part 2:1982 Commentary on the design of concrete structures

9. NZS 4203 outlines the requirements for general structural design and gives design loadings to be taken for the design. It covers the gravity loads such as dead load and live load, wind loads and seismic loads.

10. NZS 4203 states in the forward to the code on page 8 (**ENG.STA.0018.13**) that:

> It aims at setting minimum requirements for the general run of buildings …

However on page 9 it cautions that:

> Designers should recognise that the precise properties of construction materials and structural elements made from them are not clearly known. Furthermore, the interaction of these elements in a building frame under load is extremely uncertain, so that the total design technique is one of some degree of imprecision.

11. On Page 33 under the section *PART 3 EARTHQUAKE PROVISIONS* the first clause number 3.1, (**ENG.STA.0018.38**) states:

> 3.1 SYMMETRY
>
> 3.1.1 The main elements of a building that resist seismic forces shall, as nearly as is practicable, be located symmetrically about the centre of mass of the building.

12. The CTV Building does not comply with this instruction. The primary resisting elements in this structure are asymmetrical in the East West direction. In the North South direction the eccentricity is less. The main resisting element is the concrete core wall between lines 4 and 5 situated completely outside the main floor plate envelope (**North Shear Core**). There is a smaller much less stiff coupled shear wall on the south side of the building on line 1 (**Coupled Shear Wall**).

13. The diagram shown below, taken from the Hyland/Smith report, shows the large separation of the centre of mass from the centre of stiffness and consequently rotation. The Building will rotate about the centre of stiffness during an earthquake and place a greater demand on some of the columns, especially those further away from the centre of stiffness.

14. Clause 3.4.7 Horizontal torsional moments:

> 3.4.7. (c) For irregular structures more than four stories high horizontal effects shall be taken into account by three dimensional modal analysis method of clause 3.5.2.2.2. **(ENG.STA.0018.53)**

The Commentary cautions in C3.4.7.1:

> It should be noted that even a three dimensional analysis may not always give good predictions of the dynamic behaviour of very irregular buildings, and may indeed seriously underestimate earthquake effects in some cases. (**ENG.STA.0018.54**)

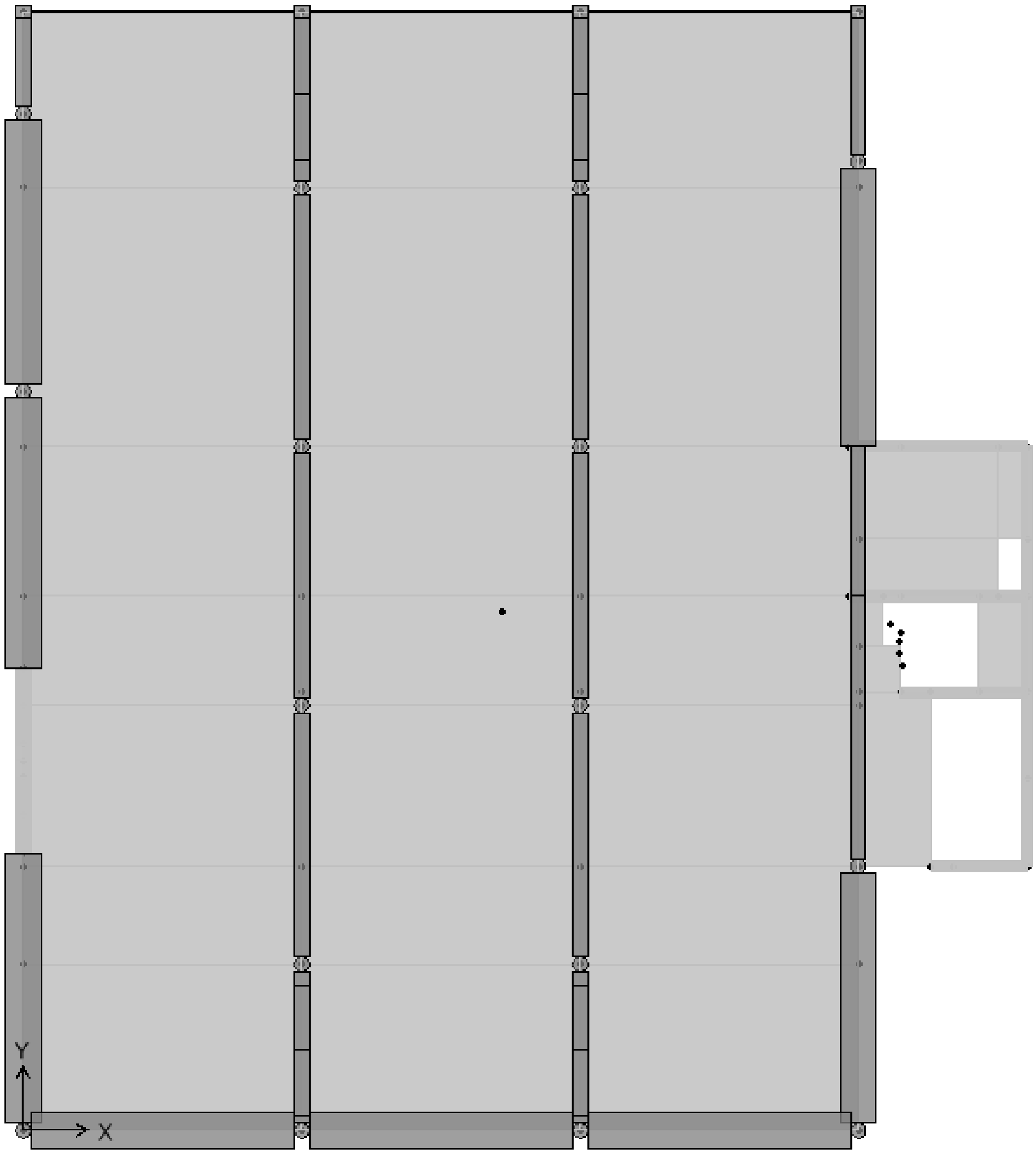

Diagram 1: Plan of building showing the centre of mass as the dot in the centre of the floor plate and the centre of stiffness shown as the collection of dots in the North Core wall (stair well)

Columns:

15. NZS 4203 states on page 33 in clause 3.2 DUCTILITY: (**ENG.STA.0018.38**)

> 3.2.1 The building as a whole and all of its elements that resist seismic forces or movement, or **that in the case of failure are a risk to life**, shall be designed to possess **ductility** (Note clause 3.4.8.1 is deleted in the 1986 version) (emphasis added)

16. The columns in the CTV building were a risk to life if they failed and they should have been designed to exhibit ductility. They were not. Concrete is a brittle material. These columns are small in diameter and are not detailed to provide ductile action. That is, they are prone to fail in a brittle manner when subjected to reverse cyclical motion such as in an earthquake. The usual failure mode is for the concrete outside the reinforced core to fall off the columns, leaving a severely limited cross section of remaining concrete column to carry the load from the floors. Concrete is strong in compression but has limited reliable strength in tension. To make up for this characteristic the concrete columns and beams are reinforced with deformed steel bars. They bond with the concrete and carry any tensile loads developed from bending moments and, importantly, shear loads.

17. In ductile columns these steel bars also serve another role. They confine the concrete inside the ties and contain it from breaking up and falling out of the column completely under the repeated cyclical loadings typical in an earthquake. Experiments have shown that if the ties are close enough and of sufficient strength they, in conjunction with the vertical longitudinal bars, are able to contain the concrete inside the area of the ties and thus provide a functioning, if reduced, area to carry the load of the column. If, however, there are insufficient ties in the column the concrete will fall out from within the inner core of the column during the reversed cyclical loading from an earthquake and the column will fail.

18. The columns in the CTV building could be expected to fail in an earthquake because of insufficient ties. The columns were reinforced with 6 longitudinal bars 20 mm in diameter contained by 6 mm spiral ties at 250 mm centres 150 radius inside. The spirals continued at 250 mm centres through the joint between the floor beams and the column. The 6 mm ties at 250 mm centres are not sufficient to provide ductile action in the columns.

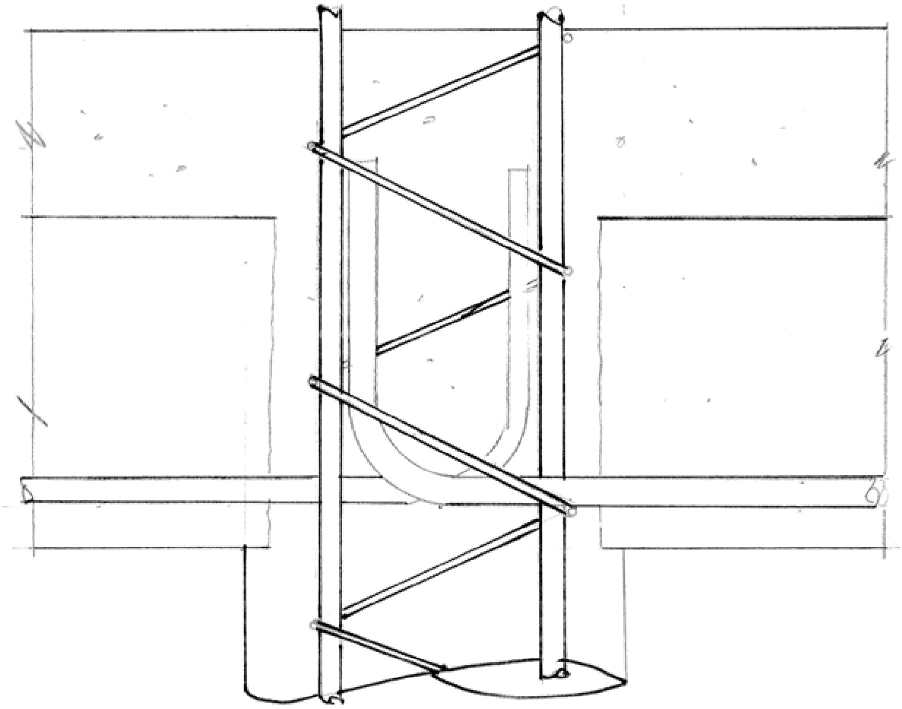

Diagram 2 shows a vertical section through the beam column joint. Note: effectively one tie 6 mm in diameter that is placed in the joint. The bent up bars from the precast beams have limited anchorage in the joint zone.

19. There is considerable congestion in the joint and it is difficult to see how the precast beam bottom reinforcement could be placed with the spiral in position as shown. The structural drawings No S14 show the spiral stopping under each beam and then starting again above the line of the bottom of the beam. There is no note on the drawings to lap the spiral bars as would be required to provide continuity of action.

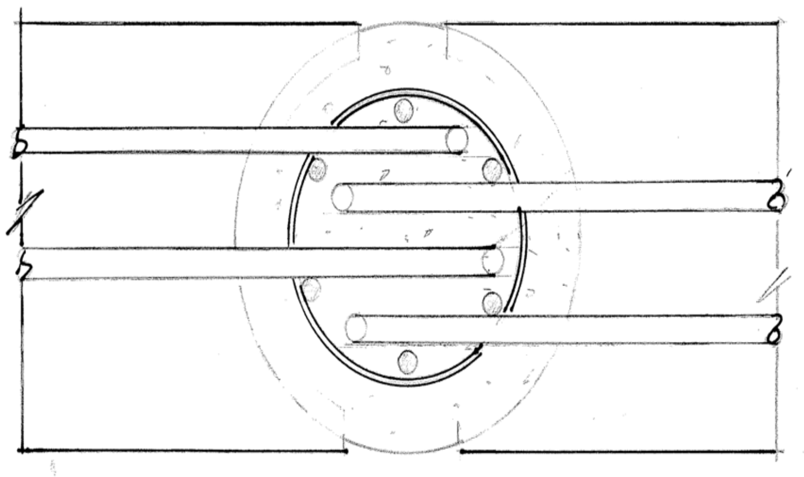

Diagram 3: shows a horizontal section through the beam column joint

20. NZS 3101 Part 1: 1982 Code of Practice for THE DESIGN OF CONCRETE STRUCTURES (**ENG.STA.0016**) and the Commentary NZS 3101 Part 2 set out standards for the design and detailing of concrete structures:

(a) On page 15 under 1 General: (**ENG.STA.0016.17**)

1.1 **Scope**. This New Zealand Standard Code of Practice specifies **minimum** requirements for the design of reinforced and pre stressed concrete structures. (emphasis added).

This means that the designer needs to realise that there well may be extra design actions and forces to provide strength for if they see fit. It is not to be taken as a code that specifies the *maximum* design actions that a building is to be designed for.

(b) **3.5 Principles and requirements additional to 3.3 for the analysis and design of structures subjected to seismic loading (ENG.STA.0016.24**)

3.5.1.4:

The interaction of all structural and non- structural elements which, due to seismic displacements, may affect the response of the structure or the performance of non-structural elements, shall be considered in the design of that structure.

3.5.1.5:

Consequences of failure of elements that are not part of the intended primary system for resisting seismic forces shall also be considered.

21. Clause 3.5.1.5 applies to the CTV building and is a warning that the internal columns shall be considered. The columns in the CTV building were small and heavily loaded. They were not detailed for ductility and as a consequence they would fail if subjected to significant reversed cyclical movements such as occur during an earthquake. The consequences outlined in this clause do not appear to have been heeded. The central columns are also heavily loaded.

22. NZS 4203 gives the various load cases that a structure is to be designed for on page 17 Clause 1.3.2.3 (**ENG.STA.0018.22**). The design load U for the strength method is:

U=1.4D + 1.7 LR……………………….(1)

Where

D = dead loads i.e. the self weight of the building

LR = reduced live load.

The live load for offices is 2.5 kPa from Table 2 page 25. This can be reduced where the tributary area exceed 20 m2 by

R= 0.3+4.6/ √B…………………… (24B)

23. When I calculated the loads specified by the Code for these columns to be designed for dead load and reduced live load, a value was obtained that was at the limit of their capacity.

24. The columns were fully stressed in axial load according to my calculations, allowing for a small SDL on each floor plus ceiling weights.

25. The graph below illustrates the performance of highly loaded columns when subjected to rotation such as would occur during a seismic event. The curve with P = O.4 fca Ag reaches its load capacity then fails soon after with very little extra curvature. This diagram is taken from the NZSEE publication: NZSEE "Assessment and improvement of the Structural Performance of Buildings in Earthquakes" report. June 2006.

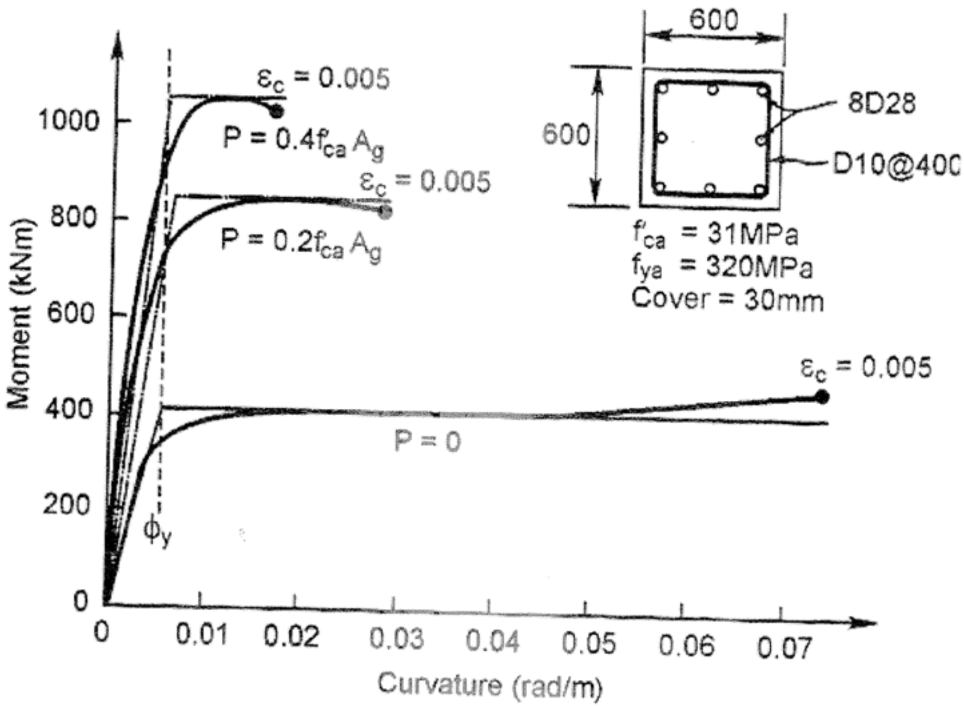

Figure 7.13: Moment curvature response of unconfined columns

26. Page G41A of the CTV Building calculations (**BUI.MAD249.0273.44**) calculates the minimum size of stirrups and spacing required in accordance with NZS 3101.clauses 5.3.29.2 and 6.4.7. and notes spirals 6 mm diameter at 250mm centres. The designer

then computes the hoop reinforcing required by the code assuming the columns develop plastic hinges from taking part in seismic action and concludes R10 at 100 mm centres or R6 at 40 mm centres. The designer then notes on the calculations – "*these do not apply as columns are non- seismic".*

CALCULATIONS

ALAN M. REAY
CONSULTING ENGINEER
CHRISTCHURCH

PAGE G41A.
SECT
FILE
DATE

Column hoops:

min bar dia 5.3.29.2 = 6 mm

$\phi' = 0.7$

non-seismic loading case 6.4.7.1

max spacing = 400 mm

16 x 20 = 320

48 x 6 = 288

use R6 at 250.

Compute seismic loading case in potential p. hinge regions.

$0.45\left(\frac{A_g}{A_c} - 1\right) = 0.45\left(\frac{400^2}{300^2} - 1\right) = 0.35 > .12$

$f'_c = 35$ $\rho_s = 0.35 \cdot \frac{35}{275}\left(0.5 + 1.25 \cdot 0\right) = 0.0223$

$\therefore A_{st} = .0223 \times \frac{\pi \cdot 300^2}{4} = 1576$ mm²/m

R10 at 100 $A_{st} = 1570$

for ⊘ R6 at 80 $A_{st} = 1413$

max spacing s/f = 400/5 = 80

(ii) 16 x 20 = 120

(iii) = 200

– these do not apply as columns are non-seismic.

all loops R6 at 250 spiral.

(BUI.MAD249.0273.44)

Secondary Elements

27. Page 26 of NZS 3101 states :(**ENG.STA.0016.28**)

> 3.5.14 Secondary structural elements:
>
> Secondary elements are those that do not form part of the primary seismic force resisting system, or are assumed not to form such a part and are therefore not necessary for the survival of the building as a whole under seismically induced lateral loading, but which are subjected to loads due to acceleration transmitted to them, or due to deformations of the structure as a whole.

28. The columns of this building were classified as Group 2 secondary elements. They are not detailed for separation and are therefore subjected to both inertia loadings, as for Group 1, and to loadings induced by the deformation of the primary structure.

29. NZS 3101, 3.5.14.3:

> Group 2 elements shall be detailed to allow ductile behaviour and in accordance with the assumptions made in the analysis. **(ENG.STA.0016.28)**

The question is, can the frames in the CTV Building on Lines 2, 3 and 4 (the East-West direction) be assumed to be secondary elements by virtue that there are stiff shear walls running in the same direction that will protect these frames from any excessive deflections? Are deflections under earthquake attack small enough for the frames to retain their integrity to carry the floor loads as elastically deformed columns, or are the deflections such that the columns and beams in these frames will be stressed to past the normal elastic limits? If they go into the post elastic mode they are required to exhibit plastic deformations and therefore ductility will be demanded of them.

30. The same question applies to the frame on Line F, which runs in the North-South direction, except these frames are even more likely to fall into the category mentioned in the Commentary to NZS 3101, (C3.5.14.1) as the shear wall in the North-South direction is more slender:

> Caution must however be exercised in assumptions made as to the significance of participation. Frames in parallel with slender shear walls should be designed and detailed as fully participating primary members… **(ENG.STA.0016A.32)**

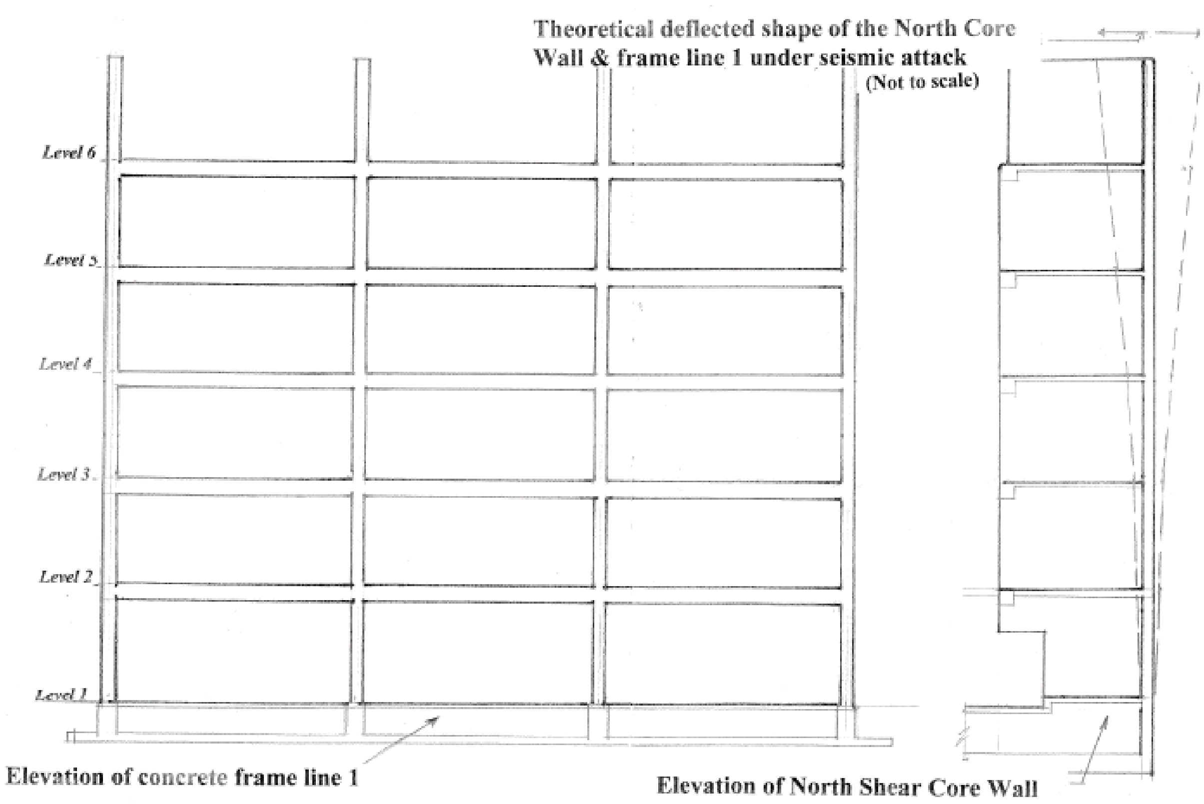

Diagram 4 showing frame line F with North Shear Core. The frame is joined to the shear wall by the slab. Note the notice in the base of the wall. The columns in frame line F have not been detailed for ductile action.

31. The reduced wall section of the North Shear Core shown in diagram 4 between Levels 1 and 2 will mean that the resulting rotation of the wall will be increased. The moment of inertia is under half for the wall at this level compared to the case if the notch was not present. I would consider this to be a slender shear wall in this direction and C3.5.14.1 would apply.

32. The frames in the Building are elements of Group 2 and NZS 3101, clause 3.5.14.3, further mentions what are sometimes confusing tests especially in light of the statement in paragraph 29:

> 3.5.14.3(a) Additional seismic requirements of this Code need not be satisfied when the design loadings are derived from the imposed deformations vΔ, specified in NZS 4203, and the assumptions of elastic behaviour. *(***ENG.STA.0016.28**)

33. My interpretation of this clause is that if the member is checked for its ability to accept deflections derived from vΔ, and the member is still within the elastic range, then there is no need for ductile behaviour to be provided for that element.

34. The results of the modal analysis completed by Compusoft using the factors in the 1984 loading code NZS 4203 have been taken and the deflections of the core wall applied to the frame. The resulting moments introduced into the frame on Line F and Line 2 have

been combined with the axial load in the columns. The elastic behaviour of the columns was exceeded and they would have possibly failed because of their lack of ability to sustain plastic action. My conclusion is that the frame did not comply with this requirement of the code and clause 3.5.14.3(b) would apply, which states:

> 3.5.14.3(b) Additional seismic requirements of this code shall be met when plastic behaviour is assumed at levels of deformation below vΔ.

The columns should have been detailed for ductility.

35. 3.5.14.3(c) sets out the inertia loading requirements from NZS 4203.

    3.5.14.3(d) cautions that the secondary member may be subject to more complex deflections and consequently loads in some localised areas.

    3.5.14.3(e) appears to set a lower bound limit on the elastic behaviour of the secondary item of one quarter of the primary elements. This is to provide a minimum strength to the secondary unit. It points out that a secondary element that responds elastically to the total deflection may be too strong for the structure as a whole.

36. Despite the clauses 3.5.14.3 (a) to (f) being difficult to interpret. the overall lack of detailing for ductility of the critically important columns in the CTV Building does not comply with the instructions of the Code. These columns were critical to the performance of the Building. The calculations from the computer are so dependant on assumptions of stiffness, material properties and the mathematical model formed, that it is not wise to rely on the results as an accurate representation of what will occur in the Building under seismic loading, especially with such an important element as all the columns in a Building. These column hold up the complete floor plate. I do not consider them to be secondary elements.

37. **NZS 3101. 3.5 Principles and requirements additional to 3.3 for the analysis and design of structures subjected to seismic loading**

> 3.5.1.6 Consequences of failure of elements that are not part of the intended primary system for resisting seismic forces shall also be considered. (**ENG.STA.0016.24)**

This clause applies to the internal and external columns in the frames of the CTV Building. They carry the major part of the weight of the Building and are critical to its survival. The internal column and beam frames will take part in the seismic movement of the Building. They are all connected by the floor acting as a diaphragm.

The significance of the floor diaphragms

(a) Diaphragm action:

38. The floors of the Building act as large in-plane ties and struts connecting all the various parts together when an earthquake occurs. They are designed to connect the critical elements such as the shear walls to the rest of the building sited away from the walls. The floor system in the CTV Building was constructed of metal deck formwork with a cast-insitu 200 mm thick slab poured. The metal deck is ribbed to give an average thickness of 175mm.

39. This is a heavier slab than is normally expected on a building with a 7.5 m span. The reinforcing is principally 664 mesh. This area of mesh of 185 mm² per metre length results in an under reinforced slab:

NZS 3101, 10.5.6.2:

> Diaphragms shall be reinforced in both directions with not less than minimum reinforcement required for two-way slabs in accordance with 5.3.32. (**ENG.STA.0016.75**)

Clause 5.3.32 outlines the minimum reinforcement for the various types of reinforcement. For mesh the following is given:

5.3.32 Shrinkage and temperature reinforcement **(ENG.STA.0014.41)** specified that Slabs where bars with fy = 430 MPa or welded wire fabric, deformed or plain, are used ………. 0.0018

40. The 664 welded wire mesh does not meet this requirement. In one direction the metal deck does provide some reinforcement, but in the other it is a series of discrete units jointed together by friction. The slab design was not covered by the Concrete Code existing at the time. The typical procedure was to refer to the manufacturers design charts and use them to select the appropriate span and thickness, including top slab reinforcement at the supports.

41. The HiBond literature current in 1985 contained a load / span chart for single spans, with a maximum of 6.6m span for a 200 mm slab thickness, superimposed load 2.2 kPa. The manual did have a statement to the effect that larger spans could be possible if span continuity was introduced along with negative reinforcement, but no guidelines on design capacities were given. The manual also indicated 664 mesh was appropriate for a 200 mm deep single span slab. However, this is in contradiction to the Code requirements. With changes in the design code, the current literature specifies a maximum span of 7 m

for a continuous internal span, and 6 m for an end span, and negative steel as H12 bars at 150 mm centres.

42. It is apparent that the original design was from first principles and not to Dimond literature at the time, and going by the current literature the design is beyond the criteria for maximum HiBond span capability. The concrete Code at this time did not address the design of Hi Bond Slabs.

43. The CTV building has an end span of 7.5 meters and negative steel of H 12 at 120 mm centres over the central support beam. This light reinforcement may have contributed to a weakness in the slabs' ability to transfer loads from the structure to the resisting shear walls by diaphragm action. The slab would have been subject to bending stresses as the shear walls moved back and forth during the seismic motion. The H 12 bars in the slab terminated 0.8 to 1.5 m approximately from the edge of the shear walls. There is a point of weakness in the slab at the line at which the abrupt termination of the top slab bars occurs and only 664 mesh is available for negative moments. The mesh has a cross sectional area of 186 mm^2 per linear meter.

(b) Connection to North Shear Core wall

44. The connection to the North Shear Core is limited. In the East- West direction the rear wall of the shear wall is 11.5 meters long and provides the shear capacity in this direction for the wall. However, the connection to this rear wall is only by a slab approximately 3.75 meters wide and 4.5 meters deep. There is also a hole in this slab adjacent to the rear wall resulting in only a 2.35 meter slab connection directly to the wall. This slab has one layer of 664 mesh top throughout as reinforcing, plus short starter bars from the return wall at Line C and Line C/D. This is below the code requirement for steel in a slab.

45. In the North- South direction the two walls on Lines C and C/D are connected by 19 D 12 diameter bars in the slab. The two return walls D & D/E of the North shear wall do not appear to be connected to the main floor slab. The effect of this would be to induce further eccentric behaviour in the wall under North-South seismic action. I have been advised by Counsel Assisting that some attempt was made to connect these walls to the slab at a later date.

46. The Hyland/Ashley-Smith report suggests from the examination of the collapse state of the Building that the North Shear Core was not stressed into the plastic range as a result of the earthquake. Normally I would expect the wall to show signs of large plastic deformation for such seismic loading. I would infer that the wall was not stressed as expected because the wall was not loaded from the main weight of the building. Either the

Building had collapsed or the attachment to the core had been insufficient to transfer the seismic loads.

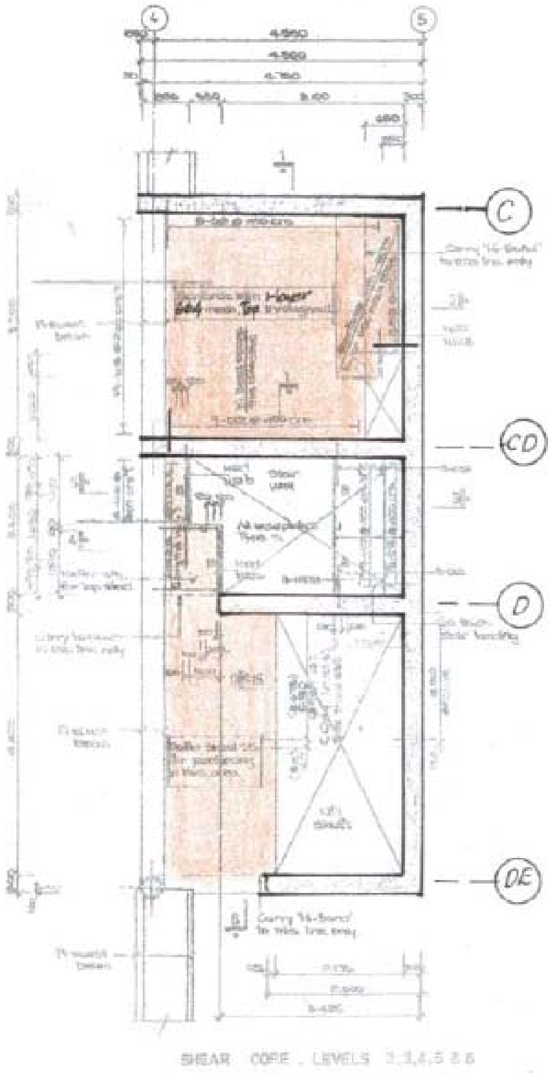

Diagram 5 showing Shear Core North Wall. Shaded areas show connection of slab to rear of wall line (5). The areas with cross lines are opening for stairs, lifts and ducts

Shear in Columns

47. In chapter 7 of the Concrete Code NZS 3101 Part 1: 1982, clause 7.3.4.3 **(ENG.STA.0016.58)** gives the minimum requirement for shear steel:

> Where shear reinforcement is required by 7.3.4.1 or by analysis, minimum area of shear reinforcement for prestressed (except as provided in 7.3.4.4) and non-prestressed members shall be computed by
>
> Av=0.35bws/fy …..(Eq. 7-12)

48. This requirement applies whether the member is a primary seismic resisting element or not. The spiral reinforcing provided in the columns of the CTV Building did not meet these requirements. R6 spirals would have been provided at much closer centres had this

minimum requirement been satisfied. The Hyland/Smith report calculated the spirals required R6 at 90 mm centres. This is 2.7 times as much shear steel as that provided.

Beam-Column Joints:

49. NZS 3101: Part 1:1982 Section 9, applies to the design of beam-column joints. (**ENG.STA.0016.69**):

9.4.1 General:

> Provisions in this Clause 9.4 apply to beam-column joints where gravity actions govern. If the joint is also subject to seismic reversals it shall be checked for compliance with the provisions of 9.5.

50. I have already stated my opinion that the beam-column joints in the CTV Building are subject to seismic load reversals and my reasons for this conclusion. In my view Clause 9.5 applied.

51. **NZS 3101 9.5 Principles and requirements additional to 9.3 for joints designed for seismic loading**

Clause 9.5 outlines design requirements to protect the joint from failure. (**ENG.STA.0016.70**)

> 9.5.1: General. Special provisions are made in this Section for beam-column joints that are subjected to forces arising as a result of inelastic lateral displacements of ductile frames. Joints must be designed in such a way that the required energy dissipation occurs in potential plastic hinges of adjacent members and not in the joint core region.

52. The joint shown in my diagrams S2 and 3 has not been designed to meet these requirements. The provision of one 6 mm diameter spiral does not provide the shear resistance needed to transfer the internal forces generated in a beam-column joint.

Summary

53. The CTV Building design did not comply with the Code or the intent of the Code in respect of the following critical structural elements:

    (a) The Building was not designed to be symmetrical despite several instructions in the Code to design building with symmetrical resisting elements. Although there is no absolute criteria specified by the Code the instruction is clear in NZS 4203: 1984 3.1 and in the Commentary on this clause 3.1.1:…. For high buildings, symmetry is one of the most basic requirements in achieving a structure of predictable performance… **(ENG.STA.0018.38).**

Buildings designed with seismic resisting elements placed to provide a symmetrical resistance to earthquake loads have been found to suffer less damage than asymmetrical buildings. Hence the instruction in the Code.

(b) The columns internal and external to the Building were not designed for ductile behaviour under earthquake loadings. This is despite several clauses in the Code which specified that they should have been designed for ductility.

(c) The designers may have assumed that they could predict accurately that the columns would be subjected to a certain amount of reversed moments from an earthquake and they somehow would not be stressed for that little bit extra that would cause complete catastrophic collapse. This is despite several cautions from the Code pointing out the limitations of assumptions for material properties and theoretical analysis results.

(d) The small diameter columns were heavily loaded and this made them further unable to accept post elastic deformations without failure.

(e) The minimum shear steel required by the Code was not provided in the columns.

(f) There was limited connection to the major shear wall on the North side of the building, situated outside the main floor plate of the building.

DATED 1 June 2012

M. L. Jacobs.

Murray Lionel Jacobs

FOREWORD

General

This standard is a revision, in the means-of-compliance format and using SI units, of NZS 1900* : Chapter 8 : 1965. It aims at setting down minimum requirements for the general run of buildings rather than for special structures (such as bridges, towers, dams, major storage tanks, or special industrial equipment) for which the provisions of this standard may be taken only as a general guide to be supplemented by special studies and judgment.

The Loadings Committee's task in drafting this standard was seen mainly to be one of providing a set of minimum design criteria of an effective and economic nature which would not be too difficult for the designer to apply, but at the same time would leave him scope for innovation and imagination.

The committee believes that the requirements of this standard provide a reasonable level of protection to life and property at an economic level of cost, taking into account the relative seismicity of New Zealand as compared with the rest of the world and the particular building practice and design methods adopted in this country.

General structural design

NZS 1900* : Chapter 8 : 1965 was based on the "working stress" method of design, which is called "the alternative method" in this standard to emphasise that the strength method is preferred.

For the strength method, the load factors and load equations have been derived from ACI-318-71: ACI *Standard Building Code: Requirements for reinforced concrete — 1971*. A load combination probability factor of 0.75 has been applied to load combinations involving dead and live loads and wind or earthquake while for dead and snow loads and wind a factor of 0.85 has been used. For both snow and wind the design loads are based upon return periods of 50 years.

For the alternative method, a significant change is the inclusion of equations to cover reversal of load under wind and earthquake where only dead load is available to stabilize the members. As E is now calculated as a design load for strength design a load factor of 0.8 is necessary.

Dead, live, and snow loads

Live loads have been set out for various types of building use so that the decision as to which loads are applicable may be made more easily.

The levels of live load have been based on BSCP 3 : Chapter V*, AS 1170 : Part 1*, and NZS 1900* : Chapter 8 : 1965.

The procedures for obtaining reduced live load (L_R) are based on those of the National Building Code of Canada*.

The section on snow loads is based on recent work at the New Zealand Agricultural Engineering Institute and information supplied by the New Zealand Meteorological Service. A distinct departure from overseas practice is the use of the "open field snow load" as the basic design load.

Earthquake provisions

Although New Zealand has suffered several major earthquakes since the last revision of NZS 1900* : Chapter 8 in 1965, these did not produce direct local evidence as to the degree of effectiveness of the applicable requirements. However, evidence from recent earthquakes in other countries, in particular Caracas (1967), Tokachi-oki (1968), and San Fernando (1971), has assisted the committee with the present document.

No evidence was available to cause any change in the basic level of seismic coefficients for ductile structures, and it is believed that those previously chosen should in general be left unaltered until shown to be inadequate or excessive by service experience under earthquake attack on buildings detailed for ductility. However, analysis indicates that

* See list of related documents.

3.1 SYMMETRY

3.1.1 The main elements of a building that resist seismic forces shall, as nearly as is practicable, be located symmetrically about the centre of mass of the building.

C3.1.1 It is recognized that the aim to achieve structural symmetry is frequently in conflict with the purpose and architectural design of a building. For high buildings, symmetry is one of the most basic requirements in achieving a structure of predictable performance. Simple geometry is essential for obtaining symmetry in practice. Notwithstanding the availability of modern computers, considerable uncertainty exists in selecting a mathematical model representing the true behaviour of complex arrangements such as combinations of geometrically dissimilar shear walls and unsymmetrical combinations of shear walls and frames. Geometrically dissimilar resisting elements are unlikely to develop their plastic hinges simultaneously, and ductility demand may also be increased by torsional effects.

3.2 DUCTILITY

3.2.1 The building as a whole, and all of its elements that resist seismic forces or movements, or that in case of failure are a risk to life, shall be designed to possess ductility; provided that this shall not apply to small buildings complying with clause 3.4.8.1 designed in accordance with clause 3.4.8.2 nor to tied veneers (item 3 (b) of table 8) and unreinforced or partially reinforced walls and partitions (item 4 of table 8) designed in accordance with clause 3.4.9.

3.2.2 Structural systems intended to dissipate seismic energy by ductile flexural yielding shall have "adequate ductility".

3.2.3 "Adequate ductility" in terms of clause 3.2.2 shall be considered to have been provided if all primary elements resisting seismic forces are detailed in accordance with special requirements for ductile detailing in the appropriate material code.

C3.2 Recent earthquake damage at Tokachi-oki (1968), San Fernando (1971), and elsewhere confirms that all seismic resisting systems, regardless of building height, designed to the seismic loadings of this standard must have ductility with the minor exceptions indicated in clause 3.2.1.

The general requirement for ductility must at present be qualitative rather than quantitative except for buildings designed to dissipate seismic energy by ductile flexural yielding. See also clause 3.2.3.

The requirement of clause 3.2.2 is in effect a practical approximation for the assessment of section curvature ductility demand. A more rigorous analytical approach, which is applicable only to reasonably regular symmetrical frames without sudden changes in storey stiffness, is a method using the following approximate criterion: the building as a whole should be capable of deflecting laterally through at least eight load reversals so that the total horizontal deflection at the top of the main portion of the building under the loadings of equations 4 and 5 and 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 percent. 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 element or that at the earthquake load E calculated on the assumption of elastic behaviour, whichever is the lesser.

For buildings other than reasonably regular symmetrical frames without sudden changes in storey stiffness as provided in clause 3.4.7, maximum section curvature ductility demand should preferably be determined by a non-linear analysis using earthquake inputs appropriate to the site.

Primary members of the seismic resisting system subjected to the displacement or curvature ductility demand determined by the above procedure, are acceptable if they lose no more than 30 percent of their strength and provided the overall building ductility requirements are met.

force resisting system is equal to or greater than 3, then 0.1 V shall be considered as concentrated at the top storey and the remaining 0.9 V shall be distributed in accordance with equation 30.

(b) For chimneys and smoke-stacks resting on the ground, 0.2 V shall be considered as concentrated at the top and the remaining 0.8 V shall be distributed in accordance with equation 30.

(c) For buildings with set-backs the load distribution shall comply with clause 3.4.11.

(d) The distribution of horizontal seismic forces in major buildings that have highly irregular shapes, large differences in lateral resistance or stiffness between storeys, or other unusual structural features shall be determined in accordance with the dynamic analysis procedure of section 3.5.

C3.4.6.1 (d) The provisions of clause 3.4.6.1 (d) will result in base shears similar to those that would be given by the equivalent static force method of analysis, but the distribution of forces will be more appropriate to the particular features of the irregular structures.

Nevertheless it will be necessary to make an equivalent static force analysis for irregular structures in order to obtain the limiting values in clauses 3.5.2.4 and 3.5.2.5. The term "major buildings" is intended to exclude low buildings that do not warrant the more complex procedure of a dynamic analysis. See also section 3.1.

3.4.6.2 At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level.

3.4.6.3 Floors and roofs acting as diaphragms and other principal members distributing seismic forces shall be designed in accordance with clause 3.4.9. Allowance shall be made for any additional forces in such members that may result from redistribution of storey shears.

*3.4.7 Horizontal torsional moments

3.4.7.1 The applicable method of design for torsional moments shall be:

(a) For structures not more than four storeys high or for reasonably regular structures more than four storeys high which are symmetric or of moderate eccentricity, horizontal torsion effects shall be taken into account either by the static method of clause 3.4.7.2, or by the two-dimensional modal analysis method of clause 3.5.2.2.1 (which also uses clause 3.4.7.2), or by the three-dimensional modal analysis method of clause 3.5.2.2.2.

(b) For reasonably regular structures more than four storeys high with a high degree of eccentricity, horizontal torsional effects shall be taken into account either by the static method of clause 3.4.7.2, or by the two-dimensional modal analysis method of clause 3.5.2.2.2. However, it is recommended that the three-dimensional modal analysis of clause 3.5.2.2.2 be used for such structures.

(c) For irregular structures more than four storeys high, horizontal torsional effects shall be taken into account by the three-dimensional modal analysis method of clause 3.5.2.2.2.

C3.4.7.1 Horizontal torsional effects are difficult to estimate. Both excitation and response are known with far less certainty than for translational behaviour. The effects are important however; a number of failures have been caused by horizontal torsion particularly at the ends and corners of buildings, and at re-entrant angles.

A designer's first aim should be to achieve symmetrical structures of similar resisting elements.

Three types of design approach are considered in this standard: a wholly static approach; a combined approach in which the vertical distribution of horizontal forces is given by a two-dimensional modal analysis (clause 3.5.2.2.1) and torsional effects are obtained from the static provisions of clause 3.4.7, and a three-dimensional spectral modal analysis (clause 3.5.2.2.2).

The static method given in clause 3.4.7.2 is intended to apply to reasonably regular buildings such as square, circular, or rectangular structures which have no major re-entrant angles and which are substantially uniform in plan.

Structures of moderate eccentricity are those for which the torsional component of shear load in the element most unfavourably affected does not exceed three quarters of the lateral translational component of shear load.

For exceptionally flexible buildings which are highly irregular and not more than four storeys high (clause 3.4.7.1 (a)) it is recommended that a three-dimensional modal analysis should be used, as the dynamic behaviour in such cases is likely to be more complex than for stiff buildings.

It should be noted that even a three-dimensional modal analysis may not always give good predictions of the dynamic behaviour of very irregular buildings, and may indeed seriously under-estimate earthquake effects in some cases.

It should also be remembered that in torsional situations energy dissipation cannot usually be distributed evenly among resisting elements.

For severely eccentric buildings with L, T, U, or similar irregular plan form, seismic separation of the wing is recommended regardless of the method of analysis.

As less ductile buildings are particularly vulnerable to torsional effects, buildings of more than three storeys and with a structural type factor S equal to or greater than 1.6 should be so designed as to have no worse than moderate eccentricity.

C3.4.7.2 The torsional effects which would be obtained by applying the horizontal force at the centre of mass at each level may be increased due to a number of causes. Some of these are of an accidental nature, such as those resulting from the limitations of stiffness calculation, degree of accuracy of assumptions, mass variation, construction variations, and, in severe earthquakes, asymmetric failure of torsion resisting elements.

Interaction between torsional and translational modes can lead to amplification effects, and torsional ground motion is a further cause of building torsion. The term 0.1b is introduced to allow for all these effects.

3.4.7.2 To provide for shear resulting from torsional motion, the horizontal force at the level considered shall be applied in turn at each of two points distant 0.1b from the centre of mass at that level and either side of it, measured perpendicular to the direction of loading.

3.4.8 Clause deleted.

C3.4.9 In the preparation of clause 3.4.9 the following assumptions were made:

(a) For practical purposes pseudo-acceleration is an acceptable measure of damage potential to fixed parts and portions in a building. (See also clause 3.8.4.)

(b) The maximum accelerations in a building prior to yielding are related to its elastic damped response, and after yielding to the C_d values.

(c) Because of the requirements of New Zealand standards for design in various materials, and because of design and detailing practices (such as under-capacity factors, use of minimum yield stresses, rounding-up of sizes and the like), building over-capacities corresponding to 1.5 C_d should be common in future buildings, resulting in accelerations greater than $K_x C_d$.

(d) These accelerations will be reached in earthquakes of moderate intensity and will thus be relatively frequent in modern buildings having low damping. (This

3.4.9 Parts or portions of buildings

3.4.9.1 Except as provided by clause 3.4.9.3 and subject to section 3.6, any part or portion of a building shall be designed for a seismic force F_p in each direction under consideration as given by:

$$F_p = C_p W_p \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (33)$$

where

C_p shall be as given by clause 3.4.9.2 and

W_p shall be determined in the same manner as W_t (see clause 3.4.2.1).

NEW ZEALAND STANDARD

Code of practice for
THE DESIGN OF CONCRETE STRUCTURES

1 GENERAL

1.1 Scope. This New Zealand Standard Code of Practice specifies minimum requirements for the design of reinforced and prestressed concrete structures. It serves as a means of compliance with the relevant requirements of NZS 1900, Chapter 9.3.

It is applicable only to structures and parts of structures complying with the materials and workmanship requirements of NZS 3109.

For special structures such as shells, arches, tanks, reservoirs, bins and silos, blast-resistant structures and chimneys, the provisions of this Standard Code of Practice shall govern where applicable.

1.2 Interpretation

1.2.1 In this Standard the word "shall" indicates a requirement that is to be adopted in order to comply with the standard, while the word "should" indicates a recommended practice.

1.2.2 Cross-references to other clauses or clause subdivisions within this Standard quote the number only, for example: " . . . as required by 4.4.1.3 (d) for shored construction."

1.2.3 The full titles of reference documents cited in this Standard are given in the "List of related documents" immediately preceding the Foreword.

1.2.4 Where any other standard named in this Standard has been declared or endorsed in terms of the Standards Act 1965, then:

(a) Reference to the named standard shall be taken to include any current amendments declared or endorsed in terms of the Standards Act 1965; or

(b) Reference to the named standard shall be read as reference to any standard currently declared or endorsed in terms of the Standards Act 1965 as superseding the named standard, including any current amendments to the superseding standard, declared or endorsed in terms of the Standards Act 1965.

NOTE – The date at which an amendment or superseding standard is regarded as "current" is a matter of law depending upon the particular method by which that standard becomes legally enforceable in the case concerned. In general, if this is by contract the relevant date is the date on which the contract is created, but if it is by Act, regulation, or bylaw then the relevant date is that on which the Act, regulation, or bylaw is promulgated.

3.5 Principles and requirements additional to 3.3 for the analysis and design of structures subjected to seismic loading

3.5.1 *Methods of design*

3.5.1.1 To provide minimum resistance for the appropriate combination of gravity and seismic loads specified by NZS 4203 or other appropriate loading code, design methods shall be used which are applicable to the structural systems as follows:

(a) Ductile structures resisting seismic loading and undergoing inelastic displacements are required to dissipate energy by ductile flexural yielding in specified localities of the structure. Ductile structures shall be subject to capacity design as defined in Section 2. Adequate ductility and hysteretic dissipation of seismic energy may be considered to have been provided for, if all primary earthquake resisting elements of such structures are designed and detailed in accordance with this Code

(b) Structures of limited ductility are assumed to have low inelastic deformation demand and are designed to resist seismic loads derived with the use of larger structural type factors, as specified in NZS 4203 or other appropriate loading code. Member strength is determined either with capacity or strength design procedures according to Section 14

(c) Elastically responding structures are not expected to develop inelastic deformations while resisting the largest seismic loads specified by NZS 4203, or other appropriate loading code. Accordingly they may be designed to conform to 3.3 and are exempt from the seismic requirements for detailing for ductility.

3.5.1.2 For structures subjected to seismic loading, the alternative method of design, given in Appendix B, shall not be used.

3.5.1.3 Wherever the requirements of a capacity design procedure apply, the maximum member actions to be expected during large inelastic deformations of a structure shall be based on the overstrength of the potential plastic hinges.

3.5.1.4 The interaction of all structural and non-structural elements which, due to seismic displacements, may affect the response of the structure or the performance of non-structural elements, shall be considered in the design of that structure.

3.5.1.5 Consequences of failure of elements that are not a part of the intended primary system for resisting seismic forces shall also be considered.

3.5.1.6 Floor and roof systems in buildings shall be designed to act as horizontal structural elements, where required, to transfer seismic forces to frames or structural walls.

3.5.1.7 Structural systems and design methods, other than those covered in this Code, may be used only if it can be shown by analysis or experiment, based on accepted engineering principles, that adequate strength, stiffness and ductility for the anticipated seismic movements have been provided for.

3.5.2 *Seismic loading*

3.5.2.1 In the derivation of the lateral seismic loading, to be considered with the appropriately factored gravity load, the structural type factor S, the structural material factor M, specified by NZS 4203 or other approved codes, shall be used. The same structural type factor S shall be substituted in all relevant equations of the additional seismic requirements of this Code.

3.5.2.2 Where modified capacity design procedures are used, the appropriate factors for member overstrength, dynamic moment and shear magnification shall be used to determine the design actions on members.

3.5.2.3 In considering the concurrency of seismic effects in two-way horizontal force resisting systems the following requirements shall be satisfied:

(a) Columns and walls, including their joints and foundations, which are part of a two-way horizontal force resisting system, shall be designed, in accordance with the requirements of NZS 4203, for concurrent effects resulting from the simultaneous yielding of all beams or diagonal braces framing into such columns or walls from all directions at the level under consideration and as appropriate at other levels

(b) When the design actions on columns, walls or foundations have been derived from capacity design procedures with appropriate magnifications for dynamic, concurrency and other extreme seismic effects, the intent of 3.5.2.3 (a) may be deemed to have been satisfied if components of such two-way framing systems are designed separately for the maximum actions so derived for each of the principal directions of the seismic loading

(c) Bridge members shall be designed for any additional forces resulting from seismic actions along both major axes of the structure concurrently, such as those due to friction or shear stiffness of devices intended to prevent horizontal movement in a direction perpendicular to that being considered.

3.5.3 *Assumptions and methods of analysis*

3.5.3.1 In determining the minimum strengths for members, designed for the maximum effects of factored static loads determined by elastic analysis, or for effects derived from dynamic analysis, as permitted by NZS 4203 or other appropriate loading code, the strength reduction factors specified in Section 4 shall be used.

3.5.3.2 Structures classified in 3.5.1.1 (a), such as ductile frames composed of beams and columns with or without shear walls, and also cantilever or coupled shear walls and bridge piers, shall be assumed to be forced into lateral deformations sufficient to create reversible plastic hinges by actions of a severe earthquake.

C1.3.2.2 In determining the maximum load effects due to live load and earthquake in continuous beams and the like, it is acceptable to assume that all spans carry reduced live loads, or that there is no live load, whichever combination produces the greater strength demand.

C1.3.2.3 The word 'loads' is used loosely and is here employed as a general term standing for either loads or member internal actions such as bending moments and shear forces. The fact that different load factors are used for dead and live loads reflects the greater certainty with which the dead load is known. The values of live load given in table 2 could have been increased by a factor of (1.7/1.4) so as to allow a factor of 1.4 to be applied to both dead and live load in equation 1, but it was decided to keep the live loads at generally familiar levels and use separate load factors instead.

Equation 2 is derived from

$$U = 3/4\,(1.4\,D + 1.7\,L_R + 1.7\,W)$$

and equation 4 is derived from

$$U = 3/4\,(1.4\,D + 1.7\,L_R) + E$$

but terms 1.05 D and 1.27 L_R have been rounded off to 1.0 D and 1.3 L_R respectively.

Although significant vertical acceleration components of ground motions have been recorded during earthquakes (for example 0.2 to 0.3 g in the 1971 San Fernando earthquake) no vertical acceleration load terms have been included in the design loads of this standard except for parts such as horizontal cantilevers and anchorage of machinery because there is at present no certainty about the damage potential of combined dynamic effects.

In New Zealand, snow loads are of short duration at low altitudes, so that the probability of a severe earthquake coinciding with a full snow load is remote. Accordingly, equation 7A applies only to buildings at comparatively high altitudes.

1.3.2.2 Except for load cases involving earthquake, the most adverse distribution of live loads shall be considered. For seismic loading cases, the reduced live load may be considered to be applied uniformly over all relevant floor areas.

1.3.2.3 The design loads U for the strength method shall be not less than whichever of the following load combinations is applicable and gives the greatest effect:

	$U = 1.4D + 1.7L_R$	(1)
with wind	$U = 1.0D + 1.3L_R + 1.3W$	(2)
	$U = 0.9D + 1.3W$	(3)
with earthquake	$U = 1.0D + 1.3L_R + E$	(4)
	$U = 0.9D + E$	(5)
with snow	$U = 1.4D + 1.4S$	(6)
	$U = 1.2D + 1.2S + 1.1W$	(7)
with snow and at altitudes exceeding 1500 m in snow zone 1 and 1000 m in snow zones 2, 3, 4, and 5.	$U = D + S + E$	(7A)
with earth pressure	$U = 1.4D + 1.7L_R + 1.7Q$	(8)
	$U = 0.9D + 1.7Q$	(9)
with liquid pressure	$U = 1.4D + 1.7L_R + 1.4F$	(10)
	$U = 0.9D + 1.4F$	(11)

1.3.2.4 When forces are included that result from contained liquids or solids, filled to maximum capacity, the load factors for contents shall be those applicable to dead loads and not those applicable to live loads or reduced live loads.

1.3.2.5 Impact effect, if any, shall be included with the live load L.

1.3.2.6 Where the structural effects of differential settlement, creep, shrinkage, or temperature change might be significant, realistic service loads only may be used in assessing deformations, crack widths, or other forms of local damage.

C1.3.3 Previously, the alternative method was generally referred to as the working stress method. Use of the alternative method for any loading case involving earthquake loads is provided for in some materials codes.

1.3.3 Design load combinations: Alternative method

1.3.3.1 Where the alternative method is permitted, structures and members designed by the alternative method shall be designed in accordance with the allowable stresses given in the relevant material code to resist the loading combinations specified in clause 1.3.3.3 as applicable, except that none of: wind load, roof live load, earthquake load, and other transient dynamic effects, need be combined with each other.

CALCULATIONS

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CHRISTCHURCH

PAGE G41A.
SECT
FILE
DATE

Column hoops:

min bar dia 5.3 29.2 = 6 mm — all hoops

$\phi' = 0.7$ — R6 at 250. spiral.

non-seismic loading case 6.4.7.1

max spacing = 400 mm

16 x 20 = 320.

48 x 6 = 288

use R6 at 250.

Compare seismic loading case in potential p'hinge regions.

$$0.45\left(\frac{A_g}{A_c}-1\right) = 0.45\left(\frac{400^2}{300^2}-1\right) = 0.35 \;\; > .12$$

$f'_c = 35$ $\quad \rho_s = 0.35 . \frac{35}{275}\left(0.5 + 1.25 . 0\right) = 0.0223$

$\therefore A_{st} = .0223 \times \frac{\pi . 300^2}{4} = 1576 \; mm^2/m$

R10 at 100 A_{st} = 1570

for ○ R6 at 40 A_{st} = 1413

max spacing s/f = 400/5 = 80

(ii) 6 x 20 = 120

(iii) = 200

– these do not apply as columns are non-seismic.

(b) The requirements of Section 14, wherever the actions that could be transmitted by the superstructure at the top of the foundations are equal or larger than those which would result from the application of lateral earthquake loading to the superstructure corresponding with $SM = 1.6$.

3.5.12.5 *Rocking foundations.* When special studies are carried out to the satisfaction of the Engineer, structural walls may be assumed to limit the seismic loads induced in the structure by rocking with their foundations, provided that:

(a) The vertical design loads on the foundations are determined from factored gravity loads together with overstrength contributions of adjacent slabs, beams and other elements which may be yielding during the rocking of the wall system, and having regard to all accelerations induced in the superstructure during rocking

(b) The lateral design load acting simultaneously with the vertical forces, in accordance with 3.5.12.4 (a), are determined from special studies.

3.5.12.6 *Lateral forces on retaining walls and piles.* Particular attention shall be given to forces that might develop against retaining walls and piles during earthquakes.

3.5.12.7 *Uplift forces.* Uplift forces that may act on foundation pads during earthquakes, shall be considered to ensure that, when necessary, adequate flexural tension reinforcement is provided in the top of isolated footing pads or in other localities of continuous or combined footings or rafts, where under gravity load compression stresses would prevail. Such reinforcement shall not be less than 0.001 times the gross sectional area of such a pad.

3.5.13 *Structures incorporating mechanical energy dissipating devices.* The design of structures incorporating flexible mountings and mechanical energy dissipating devices is acceptable provided that the following criteria are satisfied:

(a) The performance of the devices used is substantiated by tests

(b) Proper studies are made towards the selection of suitable design earthquakes for the structure

(c) The degree of protection against yielding of the structural members is at least as great as that implied in this Code relating to the conventional seismic design approach without energy dissipating devices

(d) The structure is detailed to deform in a controlled manner in the event of an earthquake greater than the design earthquake.

3.5.14 *Secondary structural elements*

3.5.14.1 Secondary elements are those which do not form part of the primary seismic force resisting system, or are assumed not to form such a part and are therefore not necessary for the survival of the building as a whole under seismically induced lateral loading, but which are subjected to loads due to accelerations transmitted to them, or due to deformations of the structure as a whole. These are classified as follows:

(a) Elements of Group 1 are those which are subjected to inertia loading but which, by virtue of their detailed separations, are not subjected to loading induced by the deformation of the supporting primary elements or secondary elements of Group 2

(b) Elements of Group 2 are those which are not detailed for separation, and are therefore subjected to both inertia loadings, as for Group 1, and to loadings induced by the deformation of the primary elements.

3.5.14.2 Group 1 elements shall be detailed for separation to accommodate deformations $\nu\Delta$ and Δ_p. Such separation shall allow adequate tolerances in the construction of the element and adjacent elements, and, where appropriate, allow for deformation due to other loading conditions such as gravity loading. For elements of Group 1:

(a) Loading E_p used in the design shall be that specified in NZS 4203

(b) Analysis may be by any rational method

(c) Detailing shall be such as to allow ductile behaviour and in accordance with the assumptions made in the analysis. Fixings for precast units shall be designed and detailed in accordance with 3.5.15.

3.5.14.3 Group 2 elements shall be detailed to allow ductile behaviour and in accordance with the assumptions made in the analysis. For elements of Group 2:

(a) Additional seismic requirements of this Code need not be satisfied when the design loadings are derived from the imposed deformations $\nu\Delta$, specified in NZS 4203, and the assumptions of elastic behaviour

(b) Additional seismic requirements of this Code shall be met when plastic behaviour is assumed at levels of deformation below $\nu\Delta$

(c) Inertia loading E_p shall be that specified by NZS 4203

(d) Loadings induced by the deformation of the primary elements shall be those arising from the level of deformation $\nu\Delta$, specified in NZS 4203 having due regard to the pattern and likely simultaneity of deformation

(e) Analysis may be by any rational method, in accordance with the principles of elastic or plastic theory, or both. Elastic theory shall be used to at least the level of deformation corresponding to and compatible with one-quarter of the amplified deformation, $\nu\Delta$, of the primary elements, as specified in NZS 4203 optimum solution between construction economies and anticipated frequency of earthquake induced damage. However, the Code requires that the degree of protection against yielding of the structural members is to be at least as great as that implied for the conventional seismic design approach without dissipators. (In many cases this could be achieved with substantial construction cost savings. That is, the lower structural member strength requirements more than compensate for the extra costs of the devices.) It is recommended that the extent to which the degree of protection is increased above that minimum, to reduce the anticipated frequency of earthquake induced damage, should be resolved with regard to the client's wishes.

Extreme earthquake. For an extreme earthquake there is to be a suitable hierarchy of failure of the structural and foundation members that will preclude a brittle collapse. This may be achieved by appropriate margins of strength between non-ductile and ductile members and with attention to detail.

Although the design criteria outlined above encompass three earthquake levels, the design practice need only be based on the "design" earthquake. In the course of that design, the implications of yield levels on response to the "moderate" earthquake would have to be considered, as would also the implications of strength margins and detailing for an "extreme" earthquake. In general, the lower ductility demand on the structure means that the simplified detailing procedures of Section 14 would be satisfactory.

Because application of these devices to seismic resistant structures is still in its infancy, dynamic inelastic analyses should generally be undertaken for design purposes. Such analyses should consider acceleration records appropriate for the site, in particular taking account of any possibility of long period motions. As experience is accumulated, there is potential for development of standardized design procedures for common applications.

C3.5.14 *Secondary structural elements*

C3.5.14.1 The definition of a secondary element is more particular than that in NZS 4203, and includes such primary gravity-load resisting elements as frames which are in parallel with stiff shear walls and do not therefore participate greatly in resistance to lateral loads. Caution must however be exercised in assumptions made as to the significance of participation. Frames in parallel with slender shear walls should be designed and detailed as fully participating primary members. For convenience of reference and specification of requirements, secondary elements have been subdivided into groups, that is, Group 1 and Group 2 elements.

C3.5.14.2 To avoid any form of deformation – induced loading, in Group 1 elements, separations must be meticulously detailed. Similarly close attention must be given to details of supports, and to their positioning. Reference[3.26] discusses separation, while reference[3.27] discusses such aspects as the conflict between these separation requirements and the requirements of sound attenuation, fire protection and the like. The loading is specified as an equivalent static load. Since these loads are already scaled to account for amplification of accelerations within the structure, no additional scaling of deflections and element actions is required. Often Group 1 elements are geometrically complex, and where appropriate the yield line method, for instance, of Section 11 would be appropriate to their analysis.

Ductile behaviour remains the prime objective of adequate detailing and must be sought by the detailer. The details however need not be elaborate to allow such behaviour. Wall panels, for instance, may be reinforced with a single layer of reinforcement without any additional confinement, and still provide adequate ductility.

C3.5.14.3 In the consideration of Group 2 elements:

(a) The additional seismic requirements of the relevant sections of the Code need not be complied with when the elastic deformation-induced actions on the element are derived from elastic analysis using deformation not less than $\nu\Delta$

(b) Where ductile action is relied on to produce adequate inelastic deformation capacity, all additional seismic detailing requirements of relevant sections must be met

(c) NZS 4203 sets out the requirements to be met in regard to inertia loading and to amplified deformations, and the commentary to that code provides guidance on methods of calculation

(d) The deformation calculated in accordance with NZS 4203 may be exceeded in some structures and in localized areas. Furthermore the pattern of deformation will usually vary significantly from the first mode pattern assumed in calculation. These variations should be taken into account in assessing member actions when they might have a marked effect on element performance

(e) In certain cases elastic response may not be desirable, as forces may become excessive and even lead to inferior performance of the primary structure. Therefore inelastic action is permissible. However, elements must be designed for at least the elastic fraction of the total deformation of the primary elements, to prevent excessive damage in moderate earthquakes. Normally elastic actions will be selected. In most instances achievement of this will not prove to be unduly onerous. In many cases design will be controlled by

$$U = 1.4D + 1.7L_R$$

For the seismic load to predominate would require in this event $\nu E^* > 0.25\ (1.4D + 1.7L_R)$ in which E^* is the action induced by Δ, and in which $0.75\ (1.4D + 1.7L_R)$ is an approximation for $D + 1.3L_R$ (for a definition of terms refer to NZS 4203). Therefore with $\nu = 2.5$:

$E^* < 0.1\ (1.4D + 1.7L_R)$ would require no additional demand on strength

(f) Inelastic action may only be assumed when detailing allows adequate ductility. Where strength is derived

10.5.4.4 In areas where the compressive yield strength of the longitudinal bars, required for the ideal strength of the wall section in accordance with the appropriate bending moment envelope, cannot be developed, lateral reinforcement shall be provided as required for beams by 6.4.7.2(b) unless:

(a) For the same load conditions the steel compression stresses will not exceed $0.5 f_y$

(b) The wall is exempted from the requirements for transverse reinforcement in accordance with 10.5.4.3.

10.5.4.5 When the neutral axis depth in the potential yield regions of a wall, computed for the appropriate design loading, exceeds

$$c = 0.10\, \phi_o\, S \ell_w \quad \text{(Eq. 10-3)}$$

or the value obtained from more detailed calculation

$$c = \frac{8.6\, \phi_o\, S\, \ell_w}{(4 - 0.7S)\left(17 + \frac{h_w}{\ell_w}\right)} \quad \text{(Eq. 10-4)}$$

the following requirements shall be satisfied in the outer half of that part of the wall section which is subjected to compression strains due to the design loading:

(a) Rectangular or polygonal closed hoops, surrounding longitudinal bars, shall be used as in confined columns so that

$$A_{sh} = 0.3\, s_h h'' \left(\frac{A_g^*}{A_c^*} - 1\right) \frac{f'_c}{f_{yh}} \left(0.5 + 0.9 \frac{c}{\ell_w}\right) \quad \text{(Eq.10-5)}$$

or

$$A_{sh} = 0.12\, s_h h'' \frac{f'_c}{f_{yh}} \left(0.5 + 0.9 \frac{c}{\ell_w}\right) \quad \text{(Eq. 10-6)}$$

whichever is greater, where the ratio c/ℓ_w need not be taken more than 0.8

(b) Longitudinal bars shall be restrained against possible buckling in accordance with 10.5.4.3 (a)

(c) The centre-to-centre spacing of hoops along longitudinal bars shall not exceed six times the diameter of the longitudinal bar, nor one-half of the thickness of the confined region of the wall, nor 150 mm

(d) The potential yield region of the wall, over which the requirements for hoops in accordance with 10.5.4.5 (a) to (c) is to be satisfied, shall be assumed to extend above the critical section by ℓ_w or 1/6 of height of wall measured to the top of the wall, whichever is larger

(e) Walls with a single layer of reinforcement shall not be used.

10.5.5 *Shear strength*

10.5.5.1 The evaluation of shear strength of, and the determination of shear reinforcement for, walls shall be in accordance with Section 7. For ductile walls, conforming with requirements of Section 10, the shear stress shall not be greater than permitted by 7.3.14.

10.5.5.2 In the end region of ductile walls the shear stress limitations of 7.5.5.2 shall not be exceeded.

10.5.5.3 The height of the end region in walls, for which the special shear stress limitations apply, shall be taken as the length of the wall ℓ_w or 1/6 of the height of the wall, whichever is larger, measured from the section at which the first flexural yielding is expected. The height of the end region need not be taken larger than 2 ℓ_w.

10.5.5.4 Where applicable, ties may be assumed to contribute to the shear strength of a wall element.

10.5.6 *Diaphragms*

10.5.6.1 Diaphragms, intended to transfer earthquake induced horizontal floor forces to primary lateral load resisting elements or which are required to transfer horizontal seismic shear forces from one vertical primary lateral load resisting element to another, shall be designed for the maximum forces that can be resisted by the vertical primary load resisting system, or for forces corresponding with the seismic design coefficients specified by NZS 4203 for parts or portions of buildings, whichever is smaller.

10.5.6.2 Diaphragms shall be reinforced in both directions with not less than the minimum reinforcement required for two-way slabs in accordance with 5.3.32.

10.5.6.3 When it is shown that a diaphragm can introduce forces required to develop the overstrength of the primary lateral load resisting system, without yielding in the diaphragm; or at dependable strength the forces specified in NZS 4203, the special requirements of seismic detailing of the diaphragm for ductility need not be complied with.

10.5.6.4 When the design forces to be transmitted by diaphragms do not lead to the development of the full strength of the primary lateral load resisting system, diaphragms shall be designed in accordance with the requirements of 14.9.

10.5.6.5 Where joints across diaphragms are provided, only the effective area over which interface shear transfer, in accordance with 7.3.11 can occur, shall be considered.

10.5.6.6 Where precast elements are used for floor construction, cast-in-place reinforced concrete topping, at least 50 mm thick, may be used to transfer seismic shear forces through diaphragm action, provided that:

(a) Minimum reinforcement in two directions in accordance with 5.3.32 is placed in the topping slab

5.3.26.3 At least one-third the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection, according to the appropriate bending moment envelope, for a distance of not less than 1.3 times the effective depth of the member.

5.3.27 *Special details for columns and piers*

5.3.27.1 Where longitudinal bars are offset, the slope of the inclined portion of the bar with the axis of the column shall not exceed 1 in 6, and the portions of the bar above and below the offset shall be parallel to the axis of the column. Adequate horizontal support at the offset bends shall be treated as a matter of design, and shall be provided by ties, spirals or parts of the floor construction. Ties or spirals so designed shall be placed not more than 150 mm from the point of bend. The horizontal thrust to be resisted shall be assumed as one and one-half times the horizontal component of the nominal force in the inclined portion of the bar, assumed to be stressed to f_y.

5.3.27.2 Where column faces are offset 75 mm or more, splices of vertical bars adjacent to the offset face shall be made by separate dowels lapped as required herein.

5.3.27.3 Where the design load stress in the longitudinal bars in a column calculated for any loading condition exceeds 0.5 f_y in tension, lap splices designed for full yield stress in tension, or high strength welded splices or high strength mechanical connections in accordance with 5.3.17.6 (b) and (c) shall be used.

5.3.27.4 Steel cores in composite columns shall be accurately finished to bear at end bearing splices, and positive provision shall be made for alignment of one core above another. Bearing shall be considered effective to transfer 50% of the total compressive stress in the steel core. At the column base, provision shall be made to transfer the load to the footing, in accordance with 12.3.7.

The base of the metal section shall be designed to transfer the load from the entire composite column to the footing, or it may be designed to transfer the load from the metal section only, provided it is so placed as to leave ample section of concrete for the transfer of load from the reinforced concrete section of the column by means of bond on the vertical reinforcement and by direct compression of the concrete.

The steel core shall comply with NZS 3404.

5.3.28 *Connections.* At connections of principal framing elements such as beams and columns, enclosure shall be provided for splices of continuing reinforcement and for end anchorage of reinforcement terminating in such connections. Such enclosure may consist of external concrete or internal closed ties, spirals or stirrups. Joints shall be subject to rational analysis in accordance with Section 9.

5.3.29 *Spiral or circular hoop reinforcement for columns and piers*

5.3.29.1 Spiral or circular hoop reinforcement shall be of such size and so assembled to permit handling and placing without distortion from designed dimensions.

5.3.29.2 For cast-in-place construction, size of spiral or circular hoop bar shall not be less than 6 mm diameter.

5.3.29.3 Anchorage of a spiral bar at the termination of the length of spiral shall be provided by an extra one-half turn of spiral bar plus either a 135° stirrup hook or welding the spiral bar on to the previous turn to develop in tension 1.6 f_y or the breaking strength of the bar, whichever is smaller.

5.3.29.4 Spiral or circular hoop bar shall not be lap spliced.

5.3.29.5 Ends of circular hoop bar, or spiral bar within the length of the spiral, shall either be welded to develop the breaking strength of the bar, or anchorage may be provided by at least a 135° stirrup hook.

5.3.29.6 Spacing and arrangement of spiral or circular hoop reinforcement are covered in 5.4.1 and 5.5.4.

5.3.30 *Rectangular hoop and tie reinforcement for columns and piers*

5.3.30.1 Rectangular hoop or tie reinforcement shall be at least 6 mm in diameter for longitudinal bars less than 20 mm in diameter, 10 mm in diameter for longitudinal bars from 20 to 32 mm in diameter and 12 mm in diameter for longitudinal bars 36 mm in diameter or larger and for bundled longitudinal bars.

5.3.30.2 Rectangular hoop or tie reinforcement shall enclose all longitudinal bars.

5.3.30.3 Spacing, arrangement and anchorage of rectangular hoop and tie reinforcement are covered by 5.4.2 and 5.5.5.

5.3.31 *Stirrup and tie reinforcement in beams*

5.3.31.1 Stirrup or tie reinforcement shall satisfy the size limitations in 5.3.30.1.

5.3.31.2 Stirrup or tie reinforcement shall enclosed the longitudinal compression reinforcement in beams.

5.3.31.3 Spacing, arrangement and anchorage of rectangular stirrup and tie reinforcement in flexural members are covered by 5.4.3, 5.5.6 and 7.3.5.

5.3.32 *Shrinkage and temperature reinforcement*

5.3.32.1 Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement shall be provided in structural floor and roof slabs where the principal reinforcement extends in one direction only. At all sections where it is required, such reinforcement shall be developed for its specified yield strength in conformance with 5.3.6 or 5.3.18. Such reinforcement shall provide at least the following ratios of reinforcement area to gross concrete area, but not less than 0.0014 and in no case shall such reinforcement be placed farther apart than five times the slab thickness nor more than 450 mm in buildings, nor 300 mm in bridges.

centroidal axis of member, or at intersection of flange and web when centroidal axis is in the flange. In composite members principal tensile stress shall be computed using the cross-section that resists live load.

7.3.3.4 In a pre-tensioned member in which the section at a distance $h/2$ from face of support is closer to end of member than the transfer length of the prestressing tendons, the reduced prestress shall be considered when computing v_{cw}. This value of v_{cw} shall also be taken as the maximum limit for eq. 7-8. Prestress force may be assumed to vary linearly from zero at end of tendon to a maximum at a distance from end of tendon equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

7.3.3.5 With the exception of the allowance in eq. 7-11 the transverse component of the longitudinal prestressing force V_p, shall not be considered to contribute to the shear resistance of prestressed concrete beams.

7.3.4 *Shear reinforcement - Minimum requirements*

7.3.4.1 A minimum area of shear reinforcement shall be provided in all reinforced, prestressed and non-prestressed concrete where shear stress v_i required to resist V_u exceeds half the shear strength provided by concrete v_c, except:

(a) Slabs and footings

(b) Concrete joist construction defined by 3.4.2

(c) Beams with total depth not greater than 250 mm, two and a half times thickness of flange, or one-half the width of web, whichever is greater.

7.3.4.2 Minimum shear reinforcement requirements of 7.3.4.1 may be waived if shown by test that required ultimate flexural and shear strength can be developed when shear reinforcement is omitted.

7.3.4.3 Where shear reinforcement is required by 7.3.4.1 or by analysis, minimum area of shear reinforcement for prestressed (except as provided in 7.3.4.4) and non-prestressed members shall be computed by

$$A_v = 0.35\frac{b_w s}{f_y} \quad \ldots\ldots \quad \text{(Eq. 7-12)}$$

where b_w and s are in millimetres.

7.3.4.4 For prestressed members with effective prestress force not less than 40% of the design tensile strength of flexural reinforcement, minimum area of shear reinforcement may be computed by eq. 7-12 or 7-13

$$A_v = \frac{A_{ps}}{80}\cdot\frac{f_{pu}}{f_y}\cdot\frac{s}{d}\sqrt{\frac{d}{b_w}} \quad \ldots\ldots \quad \text{(Eq. 7-13)}$$

7.3.5 *Shear reinforcement details*

7.3.5.1 Shear reinforcement may consist of:

(a) Stirrups perpendicular to axis of member

(b) Welded wire fabric with wires located perpendicular to axis of member

(c) Stirrups making an angle of 45° or more with the longitudinal tension bars

(d) Vertical or inclined prestressing.

7.3.5.2 For non-prestressed members, shear reinforcement may also consist of:

(a) Longitudinal reinforcement with bent portion making an angle of 30° or more with the longitudinal tension reinforcement

(b) Combinations of stirrups and bent longitudinal reinforcement

(c) Spirals.

7.3.5.3 Stirrups and other bars or wires used as shear reinforcement shall extend to a distance d from extreme compression fibre and shall be anchored at both ends according to 5.4.3.2 to develop the yield strength of reinforcement.

7.3.5.4 Spacing limits for shear reinforcement shall be as follows:

(a) Spacing of shear reinforcement, placed perpendicular to axis of member, shall not exceed the lesser of either (1) and (2) as appropriate or (3):

  (1) 0.5 d in non-prestressed members

  (2) 0.75 h in prestressed members and non-prestressed members provided that P_u/A_g exceeds $0.12 f'_c$

  (3) 600 mm

(b) Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45° line, extending towards the reaction from mid-depth of member 0.5 d to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement

(c) When $(v_i - v_c)$ exceeds $0.07 f'_c$ maximum spacings given in 7.3.5.4 (a) and (b) shall be reduced by one-half.

7.3.6 *Design of shear reinforcement*

7.3.6.1 Design yield strength of shear reinforcement shall not exceed 415 MPa.

7.3.6.2 When the total shear stress v_i exceeds the ideal shear stress provided by concrete, v_c, shear reinforcement shall be provided for the difference $(v_i - v_c)$.

9 BEAM-COLUMN JOINTS

9.1 Notation

- A_g gross area of section, mm^2
- A_{jh} total area of effective horizontal joint shear reinforcement, mm^2
- A_{jv} total area of effective vertical joint shear reinforcement, mm^2
- A_s area of non-prestressed tension beam reinforcement, mm^2
- A'_s area of non-prestressed compression beam reinforcement, mm^2
- A_{sc} area of non-prestressed tension reinforcement in one face of the column section, mm^2
- A'_{sc} area of non-prestressed compression reinforcement in one face of the column section, mm^2
- b_c overall width of column, mm
- b_j effective width of joint, mm
- b_w web width of beam, mm
- C_j $\dfrac{V_{jh}}{V_{jx} + V_{jz}}$
- eccentricity between the centre lines of the webs of a beam and a column at a joint, mm
- f'_c specified compressive strength of concrete, MPa
- f_y specified yield strength of non-prestressed reinforcement, MPa
- h_b depth of beam, mm
- h_c overall depth of column in the direction of the horizontal shear to be considered, mm
- P_{cs} force after all losses in prestressing steel passing through a joint, N
- P_e design axial load in compression with given eccentricity due to gravity and seismic loading acting on the member during an earthquake, N
- P_u design axial compression column load including vertical prestressing where applicable occurring simultaneously with V_{jh}, N
- v_{jh} nominal horizontal shear stress in joint core, MPa
- V_{col} horizontal shear force across a column, N
- V_{ch} ideal horizontal joint shear strength provided by concrete shear resisting mechanism only, N
- V_{cv} ideal vertical joint shear strength provided by concrete shear resisting mechanism only, N
- V_{jh} total horizontal shear force across a joint, N
- V_{jv} total vertical shear force across a joint, N
- V_{jx} total horizontal joint shear force in x direction, N
- V_{jz} total horizontal joint shear force in z direction, N
- V_{sh} ideal horizontal joint shear strength provided by horizontal joint shear reinforcement, N
- V_{sv} ideal vertical joint shear strength provided by vertical joint shear reinforcement, N
- ϕ strength reduction factor:
  1.0 where joint forces are derived from overstrength member actions, or 0.85 in other cases

9.2 Scope

9.2.1 Provisions of this Section apply to design of beam-column joints subject to shear induced by gravity or earthquake loads or both. Design for shear in slab-column connections is to be in accordance with 7.3.15 and 7.3.16.

9.3 General principles and requirements

9.3.1 Beam–column joints shall satisfy the following criteria:

(a) A joint shall perform under service loads at least as well as the members that it joins

(b) A joint shall have a dependable strength sufficient to resist the most adverse load combinations sustained by the adjoining members, as specified by the appropriate loadings code, several times where necessary.

9.4 Principles and requirements additional to 9.3 for joints not designed for seismic loadings

9.4.1 *General.* Provisions in this Clause 9.4 apply to beam–column joints where gravity load actions govern. If the joint is also subject to seismic load reversals it shall be checked for compliance with the provisions of 9.5.

9.4.2 *Design forces.* The design forces acting on a beam-column joint shall be evaluated from the maximum stresses generated by all members meeting at the joint, subjected to the most adverse combination of loads as required by the appropriate loadings code, with the joint in equilibrium. At columns of two-way frames, where beams frame into the joint from two directions, these forces need only be considered in each direction independently.

9.4.3 *Strength reduction factor.* In determining the shear strength of the joint the value of the strength reduction factor ϕ shall be 0.85.

9.4.4 *Maximum permissible horizontal stress.* The nominal horizontal shear stress in the joint shall not exceed that specified in 9.5.3.2.

9.4.5 *Design principles.* The joint shear shall be assumed to be resisted by a concrete mechanism plus a truss mechanism, comprising horizontal and vertical stirrups or bars and diagonal concrete struts, in accordance with 9.4.6 and 9.4.7, except that corner joints of portal frame structures and in other appropriate applications joints may be detailed by rational analysis so that shear forces are transferred by an acceptable mechanism and so that anchorage of the flexural reinforcement within the joint is assured.

9.4.6 *Horizontal joint shear reinforcement.* The horizontal design shear force to be resisted by the horizontal joint shear reinforcement shall be

$$V_{sh} = \frac{V_{jh}}{\phi} - V_{ch} \quad \text{(Eq. 9-1)}$$

where

$$V_{ch} = 0.5\, V_{jh} \left(1 + \frac{C_j P_u}{0.4\, A_g f_c'}\right) \quad \text{(Eq. 9-2)}$$

except that in joints where the overall depth of the column, h_c, is at least two times the overall depth of the beam, h_b, V_{ch} need not be taken less than

$$V_{ch} = 0.2\, b_j h_c \sqrt{f_c'} \quad \text{(Eq. 9-3)}$$

The area of horizontal shear reinforcement shall be determined in accordance with 9.5.4.3.

9.4.7 *Vertical joint shear reinforcement.* The vertical design shear force to be resisted by the vertical joint shear reinforcement shall be

$$V_{sv} = \frac{V_{jv}}{\phi} - V_{cv} \quad \text{(Eq. 9-4)}$$

where

$$V_{cv} = \frac{A_{sc}'}{A_{sc}} V_{jv} \left(0.6 + \frac{C_j P_u}{A_g f_c'}\right) \quad \text{(Eq. 9-5)}$$

except that V_{cv} need not be taken less than

$$V_{cv} = 0.2\, b_j h_b \sqrt{f_c'} \quad \text{(Eq. 9-6)}$$

The area of vertical shear reinforcement shall be determined in accordance with 9.5.5.3 and 9.5.5.4.

9.4.8 *Confinement.* The horizontal transverse confinement reinforcement in beam-column joints shall not be less than that required by 6.4.7, with the exception of joints connecting beams at all four column faces in which case the transverse joint reinforcement may be reduced to one half of that required in 6.4.7, but in no case shall the stirrup-tie spacing in the joint core exceed ten times the diameter of the column bar or 200 mm, whichever is less.

9.5 Principles and requirements additional to 9.3 for joints designed for seismic loading

9.5.1 *General.* Special provisions are made in this Section for beam-column joints that are subjected to forces arising as a result of inelastic lateral displacements of ductile frames. Joints must be designed in such a way that the required energy dissipation occurs in potential plastic hinges of adjacent members and not in the joint core region.

9.5.2 *Design forces*

9.5.2.1 The design forces acting on a beam-column joint core shall be evaluated from the maximum stresses generated by all the members meeting at the joint in equilibrium. The forces shall be those induced when the overstrengths of the beam or beams are developed, except in cases when a column is permitted to be the weaker member. At columns of two-way frames, where beams frame into the joint from two directions, these forces need only be considered in each direction independently.

9.5.2.2 The magnitude of the horizontal shear force, V_{jh}, and the vertical shear force, V_{jv}, in the joint shall be evaluated from a rational analysis taking into account the effect of all forces acting on the joint.

9.5.3 *Design assumptions*

9.5.3.1 The design of the shear reinforcement in the joint shall be based on the effective control of a potential failure plane that extends from one edge of the joint to the diagonally opposite edge. In determining the shear strength of the joint the value of the strength reduction factor ϕ shall be 1.0 where design forces are derived from overstrength member forces.

9.5.3.2 The nominal horizontal shear stress in the joint in either principal direction, v_{jh}, shall not exceed $1.5\sqrt{f_c'}$

where

$$v_{jh} = \frac{V_{jh}}{b_j h_c} \quad \text{(Eq. 9-7)}$$

The effective joint width, b_j, shall be taken as

(a) when $b_c > b_w$
either $b_j = b_c$
or $b_j = b_w + 0.5\, h_c$, whichever is the smaller

(b) when $b_c < b_w$
either $b_j = b_w$
or $b_j = b_c + 0.5\, h_c$, whichever is the smaller.

9.5.3.3 The shear strength of a joint shall be assessed as follows:

(a) When plastic hinges could develop immediately adjacent to a joint the entire shear shall be assumed to be resisted by a truss mechanism, consisting of horizontal and vertical stirrups or bars and diagonal concrete struts, with the exception of joints where gravity load or prestressing enable transmission of shear by diagonal concrete compression forces, in which case some shear may be allocated to a concrete mechanism alone in accordance with 9.5.4.2.

(b) For the plastic hinge conditions of 9.5.3.3 (a) diagonal bars, bent across the joint in one or both directions, or other special devices, may be used if it is shown by rational analysis or tests, or both, to the satisfaction of the Engineer, that the shear forces that may be induced during large inelastic deformations of adjacent beams are adequately transferred by an acceptable mechanism and that anchorage of the flexural reinforcement across the joint is assured.