**Section 8: Compliance**

**8.1 Compliance with legal requirements**

When designing or checking the structure of a building, it is important to ensure that it complies with fundamental engineering concepts on which structural design is based. While a structure should comply with legal requirements and relevant design standards it should be noted that not every aspect of structural design will be covered in these documents. It is possible to design a structure that meets the minimum requirements set out in the standards, but which may perform poorly when subjected to critical loading conditions due to lack of consideration of basic structural concepts.

The fundamental requirement of structural design can be simply described as: every force or load that is applied to a structure must have a valid load path between the point where the load is applied and the foundation soils. This load path must satisfy the requirements of equilibrium and strain compatibility. In satisfying this basic requirement, load paths must be tracked through the entire structure, including different structural elements, such as beam-column joints and junctions between beams and structural walls.

The legal requirements relating to the engineering design of the CTV building were set out in the Christchurch City Council (CCC) Bylaw 1051. The Bylaw was adopted by the CCC using its powers to make bylaws regulating and controlling the construction of buildings under section 684(1)(22) of the Local Government Act 1974. The Bylaw came into force on 1 December 1985.

8.1.1 Background to Bylaw 105

The CCC, as with the other territorial authorities in New Zealand, made building bylaws that adopted New Zealand Standards, which were made by the Standards Institute of New Zealand.

The first Model Building Bylaw, NZSS 952, was published in 1936. NZSS 19003 was published in the mid-1960s. Up until 1970, NZSS 1900 contained a number of chapters dealing with topics such as loadings and construction materials. From 1970, separate Standards were developed from these chapters. NZS 4203:19764 dealt with general structural design and design loadings. This was superseded by NZS 4203:19845. NZS 3101P:19706, which related to concrete, was subsequently replaced by NZS 3101:19827. NZSS 1900 recognised these and other Standards as a “means of compliance” with the Bylaw. In section 2 of Volume 4 of this Report we give a fuller account of the development of these and later Standards. In this section of the Report we use the words “Standard” and “code” interchangeably. This was, and remains, a common approach.

Counsel for the CCC referred to a report8 to the CCC

from the Town Planning Committee at the time Bylaw

105 was adopted. The report said:

The Building Bylaw…had been revised to conform to the general pattern of the other bylaws but much of the text was still contained in New Zealand Standards which are often amended and are now quite expensive.

A revised Building Bylaw is attached to this report. As far as is possible it has incorporated clauses from the New Zealand Standards but the New Zealand Standards have been severely edited to remove clauses that are not particularly relevant to present building conditions.

The more recent standard bylaws have been in the form of a relatively simple bylaw with the means of compliance being contained in a separate document. The means of compliance documents, the technical documents that explain how to comply with the bylaws are not changed and are being used throughout the country.

The substance of NZSS 1900 was generally reproduced in Bylaw 105, apart from some matters that are not relevant to this Inquiry.

8.1.2 Content of Bylaw 105

**8.1.2.1 General design method – Clause 11.1.5(d)**

The content of the Bylaw was contained in the First Schedule, which comprised 12 parts. Clause 11.1.5 set out the requirements for “general structural design method”, including the principles underlying the design of a building:

11.1.5 The general structural design method

(as distinct from detailed design appropriate

to particular construction materials as required elsewhere in this bylaw) and the design loadings shall be recognised as appropriate upon achieving the following:

(a) All loads likely to be sustained during the life of the building shall be sustained with an adequate margin of safety.

(b) Deformations of the building shall not exceed acceptable levels.

(c) In events that occur occasionally, such as moderate earthquakes and severe winds, structural damage shall be avoided and other damage minimised.

(d) In events that seldom occur, such as major earthquakes and extreme winds, collapse and irreparable damage shall be avoided, and the probability of injury to or loss of life of people in and around the building shall be minimised.

The word “shall” appears a number of times in this clause. Clause 11.1.3 made it clear that it delineated a mandatory obligation:

In this bylaw the word “shall” indicates a

requirement that is to be adopted in order to comply with the bylaw.

**8.1.2.2 Compliance with Clause 11.1.5(d)**

Clause 11.1.5(d) set out clear design objectives. However, there was much discussion during the hearing about what these objectives meant in practice and how an engineer could set about achieving them.

When the CTV building was designed, granted a building permit and constructed, the Acts Interpretation Act 1924 applied to the interpretation of Bylaw 105. Under section 5(j) of the act, a bylaw is to receive:

…such fair, large and liberal construction and interpretation as will best ensure the attainment of the object of the (bylaw)… according to its true intent, meaning and spirit.

Counsel assisting submitted that the text of Clause

11.1.5(d) demonstrated a very clear purpose in that the designer was required to design the building so that collapse was avoided and the probability of injury or loss of life minimised. Clause 11.1.6 of the Bylaw provided a means by which the objectives in Clause 11.1.5 could be achieved:

11.1.6 General structural design and design loadings complying with NZS 4203 shall be approved as complying with the requirements of clause 11.1.5.

The CTV building was a concrete structure. Part 8 of the

Second Schedule to Bylaw 105 dealt with the design of concrete structures. Clause 8.4, which was entitled “Means of Compliance,” included the following:

**8.4.1 Design**

Concrete elements designed in accordance with the requirements of NZ 3101 or a recognised equivalent standard shall be deemed to comply with the requirements of this bylaw.

These references to NZS 4203 and NZS 3101 as means of compliance resemble Clause 5 of the Introduction to Bylaw 105, which said:

**Acceptance means of compliance with the provisions of this bylaw**

Proof of compliance with the specifications, standards and appendices named in the second schedule of this bylaw shall be deemed to be in the absence of proof to the contrary, sufficient evidence that the relevant degree of compliance required by this bylaw is satisfied.

The specifications, standards and appendices named in the second schedule are not part of this bylaw.

The Second Schedule of Bylaw 105 included

NZS 4203:1984, NZS 3101 Part 1:1982 and NZS 3101

Part 2:1982.

There was scope for tension between the mandatory design objectives set out in Clause 11.1.5 and the means by which they could be achieved, particularly where aspects of NZS 4203 and NZS 3101 could be interpreted as inconsistent with the objectives. This tension was the source of evidence from expert witnesses and submissions by counsel and will be discussed below.

**8.1.2.3 “Major earthquakes”**

Another issue arising from the wording of Clause

11.1.5(d) is the size of the earthquake the Clause

applied to. The clause referred to “…events that seldom occur, such as major earthquakes…”

The expression “design level earthquake” refers to an earthquake producing forces equivalent to those contemplated by the loadings standard at the time: NZS 4203:1984. However, the loadings standard did not (and still does not) nominate forces equivalent to the greatest possible forces an earthquake could generate. For example, the February earthquake produced forces on the CTV building that were greater than those specified by the loadings code applicable when the building was designed, and by the current code.

Counsel assisting submitted that a “major earthquake” as mentioned in Clause 11.1.5(d) may not be a “design level earthquake” and that, even if they were equivalent:

…neither the Bylaw nor the Codes allow the designer to design on the basis that a building is only required to withstand an earthquake at ‘design level’ but to collapse in an earthquake only marginally stronger.

It was submitted that an approach based upon just meeting a performance requirement was not compatible with maximising performance. There was an obligation to not only address the risk of collapse, but to minimise the probability that it would eventuate. The purposes in the Bylaw would not be met “if the designer seeks to draw a line beyond which collapse and death are virtually certain”.

Counsel for Dr Alan Reay, Mr Rennie QC, submitted

that neither the Bylaw nor the codes required any check of how the building might perform in an earthquake that produced stronger shaking than that required to be assumed by the Standard. In addition, they did not contemplate two or more design level or higher earthquakes in quick succession. It was accepted that the purpose of the Bylaw could not be met if the designer sought to draw a line beyond which collapse and death were virtually certain; however, Mr Rennie submitted that:

…the only quantitative means to assess whether these purposes are achieved is via design checks. Design checks already contain safety factors, so something that ‘just’ passes the Code should not be extremely vulnerable. Passing the Code means an acceptable level of design (with a risk that may be higher than significantly exceeding the Code, but an acceptable risk nonetheless).

In our view, the term “major earthquake” in the Bylaw contemplated an earthquake greater than design level. We discuss the seismic design of buildings in section 3.2 of Volume 1 of our Report. In the 1980s the design level earthquake was based on a 150-year return period. However, knowledge of typical ground motions in major earthquakes was not as great as today and the response spectra of the time overestimated ground accelerations for long period structures and underestimated values for short period structures.

It should be noted it is not practical to require that buildings be designed not to collapse under an extreme event. However, the intent of the codes was to ensure that design requirements would give protection against collapse for earthquakes with a greater magnitude than a “design level” earthquake.

Design actions and inherent factors of safety implicit in the choice of design strengths and load combinations were intended to provide a very high level of certainty that a design level earthquake could be resisted without collapse. This gave a level of protection against collapse in an earthquake that had a greater level of shaking. The margin of safety came from:

• the loads assumed in the seismic analysis using conservative combinations of loading. This resulted in design actions generally being appreciably higher than the average actions that would be induced by the assumed level of earthquake;

• the use of design material strengths and load factors, which resulted in the design strength typically being considerably smaller than the likely average strength of the members; and

• the ductility levels being selected on the basis that a critical peak displacement must be sustained through a series of load cycles (based in the 1980s on eight load reversals to the peak displacement) whereas the maximum displacement is only reached once in an earthquake.

In the 1980s the design level earthquake was intended to ensure a level of protection against collapse in an earthquake with an appreciably higher intensity of shaking than the design event. The magnitude of the higher level of earthquake was not specified. Risk was minimised for events larger than the design level by:

• the use of load and strength reduction factors;

• conservative assumptions of design loads and

forces;

• the use of lower characteristic material strengths in

design calculations;

• allowance for approximations in design calculations;

and

• conservative assessments of the ductility of

members and materials.

These matters are discussed in section 3.2 of Volume 1.

**8.1.2.4 Symmetry and ductility**

Part 11 of Bylaw 105 included some further relevant design requirements. Clause 11.2.5.2, which was

entitled “Earthquake Provisions”, included the following:

11.2.5.1 Symmetry

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.

11.2.5.2 Ductility

(a) The building as a whole, and all of its elements that resist seismic forces or movements, or that in the case of failure are a risk to life, shall be designed to possess ductility; provided that this shall not apply to small buildings having a total floor area not exceeding 140m² and having a total height not exceeding 9m.

(b) Structural systems intended to dissipate seismic energy by ductile flexural yielding shall have “adequate ductility”.

(c) “Adequate ductility” in terms of clause (b) 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.

These clauses also attracted discussion during the hearing, particularly the provisions relating to ductility. This will be referred to in the discussion below about the columns and beam-column joints.

**8.1.2.5 Obligation to comply**

As we noted in section 2.2.3.1 of this Volume,

Clause 2.2.1 of the Bylaw provided that a building could not be erected without a permit first being obtained. Clause 2.6.1 of the Bylaw required that an application for a building permit be accompanied by detailed plans and other documents of sufficient clarity to demonstrate “the provision made for full compliance with the requirements of [the] Bylaw”. Where the CCC reviewing engineer considered that the proposed building did not comply with the requirements of the Bylaw, the permit could be withheld under Clause 2.13.

Clause 2.14 provided that the permit would be issued “where the Engineer is satisfied that the drawings and specifications are in accordance with [the] Bylaw …”

Read together, the provisions discussed in section

2.2.3.1 have the consequence that a building permit should not have been issued for a building that did not comply with the Bylaw.

8.1.3 Relationship between Bylaw 105 and the codes

Counsel for Alan Reay Consultants Limited (ARCL) and Dr Reay made submissions about the Bylaw and codes, and deficiencies in the latter, apparently for the purpose of highlighting an uncomfortable relationship between the two and to illustrate the difficulty in defining mandatory legal requirements.

Mr Rennie QC submitted that NZS 4203:1984 and NZS 3101:1982 were only binding on the designers to the extent that they were specifically incorporated into the Bylaw. It was open to the designer to comply with the Bylaw in some other way. It was also submitted that the nature of engineering practice in the 1980s was such that many components of safety and loading were uncertain and that the Bylaw and standards were not a complete instruction manual. Rather, they were a starting point that would operate in tandem with the growing expertise of professional and expert engineers. Some degree of freedom was left to the designer.

It was submitted that the evidence the Royal Commission heard showed that engineers saw the application of standards as an art, informed by experience and practical knowledge. There were severe limits on the ability of the designer to achieve certainty with the design methods and computer analyses available in the 1980s. “Mere compliance” with the Bylaw was said to lead to the possibility of a compliant but unsafe design, while a “safe design” may not have been compliant with the Bylaw. Deficiencies with the codes and qualifiers such as “as far as is practicable” also introduced uncertainty.

Although the codes were a means of compliance with Bylaw 105, Clause 1.1.1.2 of NZS 4203:1984 stated that any departure was to be justified in the design calculations and application for building consent as a special study. Clause 4.2.1 of NZS 3101:1982 also set out the fundamental requirement that structures and structural members be designed to have dependable strengths at least equal to required strengths. No engineer could justifiably depart from these basic engineering requirements.

8.1.4 Council permit

The processes adopted by the CCC for the consideration of an application for a permit for the construction of a building in the 1980s are described in section 2.2 of this Volume.

The issue of whether any non-compliance should

have been identified by a CCC reviewing officer before granting a building permit will be addressed in relation to each of the items of alleged non-compliance of the CTV building discussed below.

8.1.5 Compliance with legal requirements

**8.1.5.1 Symmetry**

Clause 11.2.5.1 of Bylaw 105, which was in the same terms as Clause 3.1.1 of NZS 4203:1984, stated:

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.

Counsel assisting submitted that these clauses should be interpreted in the following way:

1. They should be regarded as an important means by which the obligations to avoid collapse and minimise the probability of injury and death (set out in Clause 11.1.5(d) of the Bylaw) were to

be achieved.

2. They set out a mandatory requirement that building elements must be located symmetrically about the centre of mass.

3. The designer could only move away from this requirement for very good reason and only after exploring ways of retaining symmetry.

4. Even when moving away from the requirement, there was still an obligation to achieve symmetry as nearly as practicable and to ensure that the overarching obligations in clause 11.1.5(d) to avoid collapse and minimise injury and death were met. This would make it even more important to ensure that the building satisfied ductility requirements and had adequate load paths.

Counsel assisting submitted that the design of the CTV building failed to comply with Clauses 11.2.5.1 and 11.1.5(d) of the Bylaw.

The commentary to Clause 3.1.1 of NZS 4203:1984 provides some insight about the purposes of the symmetry requirement in the code:

It is recognised 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.

Dr Murray Jacobs was called to give expert evidence on compliance issues by counsel assisting. It was his evidence that the combination of the north wall complex and the south shear wall was asymmetrical in the east-west direction, while the eccentricity was less in the north-south direction. He considered that there was a large separation between the centre of stiffness and the centre of mass in the east-west direction. He said that as a consequence:

…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.

In Dr Jacobs’ view, the consequences of having two unequal walls orientated in the same direction were well known at the time of the design of the CTV building. He cited a paper published in the *Bulletin of the New Zealand National Society for Earthquake Engineering* in 1980 by T Paulay and RL Williams9, which stated:

…as in all structures in seismic areas, symmetry in structural layout should be aimed at... Deliberate eccentricity should be avoided, if possible, because uneven excitations may aggravate eccentricity and this in turn may lead to excessive ductility demand in lateral load resisting elements situated far away from the centre of rotation.

Mr David Harding accepted that the centre of stiffness of the designated primary seismic resisting elements was significantly eccentric to the centre of mass in the east-west direction. However, he said it was not practicable for the stiffness of the walls to be located symmetrically because:

… the architectural requirement for the location of the walls wouldn’t have allowed a shear wall the same as the one on the north side to be located on the south.

Counsel for ARCL and Dr Reay submitted that the words “as nearly as is practicable” made this provision difficult to apply as there was no definition about what was acceptable and what was not. He submitted:

…it is impossible to define ‘as nearly as is practicable’ for all purposes. Therefore non- compliance cannot be assessed against undefined criteria…The Code did not specify a clear limit on the acceptable degree of eccentricity. There was no defined point at which acceptable becomes unacceptable.

He referred to the current loadings Standard

(NZS 1170.5:200410), in which there is still no limit on the permitted degree of eccentricity.

Mr Rennie QC also submitted that the walls were actually located symmetrically. Although the intention of the clause may have been that elements had to be similar in stiffness or strength, this was not stated in the clause. Reference was made to other buildings in Christchurch with similar levels of asymmetry (in particular, Landsborough House) and to the 1990 report11 from Holmes Consulting Group (HCG) which said, “the layout and design of the building is quite simple and straightforward”. It was submitted that this contemporaneous assessment was more reliable than a “26 year hindsight assessment”.

Dr Reay said in evidence that the walls were located symmetrically about the centre of mass, although he agreed that the centre of stiffness was eccentric to the centre of mass. He said that there was no absolute requirement for symmetry in the Code.

Dr Arthur O’Leary’s view was that Clause 3.1.1 of NZS 4203:1984 did not raise a specific compliance issue, although the issue of compliance did need to be considered under Clause 3.4.7.1, which provided quantitative guidance on the issue of asymmetry. He said that engineering judgment would be required in determining the extent to which the requirement of symmetry would apply.

Counsel assisting submitted that, even if it is accepted that there is room for an exercise of judgement in applying the words, “as nearly as practicable” to the requirement for symmetry, neither Dr Reay nor Mr Harding could point to any impracticability sufficient to justify the exclusion. Mr Harding said it was an

architectural issue. However, Mr Alun Wilkie gave evidence that there was no architectural impediment to a wall being located anywhere along the south of the building.

Although Clause 11.2.5.1 is expressed in a mandatory way, it is qualified by the words “as nearly as practicable”. This implies that there would be some designs in which symmetry could not be achieved. Clause 3.4.7.1 of NZS 4203:1984 set out provisions for the analysis of eccentric and irregular buildings. It is worth noting that relatively few buildings are symmetrical apart from high rise structures (taller than the CTV building), which have grown in number since the time the CTV building was designed and built.

In our view, Clause 11.2.5.1 is effectively an exhortation to a designer to be cautious and conservative in the design of eccentric and irregular buildings. It was not intended to prevent the design of such structures. As an example, an engineer may exercise caution by making additional provisions for safety in the performance in design. This could take the form of exceeding minimum reinforcement in areas where ductility was required or providing for more redundancy in the selection of alternative load paths. If a building was less regular, it would also lead to a designer treating the analysis of the building with more caution, which is the principle underlying Clause 3.4.7.1 of NZS 4203:1984.

It follows from what we have said that the symmetry provisions sit uncomfortably in both the Bylaw and the code. In practice, their effect was not mandatory and it would have been preferable if they were located in the commentary to NZS 4203:1984.

In our view, the CTV building design complied with Clause 11.2.5.1. However, the clause should have raised a warning that a conservative approach was required in the analysis and design. It is clear such an approach was not taken.

**8.1.5.1.1 Whether the CCC reviewing officer should have identified asymmetry**

Counsel assisting submitted that the CCC reviewing engineer should have identified asymmetry in the design of the CTV building. Dr O’Leary said that, in his opinion, a CCC reviewing engineer would note the imbalance between the north wall complex and the south wall and then look at the drawings and calculations to see whether it was adequately accounted for in the design. However, he said that a lack of balance between walls was not uncommon at the time and would not have “raised alarm bells” for him. Mr John O’Loughlin gave evidence that he

considered that the building was not particularly asymmetric. He said a CCC reviewing engineer could have reasonably formed the view that the building was reasonably symmetrical about the centre of gravity.

The CCC submitted that the concept of symmetry is not susceptible to quantitative assessment and that issues of engineering judgement arise. As such, determining compliance was said to be problematic and it is not clear how the CCC would enforce such a requirement. The CCC did however accept Mr Peter Nichols’ comments that where a building is asymmetric, a particularly careful approach to deflection limits is appropriate.

The CCC reviewing engineer should have identified a lack of symmetry. However, given that Clause 11.2.5.1 was in the nature of an exhortation rather than an enforceable obligation, this would not have resulted in a permit being refused. Instead, it should have resulted in the reviewing engineer satisfying himself that the issue had been considered and allowed for in the analysis. However, apart from looking at the calculations to see

if it had been considered, he would have had no way to check it further as the CCC did not have computers or software available. To make a detailed check would have required input from the University of Canterbury and a considerable time delay. In our view, all that could be expected was for the reviewing engineer to make sure this issue was considered. In this case the calculations showed that it was.

**8.1.5.2 Diaphragm Connections**

**8.1.5.2.1 Adequacy of connections at lines D**

**and D–E**

The north wall complex of the CTV building was designed to be one of the two lateral-resisting elements that provided seismic resistance to the building. One of the key design issues addressed during the hearing was the way in which the floor slabs (also referred to as “the diaphragms”) were connected to the north wall complex (see Figure 109). Counsel assisting submitted that the design of the original connections between the floors and the north wall complex at Lines D and D–E

did not comply with the Bylaw and codes.

C D E

North wall complex

Toilets

Landing

Stair

Lifts

Drag Bars

**Figure 109: The north wall complex. The approximate locations of the drag bars, attached in 1991, are marked in red**

The floors were connected to the north wall complex at various locations. A number of experts considered that the connections at lines D and D–E were inadequate. The Hyland/Smith12 report pointed out that no specific reinforcing steel was specified in this region. Dr Hyland and Mr Smith characterised this as an “omission”. As discussed in section 2.4, Mr John Hare of HCG described it as “a vital area of non-compliance” with design codes current at that time. He said in his report to a potential purchaser in January 1990:

Connections to the walls at the north face of the building are tenuous... in the event of an earthquake, the building would effectively separate from the shear walls well before the shear walls themselves reach their full design strength.

Mr Geoffrey Banks, who designed the drag bars retrofit, expressed the view when giving evidence that this was an area of non-compliance. Dr O’Leary said that the connection did not comply with NZS 4203:1984. The CCC also accepted that this was the case. Dr Reay said that the diaphragm connection at lines D and D–E was a “possible” area of non-compliance.

This aspect of the building was the subject of retrofit work in 1991. The circumstances of that work are addressed in section 2.4 of this Volume. The work resulted in the installation of drag bars at lines D and D–E on levels 4, 5 and 6.

When counsel assisting put it to Mr Harding that the connections were non-compliant, Mr Harding said:

No, I can’t accept that at the moment. Not on the basis of the information I have. I’ve, it could definitely have been improved, I accept that and that’s what they’ve done by connecting to the walls on line D, and D and E, but I don’t accept that you have to have a connection on D, and D and E in order to support the load specified in the code.

However, in his closing submissions, his counsel said that Mr Harding accepted the submissions of counsel assisting in relation to engineering matters.

At the time of the design, Mr Harding made some calculations of the loadings required for the floor connections in an east-west direction. He used forces from either the equivalent static method or the modal response spectrum method he had used. His notes, which were made available to the Royal Commission, did not include any calculation of the loadings applicable to the connections in a north-south direction. He said he thought there were additional pages that were not part of the set the Royal Commission received from Dr Reay.

There was nothing in Bylaw 105 relating to the design of connections. Clause 3.4.6.3 of NZS 4203:1984 addressed this issue in the following way:

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.

Clause 3.4.9 was the “parts or portions” section of NZS 4203:1984. Mr O’Loughlin was asked whether the effect of Clause 3.4.6.3 was that the loadings for diaphragm connections must be calculated using Clause 3.4.9, and he agreed that it was. Clause 10.5.6.1 of NZS 3101:1982 also provided a basis to use the loadings derived from Clause 3.4.9 of NZS 4203:1984 as illustrated below:

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.

Counsel assisting submitted that, if capacity design was applied, the loads which the floor connections would have been required to bear would have been greater than the loads required to cause yielding in the plastic hinge regions of the walls. Mr Harding agreed with this. He accepted that he did not use capacity design to calculate the applicable loadings. Neither did Mr Banks.

However, both Clause 3.4.6.3 of NZS 4203:1984 and Clause 10.5.6.1 of NZS 3101:1982 permitted the use of the parts or portions provisions of NZS 4203:1984. Dr Reay made this point when questioned by Commissioner Fenwick about whether the use of the parts or portions provisions was compatible with capacity design, which may have required the use of loadings greater than these prescribed by those provisions. Dr Reay accepted that capacity design required that the connections between the floor slabs and the wall should be capable of developing the maximum possible strength of the wall and agreed that, in hindsight, it did not make much sense to use loadings prescribed by the parts or portions provisions.

The first part of Clause 10.5.6.1 relates to capacity design, that is, the forces required to sustain the over- strength actions in the lateral load resisting elements must be sustained. The second part relates to forces required to tie parts of a building to the basic structure. Requiring the design force to be the smaller of these two is illogical as it does not require the capacity design forces to be sustained in all cases. This means that ductile behaviour would not be ensured in some situations. We do not consider that this can have been the intent of the Bylaw. The requirement should be to satisfy the greater of the two requirements.

However, the effect of Clause 3.4.6.3 of NZS 4203:1984 and Clause 10.5.6.1 of NZS 3101:1982 was that the diaphragm connection forces could be taken as those specified by Clause 10.5.6.1. Mr Banks used this clause to calculate loads for the connection of the drag bars in 1991. Mr Banks’ calculation of the loadings in the east-west direction to the north wall complex was 724 kilonewtons. Mr Harding’s figure was 300 kilonewtons.

Counsel assisting submitted that Mr Harding did not apply Clause 3.4.9 and that, as a result, he undercalculated the required connection forces, which then did not have sufficient capacity to meet the minimum required strengths.

Mr Harding’s figure of 300 kilonewtons was derived by using the equivalent static method, which should not have been used. In the Royal Commission’s opinion, the design did not comply with either the parts and portions provisions or the capacity design provision.

It was also submitted that the underestimation applied in relation to connections to the south shear wall as well. However, we consider that the connection details that were used for the south wall would have been adequate for parts or portions design forces.

**8.1.5.2.2 Non-compliance in the east-west direction**

Counsel assisting submitted that the building was non- compliant in the east-west direction not only at the time of permit but following the retrofit in 1991. Reference was made to questions from Commissioner Fenwick in which Mr Banks agreed that east-west shear would have been transmitted from the floor at line 4 to the wall at line 5 by the walls on lines C and C–D and the floor between these walls. He indicated that the design shear force calculated from Clause 3.4.9 of NZS 4203:1984 of approximately 700 kilonewtons would have acted on line 5 and generated a shear that was virtually constant over the distance between line 4 and line 5, which

was about four and a half metres. This would have generated a bending moment of the order of 3,000 kilonewton metres.

Mr Banks accepted that, by using the forces derived from Clause 3.4.9 and then considering the equilibrium of forces just south of line 4, the required design strength was a shear of approximately 700 kilonewton metres and a moment of at least 3000 kilonewton metres. Mr Banks said that he had calculated the flexural capacity at that point as being in the order of 1800 kilonewton metres. The design strength (1800 kilonewton metres) was therefore less than the required strength (at least 3000 kilonewton metres).

Mr Banks agreed when giving evidence that the floor was overloaded. He said he did not consider this issue when designing the retrofit: he had directed his attention only to the issue raised by HCG.

In addition to these issues, Dr O’Leary gave evidence that the floor connections were non-compliant for east- west seismic actions for reasons set out in calculations provided to the Royal Commission. The calculations considered shear resistance from the floor between the walls C and C–D and the wall on line 5. Mr Banks did not agree with Dr O’Leary’s calculations.

In our view, the connections between the floors and the north wall complex were non-compliant in the east- west direction, although not for the reasons given by Dr O’Leary. Mr Banks calculated the design lateral east- west force at level 6 acting on the wall on line 5 was 740 kilonewtons. This value was calculated from the parts and portions provisions in NZS 4203:1984. This value is reasonably consistent with the force of 600 kilonewtons that we calculated at line 4 (we considered line 4 to be close to the critical section). The difference between the 600 and 740 kilonewton forces arises from the mass of the building between lines 4 and 5. The flexural actions in the floor associated with the shear force were not considered in the original design, as noted by Mr Banks. His estimate of the design strength required for the east-west actions due to flexure are of the same order as those we calculated. Both Professors Nigel Priestley and John Mander acknowledged that the floors adjacent to line 4 were over-stressed in flexure but they did not indicate that they had made numerical calculations to check this.

The drag bars would have been ineffective in terms of resisting shear and flexure due to east-west seismic forces and this connection would have remained non- compliant after the drag bars were fitted.

**8.1.5.2.3 Whether the CCC reviewing officer should have identified inadequate connections**

Counsel assisting submitted that a reviewing officer should have identified inadequate connections between the floor connections and the north wall complex; the apparent absence of some calculations relating to the diaphragm connections, and an error in which Mr Harding dropped a “0” on page S57 of the calculations. We refer again below to this error in the calculations.

Counsel for the CCC submitted that it is clear from Mr Graeme Tapper’s letter dated 27 August 1986 that he identified an issue relating to the floor connections to the north wall complex, but that it is not clear exactly what this was. It was noted that some changes were made to the drawings in response to that letter.

Mr Nichols, who was a structural checking engineer

with the CCC between 1978 and 1984, said that he was astonished by the weak appearance of the connections between the floors and the north wall complex, which relied on nothing more than a single layer of 664 steel mesh and D12 slab tie starter bars at 400 centres. He said, “it jumps out of the page at you when you have some experience looking at structural drawings”.

Mr John O’Loughlin gave evidence that the connection between the floor diaphragms and the lateral load resisting system was not a significant check item for a reviewing engineer during the early 1980s, probably because at this time shear walls were generally designed to be of adequate length in relation to total floor area. He said that engineers became more focused on this issue following the 1989 San Francisco earthquake.

Mr O’Loughlin referred to reinforcing steel connecting the floors to the north wall complex. He said that he would not have considered the amount of reinforcing set out in Mr Harding’s calculations to be adequate. Although this would have required a critical review of the calculations, he said the issue should have been identified by a reviewing engineer. He also noted the error on page S57 of the calculations in which Mr Harding used a figure of 30,000 newtons for shear stress instead of 300,000 newtons. He said this had the effect of underestimating the required reinforcing. When questioned about this in evidence, Mr Harding accepted that the figure of 30,000 was wrong. However he thought this was “picked up at the time”.

Mr O’Loughlin said in evidence that a line-by-line analysis of the calculations would have been required to identify this error and it would not have been readily

apparent to a reviewing engineer. Counsel for the CCC submitted it was telling that Mr O’Loughlin was the only engineer who noticed this error, and that this reinforces the view that a reviewing officer could not have been expected to notice it. It was submitted that the absence of some calculations for the floor connections also fell into this category.

However, the inadequate connections were also identified from a review of the plans, after the building was constructed but prior to its collapse, by Mr Hare of Holmes Consulting Group, as discussed in section 2.4. We also heard evidence from Mr Murray Mitchell, a senior structural engineer with Opus International Consultants (Opus), who carried out a desktop review of the structure in 1998 or 1999 when Opus was considering leasing part of the CTV building. He was working with the original structural drawings, and was not aware of any structural modifications after the building was constructed. Although he only spent a matter of hours reviewing the drawings, Mr Mitchell formed the view that the connections between the floors and the north wall complex were not as strong as they should have been. Opus did not consider the building further.

In our view, the CCC reviewing engineer should have identified the inadequacy of the connections in the north-south direction. This problem stands out. We are satisfied that Mr Tapper identified that this connection was non-compliant, but a building permit was issued. As discussed in section 2.2.4 of this Volume, the building permit should not have been issued. It would have been more difficult to identify the east-west omission. Mr Hare did not identify the shortfall in the east-west direction and, although Professors Priestley and Mander both indicated that the area looked dubious, they did not do any calculations to justify their suspicions.

**8.1.5.3 Non-seismic detailing of columns and beam-column joints**

NZS 3101:1982 contained different methods for detailing columns and beam-column joints. Where gravity actions dominated, the columns and beam- column joints could be designed to the standard provisions of the code. Where seismic actions exceeded set limits, the columns and beam-column

joints were required to meet the minimum provisions set out in the “additional requirements for seismic loading”.

The columns and beam-column joints of the CTV building were designed using the non-seismic provisions in NZS 3101:1982. These provided the minimum level of ductility permissible under the Code.

Counsel assisting submitted that the columns should have been detailed using the “seismic” provisions set out in section 6.5. Reference was made to the evidence of Professor Priestley that, had the seismic provisions been used, the column displacement capacities would have been sufficient to resist the forces predicted by the non-linear time history analysis of the February earthquake.

Counsel assisting submitted that the legal obligations to avoid collapse and minimise the probability of injury and death were served by the use of the seismic provisions. Conversely the use of non-seismic provisions did not serve these obligations. Neither Dr Reay nor Mr Harding accepted that the objectives of the Bylaw required the use of the seismic provisions of NZS 3101:1982.

Mr Harding gave evidence that:

The beams were designed to be continuous beams and as such were designed to be moment resisting only between adjacent beams for gravity loading. The columns were not intended to be part of a moment resisting frame, and the ends of the columns were designed as pin joints. Consequently the beam-column joints were not designed to carry any bending moment from the columns, and any contribution which these columns may make toward the building lateral stiffness was not relied upon.

Professor Priestley gave evidence that, in his opinion, the columns were not pin ended. Mr Ashley Smith held the same view. When asked in cross-examination whether he agreed that it would not have been the effect of the design that they were pin ended, Mr Harding said, “I agree that it wasn’t detailed significantly to be pin ended, that the vertical reinforcement in the column did continue through the joint”. We agree that the columns were not pin ended. It was a design error to assume that they were.

Counsel assisting put forward four grounds as to why the seismic provisions of NZS 3101:1982 should have been applied and that any one of these grounds, if accepted, would be sufficient to justify this conclusion:

1. Failure of the columns was a risk to life.

2. Capacity design required that they be designed in this way.

3. The columns were not “secondary elements”.

4. If the columns were secondary elements, the prescribed drift limits were exceeded.

**8.1.5.3.1 Failure of columns was a risk to life**

For ease of reference, we set out again the provisions of Clause 11.2.5.2 of Bylaw 105:

(a) The building as a whole and all of its elements that resist seismic forces or movements, or that in the case of failure are a risk to life, shall be designed to possess ductility; provided that this shall not apply to small buildings having a total floor area not exceeding 140m2 and having a total height not exceeding 9m.

(b) Structural systems intended to dissipate seismic energy by ductile yielding shall have “adequate ductility”.

(c) “Adequate ductility” in terms of clause (b) 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.

(emphasis added)

This clause is in the same terms as Clause 3.2.1 of

NZS 4203:1984, except in relation to “small buildings”, which are not relevant for our purposes.

Counsel assisting submitted that the failure of columns in the CTV building posed a risk to life. Dr Jacobs and Professor Mander both agreed with this.

Counsel assisting argued that the CTV building was intended to dissipate seismic energy by ductile yielding and was therefore required to have “adequate ductility”. As a result, all of the primary elements resisting seismic forces, and not just the north wall complex and south shear wall, were to be detailed using the seismic provisions of NZS 3101:1982. The definition of “primary elements” in NZS 4203:1984 included beams and columns.

Counsel for the CCC noted that “ductility” was defined in Clause 1.1.3.1 of NZS 4203:1984 as:

…the ability of the building or member to undergo repeated and reversing inelastic deflections beyond the point of first yield while maintaining a substantial proportion of its initial maximum load carrying capacity.

Counsel for the CCC argued that the references

to ductility in Bylaw 105 and NZS 4203:1984 were qualitative rather than quantitative and did not provide any guidance as to the magnitude of the deflections that the building or members were required to be

designed for. The CCC did not accept that the reference to “special requirements for ductile detailing in the appropriate material Code” in Clause 11.2.5.2 (c) of the Bylaw was intended to refer to the seismic provisions

of NZS 3101:1982. This was said to be a reference to the whole of the ductility provisions of the relevant code rather than part of it.

Counsel for Dr Reay submitted that, as “ductility” was not defined, it raised a question of how it was to be measured. It was submitted that the columns in the CTV building did possess some level of ductility by using the ratio of ultimate displacement to the elastic displacement as the measure, albeit not as much as what they could have had if the seismic provisions had been used. It was also suggested that the failure limit used in the Hyland/Smith report was probably estimated too low; adopting a higher failure strain would demonstrate an increased level of ductility.

When giving evidence, Dr Reay said that compliance with NZS 4203:1984 was identified in the Bylaw as a means of compliance and:

…the way the code is written and the way we follow it, it should happen that the columns aren’t the critical element in terms of the risk to life and you could design those columns for ductility but the end result could be that there is a greater tendency for the cover concrete to fall off when they’re subject to yielding than if they were built as they had been drawn…

In our view, the potential failure of the columns was not a risk to life (within the meaning of Clause 11.2.5.2(a) of the Bylaw) if it could be shown that they had adequate ductility as designed to meet the required deflection limits. The seismic provisions of NZS 3101 were not required if the deflection limits were met.

There was considerable confusion in the Standards over the requirement for ductility. Clause 3.2.1 required that “all elements that resist seismic forces, or in the case of failure are a risk to life, shall be designed to possess ductility…” Clauses 3.2.2 and 3.2.3 stated that structural systems intended to dissipate seismic energy by ductile yielding shall have “adequate ductility”, which meant “detailed in accordance with special requirements for ductile detailing in the appropriate material Code”.

It should be noted that Clause 3.5.14.3(a) in

NZS 3101:1982 did not require ductile detailing if the member could deflect without inelastic deformation for a distance of *v*∆13. However, any member designed to NZS 3101 had some level of ductility; hence it can be argued that the columns satisfied the qualitative requirement for ductility. We note that this approach cannot be used now as all columns are required to be confined to a level of at least limited ductility.

The option was removed in NZS 3101:199514, and the columns designed under that code were required to have at least limited ductile confinement reinforcement.

**8.1.5.3.2 Capacity design required that columns be designed using seismic provisions**

Counsel assisting submitted that the CTV building was required to be designed using capacity design and that Dr Reay and Mr Harding should therefore have considered the behaviour of the structure as a whole in an earthquake and identified and designed for an acceptable ductile failure mechanism.

Capacity design was described in section 3.2.6 of

Volume 1 of our Report.

Counsel assisting referred to Clause 3.3.2.2 of

NZS 4203:1984, stating:

Buildings designed for flexural ductile yielding or for yielding in diagonal braces, shall be the subject of capacity design. In the capacity design of earthquake resistant structures, energy dissipating elements or mechanisms are chosen and suitably designed and detailed, and all other structural elements are then provided with sufficient reserve strength capacity to ensure that the chosen energy dissipating mechanisms are maintained throughout the deformations that may occur.

Counsel also cited Clause 3.5.1.3 of NZS 3101:1982, stating:

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.

It was submitted that the effect of these clauses is that the designer of the CTV building was required to identify the location of potential plastic hinges and design the remaining structural elements to be stronger than those zones.

Mr Harding said that capacity design applied to the shear walls, which were designed as the lateral load resisting elements, but not to the beam-column frame, which was not designed to be a ductile frame. Dr Reay said that capacity design applied only to the walls “and not to the gravity frames if they are based on elastic design”.

In cross-examination, Professor Mander agreed that the designers of the CTV building should have identified the ends of columns as potential plastic hinge regions and that capacity design required the use of the transverse reinforcement set out in NZS 3101 for those regions.

Counsel assisting submitted that Professor Mander’s evidence should be accepted and that the seismic loading provisions set out in Clauses 6.5.4.3 and

9.5.6.1 should have been used. As they were not, it was submitted that this amounted to a failure to comply with the Code and with Clause 11.1.5(d) of the Bylaw.

We note that capacity design requirements could be satisfied if it could be shown that the ductile failure mechanism could be maintained when over-strength actions were sustained in the chosen plastic regions. This did not necessarily require plastic hinges to develop in the columns, and an assumption that it did would be contrary to Clause 3.5.14.

We agree with Dr Reay and Mr Harding that capacity design applied to the CTV building and was used for

the walls. Capacity design did not require the use of the seismic provisions of NZS 3101:1982 in the columns, provided the columns could sustain the design inter- storey drift without sustaining inelastic deformation.

**8.1.5.3.3 The columns should not have been treated as secondary elements**

Dr Reay and Mr Harding considered that the columns in the CTV building were “secondary elements,” as

a result of which they could be detailed using the non-seismic provisions of NZS 3101:1982 if certain criteria were met. Counsel assisting submitted that the columns could not be regarded as secondary elements.

Neither primary nor secondary elements were defined in Bylaw 105. However, NZS 4203:1984 included definitions of both in Clause 1.1.3.1, as below:

ELEMENTS include primary and secondary elements. PRIMARY ELEMENTS means elements forming part

of the basic load resisting structure, such as beams, columns, diaphragms, or shear walls necessary for the building’s survival when subjected to the specified loadings.

SECONDARY ELEMENTS means elements such

as partition walls, panels, or veneers not necessary for survival of the building as a whole but subject to stresses due to loadings applied directly to them or to stresses induced by the deformations of the primary elements.

That clause also contained a definition of “horizontal force resisting system”:

Clause 3.5.14 of NZS 3101:1982 was entitled

“Secondary structural elements” and began:

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...

There is an inconsistency between the definitions of “elements” in NZS 4203:1984 and NZS 3101:1982 in that columns are classified as primary elements in the former, while they could theoretically be classified as secondary elements in the latter. Counsel assisting submitted that, in the event of an inconsistency between the two codes, NZS 4203:1984 should prevail.

The Commentary clause C1.1 of NZS 4203:1984 stated:

Pending the revision of various other New Zealand standards, this standard should be regarded as the ‘master document’ with other standards, where appropriate, subject to it.

The Foreword to NZS 4203:1984 also noted:

This edition incorporates Amendment No 3. Among the Amendment’s more significant contributions is an upgrading of the section dealing with earthquake provisions. It also irons out any parts of the Loadings Code that happened to conflict with the various materials Codes.

Rather than merely issuing an amendment slip, it was decided the extent of Amendment No 3 warranted a reprint of NZS 4203.

The Foreword to NZS 3101:1982 stated that section 3 (which set out General Design Requirements) had a particular importance because it established the relationship between the 1982 Code and the 1984

Code. It also stated:

It should be noted that some provisions in this Code are based on proposed amendments to NZS 4203 which at the time of publication are being finalised.

Clause C3.5 of the Commentary to NZS 3101:1982 stated:

HORIZONTAL FORCE RESISTING SYSTEM means that part of the structural system to which the horizontal forces prescribed by this code of practice are assigned.

The earthquake loading, principles of seismic design, recommended analysis procedures and several other aspects of earthquake structural engineering are documented in detail in NZS 4203. Therefore the commentary of NZS 4203 should also be consulted when applying this Code.

It was submitted that these references show that NZS 4203:1984 should prevail over NZS 3101:1982 where there is inconsistency. However, counsel for Dr Reay submitted that there is no inconsistency or ambiguity. NZS 4203 referred to columns as primary elements, and the columns were primary elements for gravity loadings. However, Mr Rennie QC argued that they could be classified as secondary elements with respect to the lateral load resisting system while constituting primary elements for gravity loads. Reference was made to Clause C3.5.14.1 of the commentary to NZS 3101:1982, which stated that secondary elements included “such primary gravity- load resisting elements as frames which are in parallel with stiff shear walls”.

We consider that, in the 1980s, most engineers would not have assumed that columns would provide lateral strength for seismic actions. On this basis, columns could be assumed to be secondary elements in the terms defined in NZS 3101:1982.

There is no doubt that there was confusion between NZS 4203:1984 and NZS 3101:1982. From a legal point of view NZS 4203:1984 may have prevailed. However NZS 4203:1984 did not on its own give sufficient guidance on the practicalities of design. In our view, it was not unreasonable to classify columns that were not intended to function as part of the seismic load resisting system as secondary elements under NZS 3101:1982.

Counsel assisting also referred to a distinction in Clause 11.1.5 of Bylaw 105 between “the general structural design method” and “detailed design appropriate to particular construction materials as required elsewhere in this bylaw”. It was submitted that the design of concrete elements fell into the latter category and compliance with NZS 3101 (including the provisions relating to secondary elements) was not a means of satisfying the general structural design methods and requirements set out in Clause 11.1.5 of the Bylaw. As the Bylaw prevailed over the codes, it was not permissible to classify columns as secondary elements when to do so would violate the objectives of the Bylaw.

However, counsel for Dr Reay pointed out that

NZS 4203:1984 states under the heading “General Design Principles” that “Design shall be in accordance with the appropriate materials code subject to the principles of design set out below”, and that there were no specific requirements in NZS 4203:1984 for gravity elements acting in conjunction with ductile shear walls.

In the 1980s, engineers would have believed that compliance with NZS 3101:1982 satisfied the general design requirements of the Bylaw. There is some confusion in the Bylaw and codes and it was not unreasonable for engineers to take this approach given that NZS 4203:1984 did not on its own give sufficient guidance about the design of a reinforced concrete building.

Computer analysis at that time was limited by capacity constraints and it was standard practice to ignore the lateral strength associated with structural elements that were flexible compared to the stiffer members. Hence it was frequently assumed in medium rise buildings that lateral seismic forces were resisted by walls or relatively stiff perimeter frames. This approach is not as common now. However, counting the resistance of the more flexible elements has resulted in some cases in the building being less robust than it would have been if designed neglecting the lateral contribution of the flexible elements.

Counsel assisting submitted that there were two limbs to the definition of secondary elements in Clause 3.5.14. They could be elements that did not form part of the primary seismic force resisting system or elements that were *assumed* not to form part and were therefore not necessary for the survival of the building as a whole.

In relation to the first of these two limbs, counsel assisting referred to Professor Mander’s evidence that, when the building was exposed to design level shaking, the frames, consisting of beams, columns and beam-column joints would all have been called upon to resist earthquake loads. Reference was also made to Dr O’Leary’s acceptance that beams, columns, diaphragms and shear walls must have been included in the definition of primary elements because they are the parts of the structure that would be exposed to earthquake loads in an earthquake.

However, counsel for the CCC, Mr Reid, submitted that the beams and columns in the CTV building did not form part of the primary seismic force resisting system of the building. He argued that the seismic resisting system in a “shear wall protected gravity load system” could only be the shear walls. While the columns may be subject to, and may resist, seismic forces, they were not part of the primary seismic force resisting system.

We accept this submission. The columns of the CTV building did not form part of the primary seismic force resisting system.

In relation to the second limb of the definition of secondary elements in Clause 3.5.14, counsel assisting submitted that, although the clause is very poorly worded, when interpreted in light of both text and purpose any such assumption must be consistent with the element not being necessary for the survival of the building as a whole. The clause does not allow columns to be treated as secondary elements simply because the designer mistakenly assumed that they were. This interpretation of the clause was said to be supported by Bylaw 105 and its controlling requirement of life safety and collapse avoidance.

Professor Mander described Clause 3.5.14.1 as a “loophole”. In response to questioning from Commission Chairperson Justice Cooper, he also said that he did not agree with the approach of using Clause 3.5.14 as a loophole. Counsel assisting referred to his evidence that the columns, beam-column joints, north wall complex and south shear wall would all have been necessary for the survival of the building as a whole.

However, counsel for the CCC submitted that the phrase “and are therefore not necessary for the survival of the building as a whole under seismically induced lateral loading” is not an additional requirement but a consequence of the design approach to the building, which is that the shear walls were the primary seismic force resisting system. Reference was made to the commentary to Clause 3.5.14 of NZS 3101:1982, which said:

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.

Dr O’Leary gave evidence in relation to the meaning of “stiff” as used in this clause that “the widely held interpretation at the time would have been whether the frame would provide a significant contribution to the lateral load resistance of the structure”. Dr Jacobs considered that the north wall structure should be regarded as slender in the north-south direction due to the notch at the base of the north shear core wall.

Dr Jacobs was the only expert engineer who gave evidence that the columns were not secondary elements. It was implicit in the evidence of Dr Hyland,

Mr Smith, Mr Rob Jury, Dr O’Leary, Mr John O’Loughlin,

Mr John Henry and Mr Hare that they thought it was permissible to classify a column as a secondary element. Dr Reay and Mr Harding expressed similar views. Counsel for Dr Reay pointed out in closing that this expert evidence can be taken as illustrating how the Bylaw was in fact interpreted and applied at the relevant time. In addition, the secondary elements clauses in NZS 3101:1982 remain virtually unchanged in both NZS 3101:1995 and NZS 3101:200615, although, as of 1995, far more stringent levels of confinement were required for non-seismic columns.

Counsel assisting submitted that compliance with the Bylaw is a question of law, opinions expressed by these experts are not definitive and the fact that engineers appear to have adopted a certain approach does not establish that it was lawful.

Counsel for Dr Reay submitted that this approach disregards the requirement of section 5(j) of the Acts Interpretation Act 1924; invites the Royal Commission to make a finding of law (when the Inquiry is one of fact) and to apply it retrospectively; and invites the Commission to disregard expert evidence and to adopt a legal interpretation where the issue arises in respect of the meaning of a code of engineering practice, not a statute.

Counsel for CCC agreed that expert evidence as to the correct interpretation of the codes is not determinative of the question; however, such evidence should be persuasive given that the experts worked with the codes day to day.

In our view, survival of the building as a whole depended on the ability of the columns to support gravity loads when lateral deflections were applied and not on their contribution to lateral resistance. The columns were secondary elements and should have been designed to sustain their axial load capacity with all of the lateral resistance provided by the walls. Whether they met this requirement is discussed in the next section.

**8.1.5.3.4 If the columns were secondary elements, drift limits were exceeded and seismic provisions should have been used**

*8.1.5.3.4.1 Introduction*

Counsel assisting submitted that, even if the columns were properly treated as secondary elements, the criteria applicable to them required the use of the seismic provisions of NZS 3101:1982.

The provisions relating to the detailing of secondary elements were set out in Clause 3.5.14 of NZS 3101:1982.

As the columns in the CTV building were not detailed for separation, they were classified as Group 2 elements under this clause. Clause 3.5.14.3 provided:

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 v∆, 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 *v*∆.

(c) Inertia loadings *Ep* 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 v∆, 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, *v*∆, of the primary elements, as specified in NZS 4203.

(f) Where elastic theory is applied in accordance with (e) for deformation corresponding to 0.5 *v*∆ or larger, the design and detailing requirements of Section 14 may be applied, but otherwise

the additional seismic requirements of other sections shall apply.

There were three options available in relation to the amount of reinforcing steel (and therefore ductility) that was to be used in a secondary element. The seismic provisions provided for the most reinforcing steel and the highest level of ductility. The non-seismic provisions specified the least steel, although still with some level of ductility. Section 14 included provisions for an intermediate position, described in NZS 3101:1982 as “limited ductility”.

Clause 3.5.14.3 set out the criteria for determining which of these three possibilities should be adopted. Clause (a) referred to “additional seismic requirements”, namely the seismic provisions that provided the

highest level of ductility. The clause said that those provisions need not be used when design loadings were derived from imposed deformations (*v*∆) and “the assumptions of elastic behaviour”. This is a reference to the question of whether the building element, in this

case the column, would remain in its elastic state when

earthquake loads imposed upon it caused it to deform to a certain extent, namely *v*∆.

For practical purposes a column may be assumed to remain elastic if the longitudinal reinforcement does not yield in tension and the strain in the extreme fibre of concrete does not exceed 0.003.

The question of whether the columns in the CTV building would move from an elastic to a plastic state when subjected to specified earthquake loads was an important one. Once a column becomes plastic, the level of strength that it maintains is determined by how ductile it is. The more ductility it has, the longer it can retain its strength under increasing displacements due to earthquake actions.

The effect of Clause 3.5.14.3(a) was that, if the columns of the CTV building remained in an elastic state when they were subjected to the inter-storey drift of *v*∆, the additional seismic requirements need not be applied. The calculation of *v*∆ was determined by a series of clauses which can be found in NZS 4203. The coefficient “*v*” was defined in Clause 3.8 of NZS 4203:1976 and it was replaced by *K/SM* in the later edition of the Standard, NZS 4203:1984. ∆ was the inter-storey drift found in an equivalent static or modal response spectrum analysis.

This modification factor is found in Clause 3.8.1.1 of

NZS 4203:1984 which said:

3.8.1.1 Computed deformations shall be those resulting from the application of the horizontal actions specified in section 3.4 or 3.5 and multiplied by the factor K/SM appropriate to the structural type and material, where K=2 for the method of section 3.4 and K=2.2 for the method of section 3.5.

3.8.1.2 Computed deformations shall be calculated neglecting foundation rotations.

The “methods” of sections 3.4 and 3.5 is a reference to the means by which horizontal seismic loads were calculated under NZS 4203:1984. Section 3.4 set out “Equivalent static force analysis”, which was a hand calculation, and section 3.5 described “Dynamic analysis”, which refers to the modal response spectrum method. The circumstances in which each of these was required to be used are discussed below.

“∆” was defined in Clause 3.1 of NZS 3101:1982 as:

Displacement or deformation (angular or lineal)

of the primary elements due to the loading E

“E” was defined in the same clause as:

Earthquake loads as defined by NZS 4203.

The effect of these clauses was that the designer of a building would calculate the extent of the deformations of the primary elements when exposed to specified earthquake loads (∆). This would be found using either the equivalent static method or a modal response spectrum method of dynamic analysis (such as using the computer program ETABS). ETABS is a computer program that can be used to carry out both equivalent static and modal response spectrum analyses. The deformations, which for the purposes of this calculation would be taken as the inter-storey drift, would then

be multiplied by the modification factor “*K/SM*” from

Clause 3.8.1.

*8.1.5.3.4.2 Detailing requirements if the behaviour of the columns was plastic at below v*∆

Clause 3.5.14.3(b) of NZS 3101:1982 provided that the additional seismic requirements of the code were to be met when plastic behaviour was assumed at levels of deformation below v∆.

Counsel assisting referred to Clause 3.5.3.2 of

NZS 3101 which provided:

the requirements of section 14 (limited ductility) would have applied. He said it was unclear what a design engineer would do when only the drifts at the top floors triggered this requirement. He said it would appear prudent to detail all floors to the requirement, but it was unclear what the standard practice was at the time or what councils would require. He described these requirements as vague and also said there was a lack of definition of the method to be used to establish drifts at the elastic limit.

*8.1.5.3.4.3 The method of calculation of deformations*

In response to questions from Commissioner Fenwick, Mr Harding gave evidence that he was not aware of Clause 3.5.14 of NZS 3101:1982. He did carry out calculations that produced information about the building deformation when earthquake loads were imposed. However he did not carry out any calculation to determine whether the columns would remain elastic when they underwent those deformations.

The decision about whether to carry out a three- dimensional modal analysis to determine building deformations rather than using the calculations of the equivalent static method was to be determined by reference to Clause 3.4.7 of NZS 4203:1984. This clause provided:

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.

Counsel assisting submitted that, as capacity design applied, this clause required the designer to assume the columns would be plastic rather than elastic, in which case the seismic provisions would apply. We do not accept this submission. The clause can be interpreted as referring to ductile moment resisting frames with or without shear walls, which was a hybrid structure. The CTV building did not have a ductile moment resisting frame so this clause does not apply. To apply it would not be consistent with Clause 3.5.14.3(a) of NZS 3101:1982, under which additional seismic requirements need not be met if the columns remained elastic up to *v*∆.

Mr William T. Holmes said in his report16 to the Royal Commission that it was difficult to apply Clause 3.5.14 to a frame that is not at all designed

for lateral deformations. Although certain column drift demands calculated by Dr Hyland exceeded elastic limits, the effect of Clause 3.5.14.3(f) was that only

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

(a) …

(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.

The commentary to the clause provided some explanation:

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.

Mr Harding believed that the building was an irregular structure more than four storeys high and that a three- dimensional modal analysis was required. For this reason, he arranged for a modal response spectrum analysis using ETABS to be conducted at the University of Canterbury.

Dr Jacobs gave evidence that, in his view, the building had a high degree of eccentricity in the east-west direction and the floor plan was irregular; hence Clause 3.4.7.1(c) applied. On the other hand, Dr O’Leary considered that the relevant clause for compliance purposes was Clause 3.4.7.1(b). Similarly, Mr Latham was of the opinion that the building was of moderate eccentricity only and a static analysis could be used exclusively to determine the design forces and displacements for compliance.

There was some confusion in Clause 3.4.7.1 and the commentary about what was irregular and eccentric. The commentary defined structures of moderate eccentricity but it failed to say what was required if the structure had an eccentricity greater than moderate. A three-dimensional modal analysis was recommended in the clause, but not required, for reasonably regular structures. Such an analysis generally had the advantage of reducing the required design strength and inter-storey drifts when compared to an equivalent static analysis.

We accept that the CTV building had an eccentricity very much greater than moderate but it could it be analysed by the equivalent static method in terms of the Code.

*8.1.5.3.4.4 The Hyland/Smith ERSA*

Dr Hyland carried out an elastic response spectra analysis (ERSA). An elastic response spectra analysis is the same as a modal response spectrum analysis such as could be carried out using the program ETABS. The Hyland/Smith elastic response spectra analysis is described in Appendix E of the Hyland/Smith report. Using the deformations derived from this and applying the modification factor in Clause 3.8.1.1 of NZS 4203:1984, Dr Hyland and Mr Smith identified the design inter- storey drifts in different storeys for three columns.

They selected columns on line 1 close to line C, at D-2 and at F-2 (see Figure 110) and calculated the design inter-storey drift as specified in NZS 4203:1984.

Level 3-6

A B C D E F

5 North wall complex

Landing

Toilets

Stair

Lifts

N

4

3

West block wall level 3 only

2

South shear wall

1

**Figure 110: The indicator columns identified by Dr Hyland and Mr Smith were located at the intersection of gridlines D**

**and 2, gridlines F and 2 and to the left of the intersection of gridlines C and 1**

Dr Hyland and Mr Smith concluded in their report

that columns on gridline 1, close to gridline C on levels

2–6 and at gridlines F–2 on levels 5-6 did not remain elastic at the design inter-storey drift. For this reason, they considered that the seismic provisions of NZS 3101:1982 should have been used for the design and detailing of columns of the CTV building. This finding was endorsed by the Department of Building and Housing’s Expert Panel and by Mr Jury, who gave evidence on behalf of the Expert Panel.

Dr O’Leary gave evidence that it was not appropriate for methods of analysis not available except as research tools, or excluded by standards of the day, to be used to make assessments as to whether analysis, design and detailing of the CTV building complied with the standards of the day. He provided two examples from the Hyland/Smith report, namely the use of “Cumbia” software from a paper published in 2007 for the displacement compatibility analysis, and the inclusion of the effect of flexible foundations in the elastic response spectra analysis.

We agree with Dr O’Leary that compliance must be checked in terms of the 1980s and using the design criteria appropriate in the 1980s.

Dr O’Leary expressed the view that Dr Hyland did not include the most critical columns in the centre of the building as sample columns in his analysis. Dr O’Leary concluded that the columns located at gridlines B-2, B-3, B-4, C-2, C-3, C-4 and C-5 and the columns at A-B-1 and B/C-1 complied with the requirements of Clause 3.5.14.3(a). This was based on the inter-storey drifts recorded in pages S15 and S16 of Mr Harding’s calculations. We note that these appear to have been underestimated (see section 6.2). He considered that the columns on line F did not meet these requirements and should have been designed for seismic loading.

*8.1.5.3.4.5 The direction to experts to confer*

On 18 June 2012, the Royal Commission directed that relevant experts confer and:

…endeavour to reach agreement on the input data to be used to conduct an elastic response spectra analysis of the response of the CTV building to determine whether the design of the building was consistent with the provisions of NZS 3101:1982 and NZS 4203:1984.

If the elastic response spectra analysis carried out

by Dr Hyland did not meet this purpose, they were to carry out a further elastic response spectra analysis. Professor Athol Carr was appointed as facilitator. All of the experts except Mr Douglas Latham agreed that the elastic response spectra analysis prepared by Dr Hyland and Mr Smith was sufficient. Mr Latham considered that a further elastic response spectra analysis should be carried out.

We do not accept that the Hyland/Smith response spectrum analyses are valid for assessing compliance in terms of structural practices in the 1980s for the two reasons identified by Dr O’Leary:

1. Soil springs were used in the model and in the

1980s it was standard practice to assume the soil was rigid for vertical loading for seismic actions. Dr Davidson confirmed this was the case in the 1980s.

2. The member stiffness characteristics assumed for the columns were based on software and research findings that were not available in the 1980s.

For the reasons noted above, Compusoft Engineering

Limited was commissioned to carry out a further set of equivalent static and modal response spectrum analyses as specified in NZS 4203:1984 in which the soil was assumed to:

• be rigid for seismic actions;

• have the spring stiffness values assumed in the

Hyland/Smith report; and

• have a set of soil springs representing soil stiffness based on predicted or long-term (settlement) deformation.

We consider compliance should be based on the results of analysis with rigid soils. The other two sets of analyses were requested so that comparisons could be made to the Hyland/Smith analyses and analyses carried out by Mr Latham of ARCL, which are briefly described below.

*8.1.5.3.4.6 Mr Latham’s ERSA*

In his ERSA and equivalent static analyses, Mr Latham used linear springs to represent the soil. The properties of these springs were derived by Mr Ian McCahon of Geotech Consulting Limited from a soils report that he had earlier prepared, which was commissioned for the design of the CTV building. The soil stiffness values in this report were given for the purpose of assessing long-term settlement. It is normally accepted that the soil is much stiffer for dynamic loading, such as occurs in an earthquake, than for long-term loading. Mr Latham based his analyses on the long-term values, on the basis of the following comments from Mr McCahon:

Tonkin and Taylor (T + T) have reported on the

site in their letter titled *CTV Building Geotechnical Advice* dated 11 July 2011, to StructureSmith Ltd. They include a section on subgrade reaction for the dynamic analysis. I am not an expert in this field and do not wish to comment, other than making the comment that with the relatively loose cohesionless soils in Christchurch, seismic shaking appears to have generated high pore water pressures in soils even if there has not been full liquefaction. This must reduce the shear strength of the soil, and the reasoning that subgrade reaction values for dynamic analysis should be expected to be much greater than for static analysis may not be entirely applicable.

We do not accept that the use of such soil springs, which were intended for the assessment of long-term settlement, is appropriate for the seismic analysis of a building. An acceptably competent engineer might use such values as one extreme case, but he or she would need to repeat the analysis using soil stiffness springs that would be typical of the stiffness characteristics of the soil for seismic loading if liquefaction did not occur. The design would be made to ensure that the building could perform adequately for both cases.

By using the “soft” soil springs, Mr Latham greatly increased the fundamental vibration periods for the building, which decreased the seismic design forces. He then removed the component of storey drift from the inter-storey drift values on the basis of Clause 3.8.1.2 which stated:

3.8.1.2 Computed deformations shall be calculated neglecting foundation rotation.

Effectively this involved assuming soil conditions that would have allowed for foundation rotation and then at the end of the process neglecting that rotation. In our view that is not a legitimate approach to the application of Clause 3.8.1.2. We consider that, properly interpreted, that clause required the starting assumption of rigid foundation soils.

None of the other expert witnesses accepted

Mr Latham’s analysis as a valid interpretation of the design requirements of NZS 4203:1984. We note that both Mr Harding and Mr Henry assumed the ground was rigid when they made their response spectrum analyses for the CTV and Landsborough House buildings respectively.

Having obtained the design inter-storey drifts due to seismic forces, Mr Latham’s next step was to apply these displacements to selected parts of the gravity load frames, which consisted of the columns and beams to establish if the columns had sufficient elastic deformation capacity to sustain the (design, *v*∆) inter- storey drifts. If this condition could be satisfied under all the seismic loading cases the columns would not need to be designed to meet the additional requirements for seismic loading.

In the analyses of parts of the gravity load frames, Mr Latham made a number of assumptions:

• his analysis neglected the difference in the lengths

of the beam spans on line 2;

• the section properties of the beams were derived neglecting the stiffness contributed by the floor slabs, which resulted in an underestimate of the beam stiffness;

• no allowance was made for the stiffening effect in

the beam-column joint zones; and

• he used the Branson equation, which is given as equation 4 in Clause 4.4.1.3 of NZS 3101:1982, to assess the effective stiffness of both the beams and columns to check an ultimate limit state criterion.

This is an incorrect use of the equation. Clause 4.4.1.3 is in the section of the Standard that relates to the serviceability limit state. Furthermore, the clause is titled “Computation of deflection (a) one way construction (non-prestressed)”. The term “one way” refers to beams and slabs and does not include columns. The use of the equation for a column results in an underestimate of its stiffness.

The use of the Branson equation is also questionable for the beams. The commentary clause C3.5.5 recommends that for:

…the estimation of deflections for the purposes of determining periods of vibration or satisfying the requirements of structural separation and the limitations of inter-storey drifts, will be more realistic if an allowance for the effects of cracking on the stiffness of members is made. Typically the moment of inertia of a beam section may be based on 50% of the moment of inertia of the gross concrete area, whereas for columns carrying significant axial compression, 100% of the corresponding moment of inertia may be assumed.

As indicated above the stiffness of both the beams and the columns have been underestimated in Mr Latham’s analyses, which leads to an overestimate of the inter-storey drift that can be sustained by the elastic response of the columns.

**8.1.5.3.5 Whether the columns remained elastic at the design inter-storey drifts, v**∆

*8.1.5.3.5.1 Analyses*

Clause 3.5.14.3 of NZS 3101:1982 provided that additional seismic requirements of the code need not be satisfied for secondary elements where inter-storey drifts of *v*∆ could be sustained on the basis of an elastic response. The Royal Commission has carried out analyses to determine whether the columns in the CTV building would have remained elastic at an inter- storey drift of *v*∆.

*8.1.5.3.5.2 Assumptions made in analyses*

We have used the results of the response spectrum analysis carried out by Compusoft and described in their report entitled “1986 Code Compliance ETABS Analysis Report” dated August 201217 to assess whether the columns could meet this requirement. Our calculations are based on the Compusoft response spectrum analyses in which the soil was assumed to be rigid, as this was the practice in the 1980s when the CTV building was designed. It complies with NZS 4203:1984.

Clause C3.5.5 in the commentary to NZS 3101:1982 recommended that for columns the effective stiffness was taken as equal to the section property based on the gross section if there was significant axial compression. For beams it was recommended that the stiffness was based on 0.5 of the gross section.

In this analysis, where the axial load ratio, based on tributary areas, exceeds 0.2 *Ag f ’c* , the gross section properties have been used and when there is no axial load 50 per cent of the gross section properties have been used. For intermediate values of axial load, interpolation was used. The criteria on limiting deformations would have been set with the recommended stiffness values in mind in NZS 3101:1982.

The flexural capacity of the circular columns is based on a spreadsheet in which the column was split up into 100 strips. The concrete stress in each strip was calculated on the basis of plane sections remaining plane and on the stress strain relationship for unconfined concrete developed by Mander et al.18 in 1988. This stress-strain relationship satisfies the requirements of Clause 6.3.1.6 of NZS 3101:1982 and it gives similar ultimate strengths to the rectangular stress block given in Clause 6.3.1.7. However, Mander et al.’s stress strain relationship has the advantage of enabling response over the full range of concrete strains to be used. For the rectangular columns the flexural capacity was found using the standard rectangular stress block given in

NZS 3101:1982.

Where the axial loads had been reported by Mr Latham of ARCL they have been used in our analyses. Where appropriate values were not quoted by Mr Latham they were assessed on the basis of self-weight of beams, columns and block work at 6kN/m2 and the tributary floor areas supported by the Hi-Bond tray flooring at 4.55kPa for dead load with imposed dead load.

The analyses were made for the columns on line 2, line F and line 1. The analyses were carried out by moment distribution with the assumption that points of inflection would form at the mid-height of each storey. Some allowance was made for non-prismatic members where these were used.

The stiffness of the beams was, as far as is practical and reasonable, based on the recommendations in NZS 3101:1982. For the main beams the section properties assumed that a flange width of twice the thickness of the floor slab would act with the beam on each side of internal beams and on one side for perimeter beams. For the short beams the additional flange width was ignored. No allowance was made for additional stiffening in beam-column joint zones, and the columns have been assumed to behave elastically up to the load where the ultimate strength is reached. Consequently in terms of assessing compliance the calculations for the predicted inter-storey deflections that could be sustained before yielding of the columns is initiated is conservative.

*8.1.5.3.5.3 Results of analyses*

Results from the analyses are:

1. The columns on lines 2 and 3 complied with the requirements in Clause 3.5.14 that the inter-storey drift could be sustained without ductile detailing for seismic actions.

2. The columns on line F did not require ductile detailing in the first and second storeys but above those levels the ductile detailing provisions for seismic actions were required.

3. The columns in the first storey on line 1 did not require the ductility detailing provisions for seismic actions but the columns in the higher storeys did require these provisions to be satisfied.

4. No calculations were made for the columns in line A.

However, it is likely that columns in the first, second and third storeys would not have complied. In this case compliance would have depended on both the vertical and lateral restraint the concrete block walls would have provided to the beams.

**8.1.5.4 Beam-column joints**

**8.1.5.4.1 Compliance**

Counsel assisting submitted that, if the Royal Commission concluded that the columns of the CTV building should have been designed using the seismic provisions of NZS 3101:1982, it follows that the beam-column joints should also have complied with the seismic provisions set out in Clause 9.5.6. It was said that the effect of Clause 9.5.6.1 was that the horizontal transverse reinforcement in the beam-column joints was required to be no less than that in the columns. Dr Reay accepted this in evidence.

We agree that if the seismic provisions were required

to be used for columns, the same level of confinement reinforcement, as a minimum, should have been used in the beam-column joint zones.

Counsel assisting referred to Clauses 9.4.2, 9.4.5 and

9.4.6 of NZS 3101:1982, which related to horizontal joint shear reinforcement. In addition, Clause 9.4.8 specified that spiral reinforcing in the beam-column joints was to be spaced at no more than 200mm. Reference was also made to the Hyland/Smith report, which said:

The beam-column joints had no specific spiral or hoop reinforcing detailed to provide confinement or shear strength, and to hold the beams into the joint

This level of detailing is indicative of the joints having been considered to be required to satisfy only the non-seismic design requirements of the concrete structures standard NZS 3101:1982.

The R6 @ 250mm centres column spiral reinforcement would have been difficult to achieve in practice. As an integral part of the columns, the joints would also have been required to be designed using the additional design requirements of NZS 3101:1982.

Counsel assisting submitted that transverse reinforcement of R6 @ 250mm was insufficient to meet these requirements.

Mr Harding said that, for non-seismic loadings, there was no shear force in the beam-column joint. He did not believe that Clauses 9.4.2, 9.4.5 and 9.4.6 were relevant to the design.

Mr Harding accepted that the transverse reinforcement in the beam-column joints did not comply with Clause 9.4.8. Dr Reay said that this was a possible area of non-compliance.

Counsel for Dr Reay acknowledged in closing that Clause 9.4.8 was not satisfied. However, he submitted that both NZS 3101:1982 and NZS 4203:1984 allowed for testing to be used as an acceptable means of demonstrating compliance and “this is what is required for the beam-column joint, which, due to its arrangement is difficult to analyse”.

The CCC accepted that the requirements of Clauses

9.4.2, 9.4.5 and 9.4.6 were not met. Mr O’Loughlin and Dr O’Leary agreed that the CCC reviewing officer should have identified insufficient spiral reinforcement in the beam-column joints.

Clause 9.4.1 of NZS 3101:1982 required connection zones to be designed to meet the criteria for seismic design if load reversal occurred under any seismic load combinations. The commentary clause C9.3.1 made it clear that reversal occurred if the sign of the structural actions changed when seismic actions were added to gravity load actions.

The Royal Commission has analysed the columns on lines F and 2 for possible reversal of actions under any specified seismic load combination. The corresponding values for lines A and 1 can be deduced from the previous analyses. The critical load case is 0.9D plus E, where D is for dead load and E is for earthquake actions. The analysis was based on the same assumptions as used for the columns.

The beam-column joints should have been designed using the additional requirements for seismic loading in NZS 3101:1982 where reversal of actions occurred in one or more of the seismic load cases.

**Line 2**

• Reversal of actions occurred in all the beam-column

joints at levels 3, 4, 5 and 6.

• The beam-column joints at level 2 were marginal.

**Line F**

• Reversal occurred at all of the beam-column joints

on this line.

**Line 1**

• Reversal occurred on all of the beam-column joints

on this line.

**Line A**

• All the joints on line A at levels 2 and 3 were marginal and whether they were critical or not depends on the probable stiffening effect of the concrete block walls.

Where the seismic provisions were required to be applied due to reversal of actions:

• a considerable quantity of ties would need to be

added to the joint zones;

• no laps of beam bars were permitted in the

joint zones; and

• limits were placed on the diameter of bars passing

through the joint zones.

We note that longitudinal beams bars could be terminated by a 90° hook placed as near as possible to the far side of the joint zone from where the bar entered the column, but they could not be terminated in the mid-regions of the beam-column joint zones.

**8.1.5.4.2 Should non-compliance of columns and beam-column joints have been identified by a CCC reviewing engineer?**

Counsel assisting submitted that the CCC reviewing engineer should have identified:

• the inadequacy of non-seismic columns and beam- column joints to meet the requirement of Bylaw 105 and the treatment of the columns as secondary elements (which counsel assisting submitted was erroneous);

• the fact that the columns and beam-column joints

were a risk to life in the event of failure;

• the absence of calculations relating to the determination of v∆ and whether the columns would be elastic at *v*∆; and

• the fact that the building was prone to torsion and the dangers resulting from this including excessive drift levels.

Dr O’Leary expressed the opinion that a CCC reviewing engineer is likely to have looked at the overall design and noted that it was a “shear wall structure”. The reviewing engineer would know that shear wall structures are relatively stiff and therefore probably fall into the category of a structure covered by Clause 3.5.14.3(a) of NZS 3101. He said:

…the conclusion flowing from this would have been that the gravity load columns (i.e. all those in the CTV building) did not need to comply with the ‘additional seismic requirements of the Code…’ On this basis the reviewing engineer could in my view have been justified in assuming the columns complied.

He said this assessment would have been justified in Christchurch, which was an area of “only moderate seismicity”.

Dr O’Leary agreed in cross-examination that a CCC reviewing officer should give close consideration to the design of the beams, columns, diaphragms and shear walls, “within the limits of what he’s able to do”. He also agreed that the consequences of failure of the columns would probably be injury and death to people in and around the building, and that these consequences should have been clear to a CCC reviewing officer. He agreed that a CCC officer reviewing Mr Harding’s calculations could have determined that he had done no calculation of whether the columns would be elastic at *v*∆.

However, Dr O’Leary said that he did not think an experienced reviewing engineer should have identified any non-compliance because:

…the environment at the time…would have been, this is a shear wall structure and there are certain things I don’t need to consider for a shear wall structure and with that environment I think it was a legitimate position to take at the time.

He also referred to the limited time available to a reviewing engineer to assess the design. However, when questioned by Justice Cooper, Dr O’Leary accepted that, if the reviewing engineer did not have sufficient time to carry out a thorough check of the calculations, a design certificate should have been requested from the designer.

Mr O’Loughlin said in evidence that it would have been:

…completely impracticable for a reviewing engineer to carry out the kind of review necessary in order to make fine judgments about the application of NZS 4203 and NZS 3101 to the design of concrete columns.

He noted that a number of the experts who gave evidence used computer-based mathematical modelling, which would not have been readily available to the CCC reviewing engineers at the time the permit was granted. When asked about whether a reviewing engineer should have identified that the columns on line F were not compliant, Mr O’Loughlin said that these columns would not have been seen as “very special when compared with any other line”.

However, in cross-examination he agreed that a reviewing engineer should have identified the non- compliance with Clause 9.4.8 of NZS 3010:1982. We agree with Mr O’Loughlin’s evidence.

Counsel for the CCC referred to the evidence of Mr Hare that a computer analysis would be required to establish the drifts that may be imposed on the gravity structure of the building, and to Mr Nichols’ evidence that:

…at the time the CTV building was designed, it was accepted that where adequate shear walls were included to provide the required lateral restraint to the structure, the columns could be designed for gravity loads only, with the proviso that the shear wall disposition was sufficiently symmetrical to ensure an equitable distribution of lateral loadings between them.

We consider it is difficult to fault the reviewing engineer’s failure to check and identify the non-compliance of the columns and beam-column joints. A major problem here was that a modal response spectrum analysis had been carried out using ETABS and the CCC would have had no practical way of checking this. All the reviewing engineer could reasonably do was to satisfy himself that the issue had been addressed.

**8.1.5.5 Shear reinforcing of the columns**

Counsel assisting submitted that the design of the columns did not comply with NZS 3101:1982 in that, first, it required a minimum area of shear reinforcement of columns (Clause 7.3.4.3) and secondly, it specified spacing limits for shear reinforcement in columns (Clause 7.3.5.4). Reference was made to the Hyland/ Smith report, which stated that spiral reinforcing of R6 @ 90mm centres approximately or R10 @ 150mm centres, with the same steel properties as those specified, would have been required and that the spiral reinforcement of R6 @ 250mm centres was insufficient to meet these requirements. Dr Jacobs agreed that the building did not comply in this respect.

Mr Harding did not accept that these aspects of the design were non-compliant. He also said that the columns were designed to be pin ended, with no contribution to the horizontal shear capacity of the building. As such, shear reinforcement was not considered to be necessary. In our view this assumption cannot be justified.

Dr Reay said:

…shear reinforcing is only required if the certain conditions of the code aren’t met…so it’s a function of the design of the columns as to whether … that requirement is required or not.

He said he did not believe there was a breach of

the code in relation to “most of the columns”. He said that some columns “may have” breached the code in this respect, but could not give a definitive answer “because it depends on the basis on which you do that analysis”. He said he did not have the expertise “to determine which is the right answer for this”.

In his closing submissions, counsel for Dr Reay submitted:

The code provided cases where the minimum reinforcement was not required. Pursuant to clause 7.3.4.1 of NZS 3101 if the shear demand was less than half the concrete shear strength, the minimum requirements did not need to be met.

The columns satisfied this requirement, depending on the assumptions made during the analysis. Further to this, clause 7.3.4.2 of NZS 3101:1982 allowed the minimum shear reinforcement to be waived if it could be shown by test that the ultimate flexural and shear strength could be developed when the shear reinforcement is omitted.

We do not accept this submission. “Test” means building a significant number of members, testing them to destruction and showing there is a sufficient margin of strength above the maximum design action that may be required. This process was not carried out for the CTV building.

The Royal Commission carried out analyses that were described in section 8.1.5.3.5. We analysed columns on lines 3 and F for the shear forces sustained when the design level inter-storey drift was applied in the first to fifth storey in the building. Our calculations show that the shear force resisted by the columns in the third, fourth and fifth storeys exceeded half of the shear resistance provided by concrete and were marginal in the second storey. Where the 50 per cent limit was exceeded, Clause 7.3.4.1 from NZS 3101:1982 required nominal shear reinforcement to be used. This provision

would require a pitch of the spiral mode from the 6mm reinforcement to be equal to or less than 110mm.

Our conclusion on the non-compliance of the 6mm bar spiral with a 250mm pitch with the shear force design requirements in Clause 7.3.4.1 is supported by the findings in the Hyland/Smith report.

However, we do not consider that this was a material issue in relation to the collapse of the building.

Mr John O’Loughlin gave evidence that a CCC

reviewing officer would not normally “enter into a

debate about the design options chosen for the building on these fine matters of interpretation”. However, in our view, a reviewing engineer should have identified this.

**8.1.5.6 Anchorage of spirals on columns**

Clause 5.3.29.3 of NZS 3101:1982 required anchorage of spirals. In response to questions from counsel assisting, Mr Smith said that he saw no indication in the drawings of any anchorage.

In his closing submissions, counsel for Dr Reay pointed out that the structural specification for the building required all reinforcing steel to comply with the requirements of NZS 3109:198019, which gave detailing requirements for anchorage including a hook detail. Dr Reay also produced a photograph of the remains of a column in which anchorage had been provided.

In our view, the design was compliant with the Code in this respect. Anchorage was provided for in the specification, which referred to NZS 3109:1980. Although one photograph does not prove or disprove whether all of the columns had anchorage, it lends weight to a conclusion that anchorage was in place.

We do not think it reasonable to expect a reviewing engineer to identify this as an issue.

**8.1.5.7 Adequacy of the R6 @ 250mm**

**spirals in the regions of the cranked splices in the columns**

Counsel assisting referred to a region in the columns in which splices were to be cranked, and submitted that spirals of R6 @ 250mm were insufficient to meet the requirement of Clause 5.3.27.1 of NZS 3101:1982 that ties or spirals were to be placed no more than 150mm from the point of bend.

Mr Harding said in evidence that “on face value it would appear correct that that may not comply”. However, Dr Reay said that, as the spiral was at 250mm pitch, the line of the spiral would have been within 150mm of the change in angle of the bar.

Counsel for Dr Reay submitted in closing that the greatest distance that a bend could possibly be from a spiral would be 125mm, that being when the bend is exactly halfway between the two spiral ties 250mm apart, and that the 125mm is less than the required maximum of 150mm. For this reason, the specified detail was said to be in compliance with NZS 3101:1982.

In our view, the spiral ties at the cracked splices complied with spacing requirement of NZS 3101:1982. However, they would only be adequate in terms of Clause 5.3.27.1 if the bar was stressed to less than

100MPa.

We do not think it reasonable to expect a CCC

reviewing engineer to have identified this issue.

**8.1.5.8 Diaphragms**

Dr Jacobs gave evidence that the floors of the CTV building acted as large in-plane ties and struts connecting all the various parts together in an earthquake. He described the floor system as metal deck formwork with a cast in situ 200mm thick slab poured with reinforcing principally consisting of 664 mesh. He said that the 664 mesh did not satisfy Clauses 10.5.6.2 and 5.3.32 of NZS 3101:1982, which required the diaphragm to be reinforced in both directions with not less than minimum reinforcement required for two-way slabs as well as shrinkage and temperature requirements. He said that, in one direction the metal deck provided some reinforcement while in the other it was a series of discrete units jointed together by friction. He noted that the slab design was not covered by the concrete code at that time and the typical procedure was to refer to manufacturers’ design charts to select appropriate span and thickness, including top slab reinforcement at the supports.

Dr Jacobs said that the Hi-Bond literature current in

1985 indicated that 664 mesh was appropriate for a

200mm deep single span slab, but this contradicted code requirements. He said that the concrete code at the time did not address the design of Hi-Bond slabs.

Dr Reay agreed that “at face value” 664 mesh did not meet the requirements of Clauses 10.5.6 and 5.3.32 of NZS 3101:1982, but said:

…if you actually allow for the effect of all the laps that are put in as a result of using mesh I think it could, because it comes close to meeting it, I think it would then meet the code requirement.

Mr Harding said that, while mesh would not be used in a floor today, he believed the mesh met the requirements of the time.

Counsel assisting also referred to Clause 3.4.6.3

NZS 4203:1984, which required floors to be designed using the loadings set out in Clause 3.4.9, and submitted that the loadings in Clause 3.4.9 were not used for the floors or floor connections.

In his closing submissions, counsel for Dr Reay said that the Hi-Bond manufacturer’s product literature applicable at the time recommended the use of 664 mesh for slabs 151–200mm thick. It was accepted that the slab reinforcement was marginally less than the code specified minimum if the contribution from the Hi-Bond decking is ignored, but that, allowing for the Hi-Bond decking and areas where the mesh was lapped, the minimum reinforcement levels specified in NZS 3101:1982 were met. As already noted, Mr Harding’s counsel said in closing that he accepted all of the submissions made by counsel assisting in relation to engineering matters.

In our view, this is a minor issue given the use of a

metal tray. We also note the technical literature required

664 mesh so we do not consider this to be a design fault.

It appears that this issue may have been identified by Mr Tapper in his letter dated 27 August 1986. However, as we do not consider it to be an area of non-compliance, it would not have provided a basis to refuse a permit.

**8.1.5.9 Spandrel panel separation**

The Hyland/Smith report said:

A nominal gap of 20mm was specified between the ends of adjacent precast concrete spandrel panels on lines 1, 4 and F. However, the drawings didn’t specify a minimum clearance gap to the columns, or that it was required as a seismic separation. This allowed it to be interpreted as an allowance for construction tolerance only.

Mr Harding said that there is no evidence that adequate separations between the columns and the spandrels were not provided. According to the Hyland/Smith report, a minimum gap of 7mm would have been required. However, the specified gap was 10mm and the most likely construction gap would have been closer to 16mm on both sides of the columns.

Counsel for Dr Reay and ARCL submitted in closing that the drawings in fact provide for a 10mm gap and that this clearance was sufficient to allow for seismic drifts. However, Dr Reay said in evidence that he thought, “it could have been specified better than it was”.

The gap was fairly clearly fixed by the dimensions of

the precast units and it was clearly labelled. In our view, it did not need to be more clearly identified given the very limited forces that contact could induce. A gap of 10mm per side would have resulted if construction toelerance did not compromise the opening.

**8.1.5.10 Request for submissions on draft report contents**

Material now forming part of sections 8.1.5.3.5,

8.1.5.4.1 and 8.1.5.5 of our Report was sent in draft form to Buddle Findlay, acting for ARCL and Dr Reay, to Simpson Grierson acting for the CCC, and to Saunders & Co acting for Mr Harding. We also forwarded to them a full set of our calculations on compliance of the columns and beam-column joints. There was no response on behalf of Mr Harding. Simpson Grierson conferred with Dr O’Leary and their letter of 6 November did not raise any issue about the

Royal Commission’s approach and calculations. Buddle Findlay, in a letter dated 6 November 2012, recorded their clients’ disagreement with the Royal Commission’s calculations for a number of reasons that were set out

in the letter.

We summarised the analysis made by Mr Latham in section in 8.1.5.3.4.6. Buddle Findlay submitted that analysis was valid and that it showed that the columns and beam-column joints complied with the design Standards NZS 4203:1984 and NZS 3101:1982.

We noted in section 8.1.5.3.4.6 that the ARCL analysis is based on a number of assumptions that are erroneous. These include:

• allowing for the flexibility of the soil in determining the design forces but not allowing for the component of inter-storey drift associated with the soil deformation, which we consider is not a rational or legitimate approach;

• neglecting the stiffening of the beams due to the

slab acting as flanges; and

• using a design equation (equation 4 in clause

4.4.1.3 of NZS 3101:1982 for beams in the serviceability limit state) to calculate the stiffness of a column subjected to axial load for an ultimate load condition.

We note that it is incorrect to use this equation, which was developed for the purpose of assessing deflection of beams and slabs and is not appropriate for members resisting axial loads. Using this expression for a member subjected to axial load gives an incorrect stiffness. Branson, who developed this equation, gave a similar expression which gives the effective stiffness of a section. This equation used the same terms as the first equation but the power of (M*a* / M*er*) was changed. To develop it so that it can be used for members subjected to axial loads it is necessary to redefine the variables M*a* and M*cr*.. It is also desirable to allow for the influence of long term behaviour of concrete (creep and shrinkage) on the short term properties. Section stiffness values found at short intervals along the member can then be used in structural analyses. However, the use of either of the equations for member stiffness or section stiffness is appropriate for assessment of serviceability actions, and it is not appropriate for ultimate limit state requirements where a maximum limiting deflection must not be exceeded.

We reiterate that the use of a serviceability equation for an ultimate limit state condition is inappropriate. Design criteria for the ultimate limit state need to be met with a high level of certainty. For example, member strengths are based on lower characteristic material strengths to give the ideal (nominal) strength and the design value is further reduced by multiplying by a strength reduction factor. A similar level of certainty in calculating the stiffness of the beams and columns is required for calculating the ultimate limit state drift capacity of the columns. This is not achieved in using serviceability criteria, which are based on average characteristics.

We reject all the conclusions of the ARCL analysis because it is based on a number of incorrect assumptions as set out above and in section 8.1.5.3.4.6.

8.1.6 Summary of aspects of the design not compliant with legal requirements

We have considered a number of areas of alleged breach of legal requirements and our conclusions are summarised below.

**8.1.6.1 Connections between the floor slabs and the north wall complex**

The effect of Clause 3.4.6.3 of NZS 4203:1984 and Clause 10.5.6.1 of NZS 3101:1982 was that the minimum loadings required for diaphragm connections were those specified by Clause 3.4.9 of NZS 4203:1984.

Original calculations for the connection of the floors to the north wall complex for seismic actions in the north-south direction were not found. Mr Hare of HCG and Mr Banks of ARCL found the connection between the floors and the north wall complex to be inadequate for the design actions specified in NZS 4203:1984. Our own calculations led to the same conclusion. In the Royal Commission’s opinion, the design did not comply with Clause 3.4.9 of NZS 4203:1984 for seismic forces in the north-south direction as designed in 1986. The addition of the drag bars in 1991 remedied the non- compliance in the north-south direction, though the brittle nature of the drag bar connections to the floors reduced their effectiveness.

In our view, the connections between the floors and the north wall complex were also non-compliant for seismic forces in the east-west direction. Mr Harding carried out calculations for connection forces in the east-west direction. These were based on equivalent static forces, which were approximately half of the minimum connection forces derived from NZS 4203:1984. Furthermore, in carrying out the design Mr Harding failed to allow for the in plane bending action associated with the connecting shear force. Mr Banks confirmed the need to allow for the in plane bending moment and both Professors Priestley and Mander indicated that the area looked dubious, though they did not do any calculations to justify their suspicions.

**8.1.6.2 Columns**

In our view, survival of the building as a whole depended on the ability of the columns to support gravity loads when lateral deflections were applied and not on their contribution to lateral resistance. The columns were secondary elements and should have been able to act as props with the lateral resistance provided by the walls. The columns were designed on the basis that they were pin ended and that they need not be detailed to comply with the additional requirements for seismic loading. To comply with this requirement it needed to be shown that the columns could sustain the design inter-storey drift and still remain elastic under the action of the gravity loading and the bending moments and shear forces induced by the lateral displacement. No such calculations were undertaken. From our calculations we have concluded that:

• the columns on lines 2 and 3 complied with the requirements that the inter-storey drift could be sustained without ductile detailing for seismic actions;

• the columns on line F did not require ductile detailing in the first and second storeys but above those levels the ductile detailing provisions for seismic actions were required;

• the columns in the first storey on line 1 did not require the ductility detailing provisions for seismic actions but the columns in the higher storeys did require these provisions to be satisfied; and

• no calculations were made for the columns in line A. However, it is likely that columns in the first, second and third storeys would not have complied. In this case compliance would have depended on both the vertical and lateral restraint the concrete block walls would have provided to the beams.

**8.1.6.3 Beam-column joints – compliance with seismic provisions of NZS 3101:1982**

In our view, the beam-column joints should have been designed for the additional requirements for seismic design in NZS 3101:1982. Clauses 9.3 and 9.4 require the seismic provisions to be satisfied where reversal of actions occurred in one or more of the seismic load cases. We carried out analyses of the beam-column joints under seismic design actions and found that when the design seismic moments were added to gravity load actions the sign of the bending moment in one of the beams reversed direction in:

• **line 2** – all the beam-column joints at levels 3, 4, 5 and

6. The beam-column joints at level 2 were marginal;

• **line F** – all of the beam-column joints on this line;

• **line 1** – all of the beam-column joints on this line; and

• **line A** – all the joints on line A at levels 2 and 3

were marginal and whether they were critical or not depends on the probable stiffening effect of the concrete block walls.

**8.1.6.4 Shear reinforcing of columns**

In our analyses the design inter-storey drifts, “*v*∆”, were applied to the gravity load resisting frames on lines 3 and F. It was found that the shear forces induced in the columns in the third, fourth and fifth storeys exceeded 50 per cent of the shear resistance provided by the concrete, and the columns in the second storey were marginal. In this situation Clause 7.3.4.1 in NZS 3101:1982 required the columns to have shear reinforcement that satisfied nominal shear reinforcement. To satisfy this condition the pitch of the 6mm spiral should have been reduced from the 250mm pitch that was used to 111mm or less. From the calculations it is clear that the columns on line 1 would also have required nominal shear reinforcement.

**8.1.6.5 Adequacy of the R6 @ 250mm spirals in the regions of the cranked splices in the columns**

In our view, the spiral ties complied with the requirement in Clause 5.3.27.1 of NZS 3101:1982 that ties or spirals were to be placed no more than 150mm from the point of bend. However, the spiral would only be adequate if the bar was stressed to less than 100MPa.

**8.2 Best-practice requirements**

8.2.1 Best-practice requirements

The Terms of Reference require the Royal Commission to consider whether the design of the CTV building complied with best-practice requirements (if any) current when it was designed and on or before 4 September 2010. “Best-practice requirements” is defined in the Terms of Reference as including “any New Zealand, overseas country’s, or international standards that are not legal requirements”.

Professor Priestley observed in the hearing that while building codes provide a minimum level of safety, they can lag behind the current state of knowledge. In his view, if information is available, the engineer has a duty to incorporate it into the design even if it has not yet been codified. He said that although this may not be a legal requirement, it is one that the public would expect. He regarded this as a well-established principle which, to his knowledge, had always been taught in structural engineering at universities.

When Dr Reay was asked about whether parts of the design of the CTV building complied with best-practice he said there is no definition of this term. In his view, the applicable codes incorporated accepted knowledge and therefore reflected best-practice. Compliance with the codes would therefore mean that best-practice

was achieved.

When cross-examined by counsel for Dr Reay, Professor Priestley said, “it is impossible for a designer to just design in accordance with the code”. He referred to the 1975 text, *Reinforced Concrete Structures* by Professors Park and Paulay20 of the University of Canterbury. He described this as one of the most important books in reinforced concrete design internationally, particularly for seismic structures.

Counsel for Dr Reay pointed out that Professors Park and Paulay had contributed to the development of the codes applicable at the time of the design of the CTV building. Dr Reay also noted in his evidence that these codes were drafted well after publication of *Reinforced Concrete Structures* and the authors would have ensured that important design considerations were included in the code. Dr O‘Leary also said that Professors Park and Paulay, “wouldn’t have left issues out of the Standard that they considered were necessary for good practice”.

Professor Priestley said the Code was an “absolute minimum”, which “reflects a consensus of the Code Committee”. He said that if there is “an area of some conflict”, the engineer should consult textbooks to determine whether there are any concerns in relation to the design.

In closing submissions, counsel for Dr Reay said that considerable time had been occupied at the hearing trying to identify what might have amounted to best- practice at the time of the design of the CTV building, but that a focus on best-practice “adds little to the consideration of the issues”. He submitted that, given that NZS 4203:1984 and NZS 3101:1982 were still relatively new codes at the time of the CTV design and had been written by recognised leaders, it was generally unhelpful to look significantly further than the codes for best-practice requirements at the time of the design. He also submitted that if any element of best-practice was to be considered, it should be judged from a Christchurch perspective and the opinions of engineers who had never practised in Christchurch should be treated with considerable caution.

Dr Reay believed that the building complied with best- practice requirements except in those few respects in which it did not comply with the codes. Mr Harding also gave evidence that the building complied with best- practice requirements.

The CCC submitted that issues of best-practice

fall outside the ambit of its compliance assessment role. When cross-examined by counsel for the CCC, Professor Priestley agreed that best-practice is not something that could be dealt with by a council.

Best-practice can be defined as the principles of engineering that are widely accepted by engineers at the time of design which may be additional to minimum legal requirements. It is clear that meeting best practice requirements must include complying with the fundamental assumption on which all structural design is based, namely that every load or inertial force must have an adequate load path or paths from its point of application to the foundation soils, in which equilibrium of forces and compatibility of strains is satisfied. This involves identifying the tracks of compression and tension forces through beam-column and beam-wall joint zones and junctions between other structural elements under cyclic loading conditions.

Best-practice also involves ensuring that, in the event of a major earthquake, the building will develop a ductile mechanism to prevent it collapsing in a brittle failure mode. To achieve this objective, *all* potential weak zones must be identified and detailed to ensure that they have adequate ductility to enable the building as a whole to develop a ductile mechanism. This concept was widely understood by many structural engineers in New Zealand in the early 1970s.

The designers of the CTV building could be expected to comply with best-practice when designing the building. However, to be accepted as a necessary aspect a novel approach would have to have been proven by research and be in common usage by peers.

8.2.2 Compliance with best-practice requirements

**8.2.2.1 Introduction**

The Royal Commission heard evidence that the design of the CTV building did not comply with best-practice requirements in a number of respects. As we have noted, Dr Reay said that compliance with the applicable codes equated to best-practice. In his view, the CTV design complied with best-practice except in those few cases where he considered it did not comply with the codes. Similarly, when giving evidence, Mr Harding’s position was that the design complied with best- practice in most respects.

**8.2.2.2 Areas of possible non-compliance with best-practice requirements**

**8.2.2.2.1 Absence of sufficient diaphragm connection to north wall complex at gridlines D and D–E**

Professor Priestley gave evidence that the absence

of adequate connections between the diaphragm and north wall complex at gridlines D and D–E was “very remarkable” and did not comply with best-practice. Professor Mander agreed that the connection was “remarkable” and not best-practice.

In evidence, Dr Reay accepted that this connection was “potentially non-compliant with the code”. Mr Harding did not agree that the connection failed to comply with the codes or that it was not best-practice.

For the reasons discussed in section 8.1.5.2, the diaphragm connections did not comply with the applicable code. In addition, for the reasons also described in section 8.1.5.2, they did not comply with basic engineering principles. For these reasons, they did not comply with best-practice requirements.

**8.2.2.2.2 Column detailing and spacing of transverse reinforcement**

Professor Priestley expressed a particular concern about “poor detailing” of the columns, especially given what he considered to be high axial load levels. He considered this did not amount to best-practice and cited a section from *Reinforced Concrete Structures*, which he said clearly identified such an approach as dangerous. He believed it was inconceivable that the designers of the CTV building would have been unaware of this information. Professor Priestley expressed the view that the spacing of transverse reinforcement in the columns was excessive and not best-practice. Professor Mander also said that the non- seismic detailing of the columns was not best-practice.

Dr Reay gave evidence that he “did not think it would have helped to have detailed those columns for ductile behaviour without changing the whole frame”. He did not agree that there was a failure to comply with best- practice and he said that, if Professors Park and Paulay had considered this requirement to be critical, they would have insisted on it being in the code.

Perhaps this deficiency arose because of code confusion but to fail to provide robust confinement reinforcement was a failure to comply with best- practice. The cost of adequate reinforcement would be a very small amount in the context of producing a more robust structure.

Mr Harding gave evidence that he had used ductile detailing in columns for some time whether the codes required it or not. He said that he believes designers should use this approach as a matter of course. However, he did not believe this was best-practice at the time of design of the CTV building, or even now.

For the reasons described in section 8.1.5.3, at least some of the columns did not comply with the applicable code. In our view, best-practice would have required all of the columns to have at least the level of transverse reinforcement specified in section 14 of NZS 3101:1982, which included detailing for “limited ductility”.

**8.2.2.2.3 Cover to reinforcement and axial compression of columns**

Professor Priestley expressed the view that excessive cover to reinforcement of columns resulted in inadequate load capacity of the concrete core in the event of spalling of the cover concrete. He pointed out that there was 50mm of concrete outside the core of the columns. He considered that, given the small diameter of the columns, concrete spalling would rapidly lead to a reduction of axial load-carrying

capacity, resulting in failure under a straight vertical load. He also believed that there were very high levels of axial compression in the columns.

In cross-examination, counsel for Dr Reay referred Professor Priestley to a “column design chart” that was used in the design of the columns on the CTV building. When asked whether use of this was the “appropriate best-practice approach to this design”, Professor Priestley said that it was an approach, but not necessarily best-practice.

Counsel for Dr Reay also put it to Professor Priestley that, notwithstanding his comments about high levels of axial compression, the columns nevertheless complied with the code. Professor Priestley was not sure whether they actually complied or just failed to do so.

Dr Reay agreed that spalling of the concrete could occur but maintained that the columns complied with the code in terms of load capacity. He did not agree that there was a failure to comply with best-practice. Mr Harding referred to external columns requiring a certain amount of cover to protect the reinforcement against corrosion. He said that he did not believe that the cover on either internal or external columns was excessive.

From the point of view of best-practice, the amount of cover the columns should have had depends on the protection required for environmental exposure and whether they were designed to be ductile or not. Best-practice would require a high ratio of the area of confined concrete against the area of unconfined concrete, hence less than 50mm cover for a 400mm diameter column if the columns were designed to be ductile in a benign environment. If there is a low ratio then spalling of the concrete would lead to a major reduction in load-carrying capacity and poor ductile performance. Fifty millimetres cover would be best-practice for exterior columns. Forty millimetres cover is appropriate for internal columns in an office environment.

**8.2.2.2.4 Transverse reinforcement in beam-column joints**

Professor Priestley said that the transverse reinforcement in the beam-column joints did not comply with best- practice. Professor Mander held the same view.

Dr Reay said in evidence that it is quite likely that the design of the beam-column joints as shown on the permitted drawings did not meet the minimum transverse reinforcement requirements set out in NZS 3101:1982. He agreed that, if they did not comply with the code, they would not have been best-practice.

Mr Harding agreed that the minimum spacing required by the code was 200mm and that spacing of 250mm did not comply with this requirement.

For the reasons set out in section 8.1.5.4, the transverse reinforcement in the beam-column joints did not meet the requirements of the applicable code. In addition, the transverse reinforcement did not meet best-practice requirements.

**8.2.2.2.5 Connectivity between precast beams and columns**

According to Professor Priestley, connectivity between precast beams and columns in the CTV building was poor. In his view, this failed to comply with best- practice. Mr Harding did not accept this when giving evidence. He said that the beams were designed as part of a gravity frame. The bottom bars were anchored into the core of the column as compression bars while the top bars ran from one precast beam to another to “effectively tie the whole thing together”. He said that four 24mm diameter bars in the top of the beam continuous through the joint would have provided an “excellent connection between the beam and columns”. Dr Reay said, “if connectivity means ensuring that the beams on each side of the column don’t move apart, then in fact that was provided for”.

The columns were designed to be pin ended but because both columns and beams had reinforcing that carried through the joint as well as beam reinforcing anchored in the joint the beam-column joints were subject to bending moments during building sway. These joints therefore were subject to tensile and compressive strains in beam steel even if those were not relied upon as part of a seismic resisting frame. The inadequate transfer of loads from bottom reinforcing in the beam-column joints was not best-practice.

We agree with Professor Priestley that connectivity between precast beams and columns was not

best-practice.

**8.2.2.2.6 Detailing of east-west beam connection at western wall**

Professor Mander gave evidence that the beams that were seated onto a sill on the western wall were not well anchored and he described this as “quite poor”. In his opinion, the locking of the east-west beams onto their seats on the western wall probably failed to comply with best-practice at the time of design.

We agree that the detailing of this connection did not comply with best-practice or with basic engineering principles.

**8.2.2.3 Other design features**

Some other features of the design were highlighted in evidence which, while not necessarily amounting to non-compliance with applicable codes or best-practice, nevertheless could have been improved upon.

**8.2.2.3.1 Load paths, redundancy and robustness**

Dr Hyland and Mr Smith referred to the concept of robustness in their report. They defined it as the ability of the structure to sustain damage without causing progressive damage to the building as a whole. In their view, the secondary beam and column frames lacked the level of robustness expected of frames designed to cope with the cyclic drift of earthquakes. They believed that the seismic design provisions of NZS 3101:1982 would have improved robustness. Dr Hyland also said in evidence:

Limited robustness in tying together the building was another issue. There wasn’t this redundancy or alternative load path that could have happened but really that’s the consequence of not getting those requirements for the group two beams and columns to comply with the limited ductile or ductile design.

Dr Robert Heywood gave evidence that:

…combining ductility with alternate load paths within a structure (redundancy) can also help ensure that the consequence of failure is not disproportional to the effect causing the failure (robustness).

He considered that ductile structures are desirable due to the large deformations that occur before they fail, which provides a warning of impending collapse and the opportunity for the structure to find alternate load paths to support the load.

Mr Holmes said:

…one of the very first premises of any seismic code is to have a load path. A load path means all the loads can get where they’re supposed to be and certainly they have to get to the shear walls.” Mr Jury referred to “limited robustness and lack of redundancy in the whole structure”.

He said that by “redundancy” he meant the availability of alternative load paths particularly for vertical gravity loads. In his view, “once the columns’ capacities had been exceeded there was nothing effectively to separate the floors and the floors came down”.

Mr Murray Mitchell, who carried out a desk-top review of the CTV building in 1998 or 1999, said he identified that the building lacked structural redundancy, meaning that there were no alternative load paths available in the

event that the primary load path failed. He noted that this was an initial view only.

In Professor Mander’s view, good ductile detailing, including confinement of columns, is highly desirable in the delivery of a robust structure. He considered that the CTV building did have a limited degree of robustness and redundancy and it was sufficient to survive the September earthquake. However, more robustness was necessary for the CTV building to survive ground motions such as occurred in the February earthquake. He believed that one key item missing in the CTV building was a series of north- south support beams between the columns. Although not a requirement of the codes of the day he believed that such support beams would have improved the diaphragm transfer mechanism and inhibited the possibility of out-of-plane buckling of the slabs along east-west yield lines.

Dr O’Leary gave evidence that robustness was understood by structural engineers at the time of the design of the CTV building but his understanding was that, if the design complied with the standards of the day, then the required robustness was regarded as being incorporated in the design. Mr O’Loughlin said that neither NZS 4203:1984 nor NZS 3101:1982 defined the concept of robustness. He also believed that a building was robust if it complied with the standards of the day.

Professor Shepherd observed that the word “redundancy” is possibly open to misunderstanding. He suggested that it was more accurate to refer to “load paths as backup mechanisms as the preferred manner of preventing disproportionate collapse in the case of the failure of a single load-bearing element”. When asked by counsel for Dr Reay whether the concept of having redundancy or alternative load paths was understood by engineers at the time the CTV building was designed he said:

I think there’s always been confusion as what was meant by redundancy and some engineers would argue that redundancy in its genuine terminology is not desirable if you don’t know where your loads are going. Whereas, I would suggest that some engineers 30 years ago would be very well aware of alternative load paths but that the term redundancy was bandied about with all sorts of connotations and it wasn’t clearly understood what people were talking about.

In response to questions from the Royal Commission about the adequacy of the load path to transmit inertial forces from the floors into the south shear wall, Professor Priestley noted that the force transfer

was principally provided by eight H12 bars acting as shear dowels. He said it was not clear that this was sufficient to transfer expected inertia forces to the wall particularly when higher mode effects are considered. However he noted that there were H24 bars from the peripheral beams on line 1 that were anchored into the walls, which could act as collectors, and the fact that the wall was captured by the beams at each end provided compression force transfer.

Professor Mander said it was very precarious to rely on HRC mesh to transmit the inertial forces from the floors into the south shear wall. He said awareness of this issue had improved after a full-scale test of hollow- core floor slab systems at the University of Canterbury in the 2000s, which was “not that different from this class of system here where you have a relatively thin topping and then you rely more or less entirely on mesh”. He said this mesh inevitably fails following which, “it’s very difficult to get loads into the wall via classical shear”. However he pointed out that loads could still be transmitted into the walls because the diaphragm would:

…seek an alternative load path and that alternative load path will go via the columns and then the beams themselves, could provide drag forces onto the walls. Then, those large D28 bars in the top of the beams would literally drag the walls backwards and forwards while the wall is oscillating back and forth so instead of relying (on) shear coming in from the outside, which is possibly what was conceived of, it’s going to rely on these drag forces so that puts a lot of distress on the beam-column joints facing the framing into the wall and they themselves will end up eventually becoming distressed as well.

Professor Priestley also commented on the load path for shear transfer between the floors and north wall complex. He said it was not clear to him that the designer had specifically considered the load path for shear transfer between the floors and the north wall complex. He observed that, given that in the north-south direction the north wall complex was concentrated near the centre line of the building, the inertial forces from the outlying sections of the floor would need to be transferred by truss action, implying diagonal compression forces and a collector tie along line 4. He did not think this had been considered. He said that eccentricity of the lateral resistance in the east-west direction would also require moment as well as shear to be transferred across a rather small interface between the slab and the north wall complex.

Professor Mander referred to two potential load path mechanisms for shear transfer between the floors and the north wall complex, that depended on which

direction the loads came from. For east-west forces,

he referred to an inherent weakness, in that reliance on the slab steel to provide shear resistance could lead to tearing. He thought there was a weak plane beyond the starter bars although, provided the floor plate stayed intact, an alternative mechanism based on strut action could provide a secondary backup system. He said:

…that’s not something that designers even knew about really at the time. It wasn’t carefully thought through, however we now realise in hindsight that that is, it can be used as a primary mechanism if designed for accordingly and it also can be used as a secondary mechanism.

In closing submissions, counsel assisting submitted that one of the most serious consequences of some of the alleged failures to comply with the Bylaw and best- practice was the inadequacy of load paths in the CTV building. This was due to the failure to apply capacity design, the undercalculation of required loads to diaphragm-wall connections and poor anchorage and detailing at walls and beam-column joints throughout the building. It was submitted that the effect of this was that the building was incapable of carrying load through its structural elements to the walls and then to the foundations, and that this inadequacy may have been one of the most fundamental reasons for the collapse. Counsel assisting submitted that the CTV building should have been designed to have redundancy so that, if one part, such as the columns or beam-column joints, failed it should not have resulted in collapse.

When giving evidence, Dr Reay recognised the importance of providing adequate load paths in buildings. He described this as “fundamental engineering”, which “applies to every building that you design”. However he resisted any suggestion that robustness imposed an additional or desirable requirement above the code. He described robustness as a difficult concept to quantify. Dr Reay said robustness was “a word that was bandied around you might say but fundamentally if you complied with the code then the structure should have been robust”.

He did not agree that the secondary beam and column frames lacked robustness or that the building lacked redundancy in that if the columns or beam-column joints failed whole or partial collapse would result. He said, “Well that’s true of most buildings so I don’t quite understand how that’s been put in that term so I can’t, can’t really agree with, with it other than that’s inevitable with every building”.

He said that the CTV building was designed such that the shear walls would provide lateral shear resistance to the building with no assistance from the internal columns or beams and the walls and their foundations were designed adequately to carry out that function.

Dr Reay was questioned by Commissioner Fenwick about the way in which the strength of the connections between the diaphragms and the walls was calculated. Dr Reay agreed that capacity design required the connections to be capable of developing the over- strength of the wall. However, he pointed out that, while this was the case, the code allowed the use of forces derived from the parts and portions section of NZS 4203:1984. Commissioner Fenwick put it to Dr Reay that it “did not make much sense” to use these forces. Dr Reay agreed that, in hindsight, it did not.

Mr Harding acknowledged that application of the seismic provisions of NZS 3101:1982 would have improved robustness, however he said:

…it is a difficult thing to identify just what you mean by that…I guess one way of looking at it is that there’s a secondary line of defence if the first one fails, and I guess that’s really what we’re talking about. So I think we’ve learnt since the earthquakes that buildings move a lot more than we thought they would and things happen that, not just in terms of vertical acceleration but also one building hitting another, and by buildings deflecting by more than you think that they should, so I agree that in the future we should be giving that a lot more thought. But I don’t accept that. At the time we designed it, it was believed that it was sufficiently robust to be safe.

The fact that the columns would collapse if they did not have ductility meant that they forestalled the development of an alternative mechanism for resisting horizontal loads. It is in this sense that they lacked the robustness to permit an alternative load path. If they had stayed intact even though fatally damaged they would have provided an amount of propping support to prevent pancaking. The same comment relates to the limitation of elements to continue to transfer diaphragm forces once capacity of brittle membranes were exceeded. This applies to 664 mesh and drag bars.

**8.2.2.3.2 Use of different structural type factors for the north wall complex and south shear wall**

Mr Harding agreed that a structural type factor of 1 was used for the north core complex and 0.8 for the south shear wall. Mr Ashley Smith said that this resulted in a disparity between these walls, which would have led to the latter yielding before the former. In his view, this could have increased inter-storey drifts.

Mr Harding said that NZS 4203 specified different structural type factors for a coupled shear wall and cantilevered shear wall. He did not agree that the same structural type factor should have been used across the structure. Dr Reay also considered that, as long as the designer complied with code, there was no issue with using different structural type factors.

Table 5 in NZS 4203:1984 gave values of S for different structural types. For example, for a coupled shear wall, such as the south wall of the CTV building, the table gave a value of 0.8, which basically allowed design forces to be reduced by 0.8 on the basis that this form of element was ductile and could dissipate seismic energy. The commentary suggested that the 0.8 value could be applied to the individual structural element, though it cautioned that “this method has not been

fully researched”.

In our view, it was illogical to apply this coefficient to the CTV building. The S factor was, “intended to reflect the potential seismic performance of different structural systems”. In the CTV building the north wall complex was strong compared with the south shear wall. Consequently in an earthquake all the seismic energy needed to be dissipated by the south coupled wall. In short, the arrangement required the wall to work twice as hard at dissipating energy as an arrangement where there were two such walls with one at each end. To encourage this imbalance was inappropriate and in this case clearly the S factor should have been 1.0 or greater.

However, these observations can only be made in hindsight. We do not criticise Mr Harding for adopting this approach at the time, especially given that such an approach was contemplated by NZS 4203:1984.

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Note: Standards New Zealand was previously known as the Standards Association of New Zealand and the

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