**Section 9:**

**Summary of conclusions and recommendations**

At 12:51pm on 22 February 2011 the CTV building, situated on the corner of Madras and Cashel Streets in Christchurch, collapsed as a result of a 6.2 magnitude earthquake. One hundred and fifteen people who were in the building at the time of the earthquake were killed. A small number of people, who were on the top three levels of the building, were rescued and one person ran from the building during the earthquake.

The Royal Commission conducted a hearing into the collapse of the CTV building which ran for eight weeks and heard from over 80 witnesses. In accordance with our Terms of Reference, evidence was called about the design and construction of the building, subsequent alterations, and the involvement of the local territorial authority, the Christchurch City Council (CCC), in the permitting, construction and occupancy of the building. In addition, there was evidence about the damage caused by the 4 September 2010 earthquake and subsequent aftershocks, and about the building assessments that followed those events.

We heard from a number of eyewitnesses to the collapse of the building and from engineers who were working at the site after the collapse. We also heard from expert witnesses who have analysed the building closely and gave evidence as to the reasons why they believed the building failed and whether it was designed in accordance with the legal requirements at the time it was designed and built.

In this Volume of our Report we considered the evidence we heard about the building, from its design in 1986 to its collapse in 2011. We made findings and drew conclusions where we have been in a position to do so and made recommendations where appropriate.

The following section provides a summary of those conclusions and recommendations.

**9.1 Structure of the CTV building**

The CTV building was a six-storey commercial building founded on pad and strip footings bearing on silt, sand and gravels. The building had two seismic resisting elements, the north wall complex which extended out beyond the north end of the building and a coupled shear wall on the south of the building. Together with columns and beams supporting the floors, these created a shear-wall protected gravity load system. The columns between floors were designed to support gravity loads, and to flex in an earthquake without failure.

Figure 111 shows a typical upper floor structure of the building.

A B C D E F

 4750

 7000

 7000

 7000

 4500

11650

North wall complex

5

Landing N

Wall C

Wall D-E

Toilets

4500

Stair

Lifts

Wall C-D Wall D

4

Precast beams

Rectangular columns to west wall (typical)

7500

West block wall up to level 3

Precast beams

3

400mm O

columns (typical)

7500

Spandrel

22500

(typical)

Precast beams

2

Precast beams

7500

1

South shear wall

Precast beams

30250

5000

**Figure 111: Typical upper floor structure**

**9.2 Engineering design of the building** The building was the result of a speculative property development started in 1986 by Prime West Corporation Limited, which owned the land at the time. Prime West invited Williams Construction Limited to submit a proposal to design and build a commercial building on the site. Alun Wilkie Associates were engaged as the architect and Alan M Reay Consulting Engineer (ARCE) as the engineer.

Dr Alan Reay was the principal of ARCE and

Mr David Harding was employed by him as an engineer at that time. Mr Harding carried out the structural design of the building. Part of the analysis he carried out for the design involved him using the ETABS computer program at the University of Canterbury to carry out a model response spectrum analysis. He lacked experience with the ETABS program and his evidence established that he was unaware of some of the program’s important limitations.

Prior to this time, Mr Harding had not designed a multi-storey building with a significantly eccentric configuration. However we have found that he did not seek assistance with the design from Dr Reay or

anyone outside of ARCE. While Mr Harding’s position at

the hearing in 2012 was that he was not competent to design the CTV building without review of his work, we are satisfied that this was not the view he held in 1986, when he was confident that he could carry out the design. Mr Harding signed the structural drawings and a building permit was granted by the CCC. Dr Reay’s evidence was that he did not check or review any structural details for the building prior to the building permit being granted.

We have found that there were a number of non- compliant aspects of the CTV building design. We have concluded that a primary reason for this was that Mr Harding was working beyond his competence in designing this building. He should have recognised this himself, given that the requirements of the design took him well beyond his previous experience. We also consider that Dr Reay was aware of Mr Harding’s lack of relevant experience and therefore should have realised that this design was pushing him beyond the limits of his competence. Dr Reay should not have left Mr Harding to work unsupervised on the design or without a system in place for reviewing the design, either by himself or someone else qualified to do so.

The process led to a building design that was deficient in a number of important respects.

**9.3 Building permit**

An application for a permit to construct the CTV building was lodged with the CCC on 17 July 1986. Mr Bryan Bluck was the CCC buildings engineer at the time the permit for the CTV building was processed. Mr Graeme Tapper was his deputy. After structural drawings were lodged with the CCC by ARCE on 26 August 1986, Mr Tapper sent a letter dated 27 August 1986 to ARCE requesting further information. In that letter Mr Tapper identified a number of issues arising from the drawings, including the fact that they were not signed, as required by the Bylaw. He also asked for calculations to be provided to support the design.

At the hearing, neither Mr Harding nor Dr Reay could remember any involvement in responding to this letter. On 5 September 1986 a document transfer form, signed by Mr Harding, was sent to the CCC enclosing a further set of structural drawings and two additional pages of calculations. Mr Tapper signed off on the structural design of the building on 10 September 1986 and a permit was issued for the building on 30 September 1986.

We have found that at the time of writing his letter of

27 August 1986, Mr Tapper had identified that the connection of the floors (which acted as a diaphragm) to the north wall complex was inadequate and non- compliant. While a further set of drawings was provided to the CCC on 5 September 1986, we have concluded that the issue of the floor connection was not resolved in the permitted drawings signed by Mr Harding.

We have accepted the evidence given by

Mrs Patricia Tapper, the widow of Mr Tapper, that he had concerns about the design of the CTV building but was under pressure to approve it. We have also accepted the evidence of Mr Peter Nichols, a former CCC structural checking engineer who spoke to Mr Bluck in front of the CTV building while it was under construction. Mr Nichols said Mr Bluck told him that he too had had concerns about the building but that he had been “convinced” by Dr Reay that his concerns were unfounded.

We have found that Dr Reay became involved in the permit process between 5 and 10 September 1986. It is likely that there was a meeting called, which resulted in Dr Reay convincing Mr Bluck that the concerns about the building design were unfounded. This is despite the fact that on his own evidence Dr Reay knew very little about the structural details of the building, having not reviewed any of the structural drawings prior to a permit being issued. Mr Tapper was either persuaded that his concerns were unfounded, or more likely was directed

to approve the structural design, which he did on

10 September 1986.

Dr Reay’s involvement in the permitting process contributed to the issue of the building permit. We are satisfied that the permit should not have been issued because the design did not comply with the CCC’s Building Bylaw (Bylaw 105).

**9.4 Construction**

Williams Construction signed a building contract with Prime West for $2,450,000 in October 1986. Work started on the site later that month. The contract was assigned to Union Construction Limited during the construction and the building was completed by the company in late 1987 or early 1988.

After the building collapsed, a number of construction defects were identified, including the absence of roughening of construction joints between precast and in situ concrete. Some precast beams were found to have reinforcing bars bent back towards the beam instead of embedded into the north wall complex as the design intended.

On the evidence we heard we have found that the foreman, Mr William Jones, may have been a competent and experienced foreman. However, he did not receive the guidance, mentoring and technical advice he needed and expected from a competent construction manager. The construction manager, Mr Gerald Shirtcliff, did not spend sufficient time on the site to perform his role adequately.

ARCE was contractually responsible for supervising the construction. Although Mr Harding said he visited the site regularly and completed site inspection reports, this did not prevent the construction defects from occurring. We have concluded that the lack of roughening should have been visible to the engineer if he was carrying out regular site inspections, as well as to the foreman and construction manager.

In addition, the CCC’s records show a five-month gap in inspections between April and August 1987, with no apparent explanation. An expert witness called by the CCC said the inspections the CCC carried out were “a bit light” for a building of that size. We are unable to answer why there was such a gap in the CCC inspections of this building.

Notwithstanding the conclusions of the Hyland/Smith technical investigation report (prepared for the former Department of Building and Housing)1 that the strength of the concrete in the columns was insufficient,

we have concluded that the concrete was likely to have been at or above the strength specified by the designer. No reliable evidence has been given to suggest the concrete was under-strength in any columns.

**9.5 Building retrofit**

In 1990, Holmes Consulting Group (HCG) carried out a pre-purchase review of the building in the course of which Mr John Hare identified the non-compliance of the connections between the floors and the north wall complex. He reported this to Alan Reay Consultants Limited (ARCL), which ARCE had become by that time. ARCL assumed that the review carried out by HCG had identified all areas of non-compliance and they did not carry out their own full review of the building. The identification of such a fundamental design error should have signalled to ARCL the need for a more detailed review of the design.

ARCL’s response to this issue was to install steel angles (referred to in the evidence as “drag bars”) on levels 4 to 6 to connect the north wall complex and the floors. The analysis and design for these was carried out by another principal of ARCL, Mr Geoff Banks, who had had no prior involvement with the building. The drag bars were not installed until October 1991. The connection between the floors and the north wall complex was a significant issue that had the potential to affect the safety of users of the building. We consider that Dr Reay should have acted more expeditiously and proactively to resolve this fundamental defect. While the delay was unacceptable, we recognise that Dr Reay and Mr Banks did take some action after they became aware of the sale of the CTV building to Madras

Equities Limited.

Although there appears to have been an element of minimisation of the defect by Dr Reay and Mr Banks in their communications with the receiver of Prime West and with Madras Equities, we consider that this was likely to have been motivated by the perceived need to protect ARCL’s insurance cover rather than any ulterior motive.

Although retrofitted drag bars were never going to be as effective as connections designed using standard ductile reinforcement would have been if included in the original design, Mr Banks was correct in his view that the Loadings Code2 did not require that drag bars be installed on levels 2 and 3. In any event, despite the installation of drag bars on three levels, the connections between the floors and the north wall complex remained non-compliant for seismic actions in the east-west direction. This defect was not identified and therefore not remedied.

No building permit was obtained for the installation of the drag bars. The failure to apply for a permit was a clear omission which meant that the inadequacy of the connections to the north wall complex in the original design was not drawn to the attention of the CCC in 1991.

In our view, this issue illustrates that there should

be a legal obligation to disclose knowledge about a structural weakness that has the potential to affect the safety of users of a building or the public to an independent statutory body, such as the territorial authority. This would ensure such matters are rectified expeditiously. Such an obligation should not only apply to engineers, but also to people such as owners, contractors and others who become aware of such information.

We address this issue further in section 4 of Volume 7 of our Report.

**9.6 The building from 1991 to the**

**September 2010 earthquake**

The CCC issued a number of permits and consents (including resource consents) for work on the CTV building between the time of the original construction and 4 September 2010. In most cases, the approved work would have had no impact on the structural performance of the building in an earthquake.

A penetration was cut in the floor of level 2 for the installation of an internal staircase during a fit-out in 2000. We are satisfied that the penetration would not have affected the seismic performance of the building. However, in our view particular care should be taken to ensure that damage to critical reinforcing does not occur when buildings are altered.

**Recommendation**

We recommend that:

107. Where holes are required to be drilled in concrete, critical reinforcing should be avoided. If it cannot be avoided, then specific mention should be made on the drawings and specifications of the process to be followed if steel is encountered, and inspection by the engineer at this critical stage should be required.

We heard evidence claiming that a significant number of holes may have been drilled in concrete in the building during its lifetime. We were unable to find on the evidence that significant holes were drilled in the structural members of the building. In any event, we consider it is unlikely that any holes that were drilled would have had any effect on the seismic performance of the building.

In 2001 an application for a building consent for an education tenancy, Going Places, was submitted to the CCC. It was treated by the CCC as a change of use, from an office to a school. Under section 46(2) of the Building Act 1991 the CCC could have required the owner to upgrade the building to as near to the current Building Code as was reasonably practicable. It did not do so, on the basis that the building was relatively new and the change of use related to a single floor. The assumption made by the CCC in considering this application was that the building had been designed, permitted and constructed in accordance with the legal requirements of 1986, but as we have already concluded this was not the case.

Madras Equities failed to notify the CCC of a change of use when another education business, King’s Education, moved into level 4. This meant that the intended statutory protection for users of the building was unable to have any effect.

The building was not identified by the CCC as

‘earthquake-prone’ as that term is defined in the Building Act 2004. This is consistent with the opinion expressed in the Hyland/Smith report that the capacity of the building would have been in the order of 40 to 55 per cent of the standard for new buildings, when the earthquake-prone threshold was 33 per cent of that standard.

**9.7 The September 2010 earthquake and post-earthquake assessments**

The Royal Commission has conducted investigations into the nature and characteristics of the Canterbury earthquakes, with a particular focus on the earthquakes of 4 September 2010, 26 December 2010, 22 February 2011 and 13 June 2011. Section 2 of Volume 1 of the Report describes the nature and severity of the earthquakes.

The CTV building suffered some damage in the September earthquake and a number of witnesses gave evidence about this. A Level 1 Rapid Assessment (a brief exterior visual inspection) was conducted on 5 September and a green placard (a notice providing no restriction on use) allocated to the building.

On 7 September, three CCC building officers carried out a further inspection. There was no engineer present. The officers were sent out without clear instructions and this led to the decision to treat the visit as a Level 2 Rapid Assessment (a brief interior and exterior visual inspection), which resulted in the green placard being confirmed, even though an engineer had not assessed the building, generally required for a Level 2 Rapid Assessment. This inspection should not have been regarded or recorded as a Level 2 Rapid Assessment. In our view, the officers should have made it clear to the occupants that they did not have the expertise or information to conduct that type of assessment. They did, however, recommend that the owners engage an independent engineer to assess the building, although this was not recorded on the rapid assessment form. We also recognise that there can be no certainty that if an engineer had been present for this inspection that the existing green placard would have been replaced by a yellow placard (a notice providing limited access to the building).

The building manager, Mr John Drew arranged for

Mr David Coatsworth, a chartered professional engineer from CPG New Zealand Limited, to carry out a private assessment. Mr Coatsworth inspected the building on 29 September 2010 and again on 6 October 2010, following which he issued a report to Madras Equities. In Mr Coatsworth’s view, although the building showed noticeable damage to non-structural elements such as linings and finishings and some minor structural damage, there was no evidence of structural failure. Following a further inspection on 19 October 2010, he emailed Mr Drew confirming that the building remained structurally sound. Mr Coatsworth also made recommendations for further assessment. These were not carried out. It would have been preferable if Mr Drew had arranged for this to occur expeditiously.

We are of the view that, in terms of the damage- based inspections that were being conducted after the September earthquake, the inspection carried out by Mr Coatsworth was the most thorough of all of the inspections that we considered over the course of the Inquiry. Nevertheless, lessons can be learned from the evidence we heard about Mr Coatsworth’s inspection. There should be clear communication to owners and tenants about the type of assessment an engineer has carried out so that they understand what is being done. It would have been preferable for Mr Coatsworth to have clearly explained the nature, extent and limitations of his assessment. However, the way he communicated to Mr Drew and the type of assessment he recommended and then carried out was common to most engineering assessments in the post-September earthquake period.

Even though Mr Coatsworth identified that viewing structural drawings of the building would be useful, he did not examine these before forming his view about the building, as they were not able to be accessed from the CCC at the time. The majority of engineers in his position at that time would have proceeded in the same way. Despite this, it is advisable that all inspections of multi-level buildings that are owner-initiated and take place outside the emergency response period should include a review of the structural drawings if they exist.

There are difficulties with reliance on a solely damage- based assessment following a significant earthquake. While a damage-based assessment is a necessary component of the rapid assessment process, it cannot be the sole basis upon which the decision of whether a building like this should be occupied in the long term is made.

We address these and other related issues in section 2 of Volume 7 of our Report.

The CTV building sustained some further damage as a result of the “Boxing Day” earthquake on 26 December 2010. A Level 1 Rapid Assessment was conducted on 27 December which resulted in the allocation of a green placard. An Urban Search and Rescue (USAR) rapid visual survey also took place. However, no Level 2 Rapid Assessment took place after Boxing Day, and Mr Coatsworth was not asked to reassess the building.

Mr Drew gave evidence that he believed that the further damage he saw was not significant based on an earlier conversation with Mr Coatsworth. He assumed that the widening of the cracks in the concrete was normal and expected. At the very least, we consider that Mr Drew should have spoken to Mr Coatsworth about the increased damage as there was potential for the damage to be worse than it appeared. The best approach would have been for him to ask Mr Coatsworth to return to re-inspect the building in view of Mr Coatsworth’s knowledge of damage from the September earthquake.

**9.8 The building from the September**

**2010 earthquake to 22 February 2011**

The demolition of the buildings to the west of the CTV building between the September and February earthquakes caused a great deal of anxiety for occupants of the CTV building. We have considered evidence from the CCC engineer who considered the building consent for this work and from experts who investigated the collapse of the building. We agree with the experts’ opinion that it was unlikely that the demolition work caused structural damage to the CTV

building, although the noise and vibrations were clearly disturbing to its occupants.

A medical practice, The Clinic, moved into the CTV building in January 2011 after its existing premises were deemed to be dangerous following the Boxing Day earthquake. As well as being the CTV building manager, Mr Drew was also the owner of The Clinic and was legally entitled to relocate The Clinic into the CTV building without notifying the CCC because a medical practice was not a change of use under the Building Act 2004. Bereaved families were concerned about whether the building was suitable for use as a medical clinic without alteration or refurbishment. However, as this is not relevant to why the building failed, it is outside the Royal Commission’s Terms of Reference and we cannot comment on this.

**9.9 The collapse of the CTV building on 22 February 2011**

The Royal Commission heard evidence about the collapse of the building during the February earthquake. A number of witnesses gave evidence of the building twisting as it shook, a brief period when the initial twisting appeared to stop, a tilt towards the east, a vertical jolt and the building pancaking, all of which took place very soon after the shaking started. We have concluded that the collapse was completed within 10–20 seconds of onset of the earthquake

Shortly after the collapse of the CTV building a fire started that continued for some days. Mr Peter Wilding, National Manager of Fire Investigation and Arson Reduction for the New Zealand Fire Service, gave evidence that a fire investigation was not undertaken at the CTV site for a number of reasons. There was a lack of available specialist fire investigators, and the fact that Fire Service operations at the CTV site were focused on rescue, fire suppression to aid rescue and later assisting with body recovery. He said the CTV site was significantly disrupted from an evidential viewpoint which meant no credible and reliable conclusions about the origin and cause of the fire could have been reached. We agree that it would not have been possible for the Fire Service to determine the ignition point of the fire, or the sequence in which it burned.

Following the earthquake, USAR engineers

Mr Graham Frost, Dr Robert Heywood and

Mr John Trowsdale took extensive photographs and labelled building elements. Their public-spirited initiative created an excellent record of the state of the building and individual elements following collapse. There

was no formal system whereby this information was

collected and the Royal Commission commends these engineers for their very thorough documentation and assessment of the collapse debris.

There were criticisms of the absence of a system in place to preserve the scene. However, the combination of the evidence of Mr Frost, Dr Heywood and Mr Trowsdale, together with other expert observations and the eyewitness accounts, provide a reasonable forensic basis for consideration of the relevant issues the Royal Commission has to address.

Overall, we consider that the evidence provided an adequate basis to make findings about the state of

the building after its collapse and to draw conclusions about possible collapse scenarios. However, implementation of practice guidelines for forensic engineering is warranted to ensure that high quality forensic work is guaranteed for future investigations.

**Recommendation**

We recommend that:

108. The Ministry of Business, Innovation and Employment should consider developing guidelines for structural failure investigations, including circumstances in which sites should be preserved for formal forensic examination.

**9.10 Reasons for the collapse**

It is our opinion that the CTV building collapsed in the

February earthquake for the following reasons:

• the ground motion of the February earthquake was

unusually intense although it was of short duration;

• the designer failed to consider properly or adequately the seismic behaviour of the gravity load system. In particular no consideration appears to have been given to load tracking though the beam- column joint zones. The failure to consider this aspect led to joint zones that were easy to construct but lacked ductility and were brittle in character;

• the columns were inadequately confined and could not sustain the deformation they were required to undergo without failure;

• the correct tie forces between the floors and the north wall complex were not determined and the load path was not tracked between the wall and the floors;

• as designed, the connection between the north wall complex on lines D and D-E and the floors was inadequate and non-compliant. While the addition of drag bars to levels 4, 5 and 6 in 1991 remedied the non-compliance in the north-south direction, this was not the case in the east-west direction. In addition, the drag bars failed in the February earthquake, or possibly in the September earthquake, due to their lack of ductility; and

• the interface between the ends of the precast beams and the in situ concrete in the columns was not roughened, so shear could not be transmitted across the interface by aggregate interlock action.

We repeat here the conclusion that we stated in section

7.4 of this Volume of our Report. The design of the CTV building relied on the north wall complex and the south coupled shear wall to resist the lateral loads generated by earthquakes. The defects that have been identified and discussed above meant that, in the strong shaking generated by the February earthquake, these two walls were not able to function as the designer intended. We are satisfied from the eye witness accounts that the collapse of the building would have occurred within 10–20 seconds of the commencement of the earthquake. It was a sudden and catastrophic collapse, as recounted by both survivors within the building and those who observed it from nearby. After an initial period of twisting and shaking all of the floors dropped, virtually straight down, due to major weaknesses in the beam-column joints and the columns. Eyewitnesses described the collapse as a “pancake” effect. The north wall complex was left standing, the floors having torn away and coming to rest stacked up adjacent to its base. The south shear wall collapsed inwards on top of the floors in what we consider would have been the last part of the collapse sequence. The observed damage to both of these walls showed that they had not been able to perform their intended role.

Our analysis of the collapse is consistent with the eyewitness accounts.

**9.11 Issues with the structural system** There is a major question in relation to Mr Harding’s calculations of the extent to which the CTV building would deflect in an earthquake. Mr Harding based his calculations on deflections at the centre of mass and failed to allow for the increase in deformation due to torsional rotation of the building. This led to an under- estimate of the inter-storey drifts of the columns on lines 1 and 2 in the CTV building (see Figure 111). This error had implications for the seismic performance of the building.

There were major weaknesses in all of the beam- column joints in the building. These arose from the longitudinal reinforcement in the bottom of the beams and in some cases the longitudinal reinforcement in the top of the beams, being anchored into the beam column joint zones by 90º hooks located in the mid- region of the beam-column joint zones. The failure of the designer to track the load path through the beam-column joints resulted in critical tension forces being dependant on the tensile strength of concrete. When this concrete failed in tension, rapid strength degradation would have occurred.

From the design calculations and the structural drawings for the CTV building it is also clear that the floors were inadequately tied into the north wall complex. Mr Harding based his calculations for the required tie forces on the values used in the equivalent static analysis, which gave forces that were less than half those required by the Standard for general structural design and design loadings (NZS 4203:19843). In addition he failed to allow for the in plane bending moments associated with the tie forces for seismic actions in the east-west direction.

**9.12 Compliance with legal requirements**

The legal requirements relating to the engineering design of the building were set out in CCC Bylaw 105. The Bylaw listed various building standards as means of compliance, including the Standard for general structural design and design loadings for buildings (NZS 4203:1984) and the Standard for design of concrete structures (NZS 3101:19824). We have concluded that the design of the CTV building did not comply with the requirements of the Bylaw in the following respects:

• The design of the connections between the floor slab diaphragms and the north wall complex did not comply with Clause 3.4.9 of NZS 4203:1984 for seismic forces in both the north-south and

east-west directions. The addition of drag bars in

1991 remedied this non-compliance in the north- south direction only, though the brittle nature of the drag bar connections to the floors reduced their effectiveness; and

• We analysed columns and beam-column joints in the CTV building to identify whether the detailing complied with the specifications in the relevant design Standards. It was found that a number of columns did not comply in terms of the required column confinement reinforcement and the shear reinforcement provisions. Further analysis showed that many of the beam-column joints were inadequately detailed to comply with NZS 3101:1982.

These conclusions are fully detailed and explained in section 8.1 of this Volume of our Report.

It was the view of the experts we heard from on this issue that the CCC should have identified the non- compliance in the connections between the floor slabs and north wall complex. We have concluded that Mr Tapper did this. In our view, the non-compliance in relation to the shear reinforcement of columns should also have been identified. We do not think it reasonable to expect a reviewing engineer in 1986 to have identified the non-compliance of the columns or beam-column joints. All the reviewing engineer could reasonably do was to satisfy himself that the issue had been addressed.

However, we are satisfied that because of the instances of non-compliance that have been demonstrated and discussed in section 8.1, the building permit should not have been issued.

**9.13 Compliance with best-practice requirements**

As we noted in section 8.2.1 of this Volume of our Report, 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 design of the CTV building did not comply with best-practice requirements in the following ways:

• the connections between the floor slabs and the north wall complex did not comply with basic engineering principles;

• the spacing of the transverse reinforcement in the

columns was excessive;

• the transverse reinforcement of beam-column joints

was inadequate;

• the connectivity between pre-cast beams and

columns was inadequate; and

• the locking of the east-west beams onto their seats on the western wall failed to comply with basic engineering principles.

These conclusions are discussed and explained in section 8.2 of this Volume.

**9.14 The assessment of other buildings with potential structural weaknesses**

It is important to identify other buildings in New Zealand that may have characteristics that might lead to their collapse in a major earthquake, so appropriate steps can be taken to reduce the potential hazard posed by these structures. We make recommendations about the manner in which such buildings should be assessed.

**Recommendations**

We recommend that:

109. In the assessment of buildings for their potential seismic performance:

• the individual structural elements should be examined to see if they have capacity to resist seismic and gravity load actions in an acceptably ductile manner;

• relatively simple methods of analysis such as the equivalent static method and/or pushover analyses may be used to identify load paths through the structure and the individual structural elements for first mode type actions. The significance of local load paths associated with higher mode actions should be considered. These actions are important for the stability of parts and portions of structures and for the connection of floors to the lateral force resisting elements;

• the load path assessment should be carried out to identify the load paths through the different structural elements and zones where strains may be concentrated, or where a load path depends on non-ductile material characteristics, such as the tensile strength of concrete or a fillet weld where the weld is the weak element;

• while the initial lateral strength of a building may be acceptable, critical non- ductile weak links in load paths may result in rapid degradation in strength during an earthquake. It is essential to identify these characteristics and allow for this degradation in assessing potential seismic performance. The ability of a building to deform in a ductile mode and sustain its lateral strength is more important than its initial lateral strength; and

• sophisticated analyses such as inelastic time history analyses may be carried out to further assess potential seismic performance. However, in interpreting the results of such an analysis, it is essential to allow for the approximations inherent in the analytical models of members and interactions between structural members, such as elongation, that are not analytically modelled.

110. Arising from our study of the CTV building,

it is important that the following, in particular, should be examined:

• the beam-column joint details and the

connection of beams to structural walls;

• the connection between floors acting as diaphragms and lateral force resisting elements; and

• the level of confinement of columns

to ensure they have adequate ductility to sustain the maximum inter-storey drifts that may be induced in a major earthquake.

In sections 8 and 9 of Volume 2 and section 6.2.5 of Volume 4 of our Report we discuss other issues related to the assessment of the potential seismic performance of existing buildings.

**9.15 Conclusion**

The collapse of the CTV building caused much more extensive injury and death than any other building failure on 22 February 2011. Even though it was designed under relatively recent building Standards, its failure was severe and resulted in the floor slabs pancaking, leaving most of those inside the building with no chance of survival.

The engineering design of the building was deficient in

a number of respects. While there were elements of the applicable codes that were confusing, a building permit should not have been issued for the building as designed. There were also inadequacies in the construction of the building. The post-earthquake inspections of the CTV building also illustrated areas in which building assessment processes could be improved.

Our Inquiry into the failure of the CTV building has, together with other parts of our Inquiry, highlighted a number of areas of potential improvement in relation to the design, construction and maintenance of buildings in a country that is susceptible to earthquakes. The recommendations we have made in our Report are directed to ensuring that, where possible, tragedies like this one are prevented in the future.

**References**

1. Hyland C., and Smith, A. (2012). *CTV Collapse Investigation for Department of Building and Housing:*

*25 January 2012*. Wellington, New Zealand: Department of Building and Housing.

2. The Loadings Code specified the forces and loads to be used in the design of buildings.

3. NZS 4203:1984. *Code of Practice for General Structural Design and Design Loadings for Buildings*, Standards New Zealand.

4. NZS 3101:1982. *Code of Practice for Design of Concrete Structures*, Standards New Zealand.

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