Section 6: Technical discussions on structure

6.1 Design earthquake loading and analysis of the CTV building

The analytical model developed by Mr David Harding for determining the seismic design action in the CTV building consisted of the south shear wall and the north wall complex linked by the floors. The columns and beams were excluded as their stiffness was small (relative to the walls) and they would have made only a minor contribution to the lateral strength and lateral stiffness of the building. That assumption was conservative and reasonable.

The north wall complex was much stiffer than the south shear wall. Consequently there was a high degree of eccentricity of the centre of mass from the east-west centre of lateral stiffness in the building. In this situation NZS 4203:1984[1] recommended that the 3D spectral modal analysis method be used. This method is frequently referred to as the modal method or more specifically as the modal response spectrum method or the elastic response spectrum analysis (ERSA). However the Standard still permitted the equivalent static method to be used for this type of structure.

Mr Harding initially adopted the following assumptions:

- the building was intended for normal occupancy and the associated risk factor was 1.0;
- ductile cantilever shear walls provided the lateral resistance and hence the structural type factor (S) was 1.0; and
- the material was reinforced concrete, which had a material factor (M) of 0.8.

Analysis indicated that the fundamental period in both the east-west and north-south directions was equal to 1.06 seconds. This appears illogical when compared to Mr Harding's calculated deflections in the east-west direction, which were less than half the deflections calculated in the north-south direction. However, it is not clear at what point on the structure the design drift locations applied.

NZS 4203:1984 placed limits on the numerical output obtained from a modal analysis. The base shear from a modal analysis was not permitted to be less than 90 per cent of that obtained from a corresponding equivalent static value. There was a second limit imposed: no storey shear above the base level was permitted to be lower than 80 per cent of the corresponding storey shear from the equivalent static analysis.

We have studied a copy of Mr Harding's design calculations. As the computer input and output is not available there is some uncertainty about how some of the design values were obtained. Mr John Henry has also assessed the design calculations and we are in agreement with his assessment that:

1. Mr Harding found that the base shear from his modal analysis needed to be scaled up to 90 per cent of the equivalent static method.
2. Mr Harding then checked the 80 per cent minimum rule for the modal storey shear against the corresponding equivalent static values. For this exercise Mr Harding multiplied the equivalent static shears by 0.8. It appears that he adjusted the modal shear for the M factor by multiplying by 0.8. We think this was an error in that the S and M factors had already been incorporated in the modal analysis.

When the comparison was made it was found that the 80 per cent rule required the modal storey shears to be replaced by 80 per cent of the corresponding equivalent static values. In effect he appears to have designed the building for 80 per cent of the equivalent static lateral forces, which would be inappropriate. The Foreword to NZS 4203:1984 cautioned designers about the imprecision inherent in modal analysis. It stated that:

> Designers should recognise that the precise properties of construction materials and of structural elements made from them are not clearly known. Furthermore, the interaction of these elements in a building frame under load is extremely uncertain, so that the total design technique is one of some degree of imprecision. In fact, the design results depend so much on the nature of the mathematical model of the building as envisaged by the designer that the use of more advanced techniques of earthquake analysis can easily lose validity.

Furthermore Mr Harding reduced the design loadings for the south shear wall by a structural type factor, S, of 0.8. This increased the imbalance between the north wall complex and south shear wall because the reduction in load was applied to the latter wall only. The S factor, along with the material factor, M, reflects the available ductility in a system. The code commentary stated that a method of determining rational design actions for buildings having horizontal force-resisting systems in parallel, with differing S and M values in the direction being considered, is to assume that S and M are equal to 1.0 for all sub-assemblies and then design each using the load effect derived from this analysis and modified by multiplying it by the S and M values appropriate to the sub-assembly. It went on to state that as at 1984, this method has not been fully researched and therefore should be used with “prudence”, particularly for buildings over three storeys high. We see the code commentary as irrational as it required the south shear wall to work harder and dissipate more energy due to the high lateral strength of the north wall complex.

Mr Henry considered that the practical significance was that:

> … this reduction of load leads to a corresponding reduction in the reinforcing requirements for the south coupled shear wall... This increased the imbalance in the building because the reduction in loads effectively only applied to the south coupled shear wall and not the much stiffer and stronger north core [complex].

Mr Henry thought the earthquake load on the whole building would have been largely governed by the yielding of the south shear wall. He said that once it yielded the system would essentially be limited to the load at which yielding occurred and the building would rotate about the north wall complex with any application of a higher load level. However the non-linear time history analysis (NLTHA)[2] results show that this rotation about the north wall complex was partially reduced by the torsional mass inertia of the floors.

The building deflections are set out on pages S15 and S16 of Mr Harding’s original calculations. It is unclear if the inter-storey deflections are from the modal or equivalent static analysis. Mr Harding said in evidence they appear to be for the equivalent static method. Accordingly K was equal to 2.0 as set out in Clause 3.8.1.1 of NZS 4203:1984. He said the wall shear for a dynamic analysis is typically lower than for the equivalent static analysis so it was assumed at this time that these deflections were also lower and therefore not recorded in the calculations.

Mr Henry did not see calculations that showed that the rotation of the building had been taken into account to determine the maximum deflections at the south corners. He said it is likely that the calculated deflections were for the centre of mass and a separate calculation was needed to determine the deflections at the corners. Mr Harding accepted that he did not check corner deflections in the calculations. This meant the deflections were likely to have been underestimated.

The calculations show the elastic inter-storey deflections were multiplied by a $^{K}/_{SM}$ factor to give the maximum inter-storey deflection, with structural type and material factors both equal to 1.0. Mr Harding did not adjust his deflections when he changed the structural type factor for the south shear wall. An S factor of 0.8 would give greater inter-storey deflections when using the $^{K}/_{SM}$ factor.

There were also issues with the approach to the calculation of deflections in NZS 4203:1984. These are discussed in Volume 2, Recommendation 60 and in section 6.2.5.3 of Volume 4 of our Report. When a plastic hinge is formed at the base of the wall, it will rotate about the plastic hinge. This will result in greater inter-storey deflections for the lower levels than the scaled elastic deflections.

In summary, the critical inter-storey deflections that were imposed on the columns were not calculated in the more critical corner locations and did not appear to account for a revised structural type factor. We conclude that the deflections considered in Mr Harding’s design were not consistent with the design requirements in NZS 4203:1984.

6.2 Landsborough House

6.2.1 Introduction

The structural calculations of Landsborough House were essentially used as a guide by Mr Harding when designing the CTV building. Both the CTV building and Landsborough House (see Figure 71) were similar multi-storey shear wall buildings, but there were some significantly different structural characteristics that led to their differing performance during the February earthquake. Some of these major differences are discussed below.

Figure 71: Landsborough House

As discussed in section 2.1.2.2.3, Landsborough House was designed by Mr Henry when he worked at Alan M Reay Consulting Engineer (ARCE) between 1984 and 1985. A building permit was issued on 9 August 1985. It has north-south frames that were designed to support the majority of the vertical loads. Landsborough House has eight floor levels and a floor plate measuring 24.4 by 23 metres (see Figure 73). In comparison, the CTV building had six floor levels, and a larger floor plate, measuring 30.25 by 22.5 metres. The structural characteristics were similar, in that both buildings were:

- described as shear wall protected gravity frame systems;
- torsionally sensitive as the centre of lateral load resistance was eccentric from the centre of mass; and
- lacking full ductile detailing (to the code seismic provisions) in the columns and beam-column joints.

The Landsborough House building survived the February earthquake but was added to the Canterbury Earthquake Recovery Authority's demolition list in January 2012.

6.2.2 Shear wall core and floor connection

Landsborough House had a closed shear core, which was located within the building's floor plate. A closed shear core is more efficient at resisting torsional and flexural actions compared to an open wall configuration. All four shear walls were connected at the corners to provide a stiff system compared to the open and consequently more flexible wall configuration used in the CTV building. Since a closed shear core is stiff it results in smaller inter-storey drifts than an open wall system, if all other structural details are identical. If the walls crack in shear and/or torsion there is a degradation in stiffness. This problem was reduced in Landsborough House by incorporating diagonally reinforced coupling beams in the south wall of the shear core. When the building was subjected to east-west seismic actions it would have induced a direct and torsional shear demand on the four shear walls, as illustrated in Figure 72. The direct and torsional shear demands acted in opposite directions on the north shear wall whereas they acted together on the south shear wall. The result is that the south shear wall sustained higher shear forces than the other walls. The southern wall had diagonally reinforced coupling beams at both its east and west ends. The concept of a coupled shear wall is that the inelastic action takes place by yielding in tension and compression of the diagonal reinforcement, which gives a shear deformation mode. A diagonally reinforced coupling beam does not degrade in stiffness to the same extent as a normally reinforced beam undergoing inelastic cycles. Therefore the core wall system maintains its stiffness during inelastic cyclic behaviour during an earthquake. This is important in limiting the inter-storey displacements.

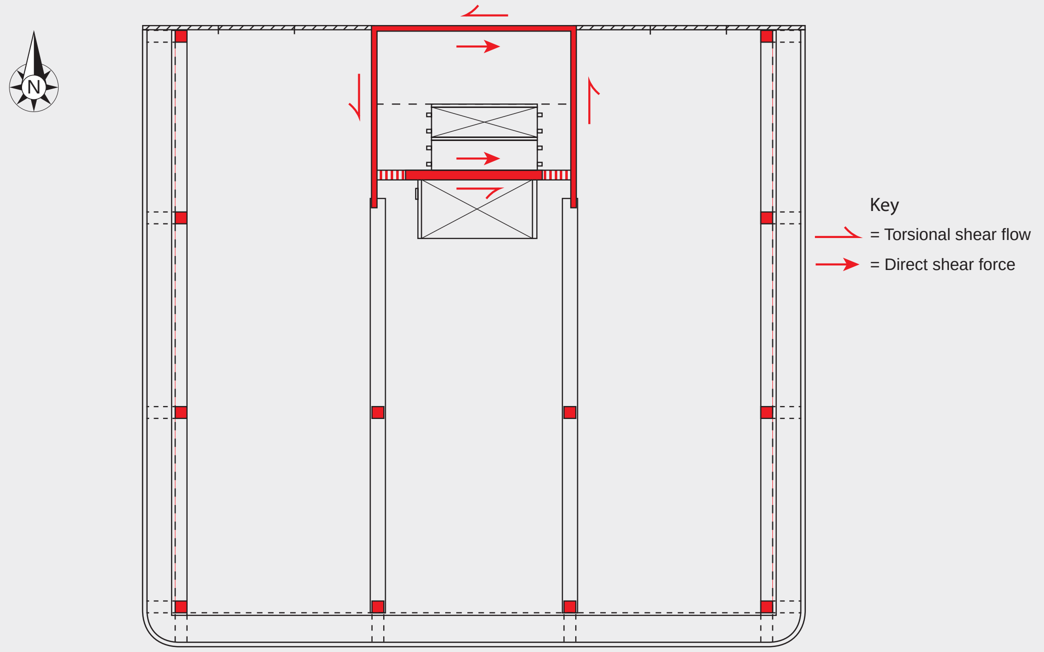


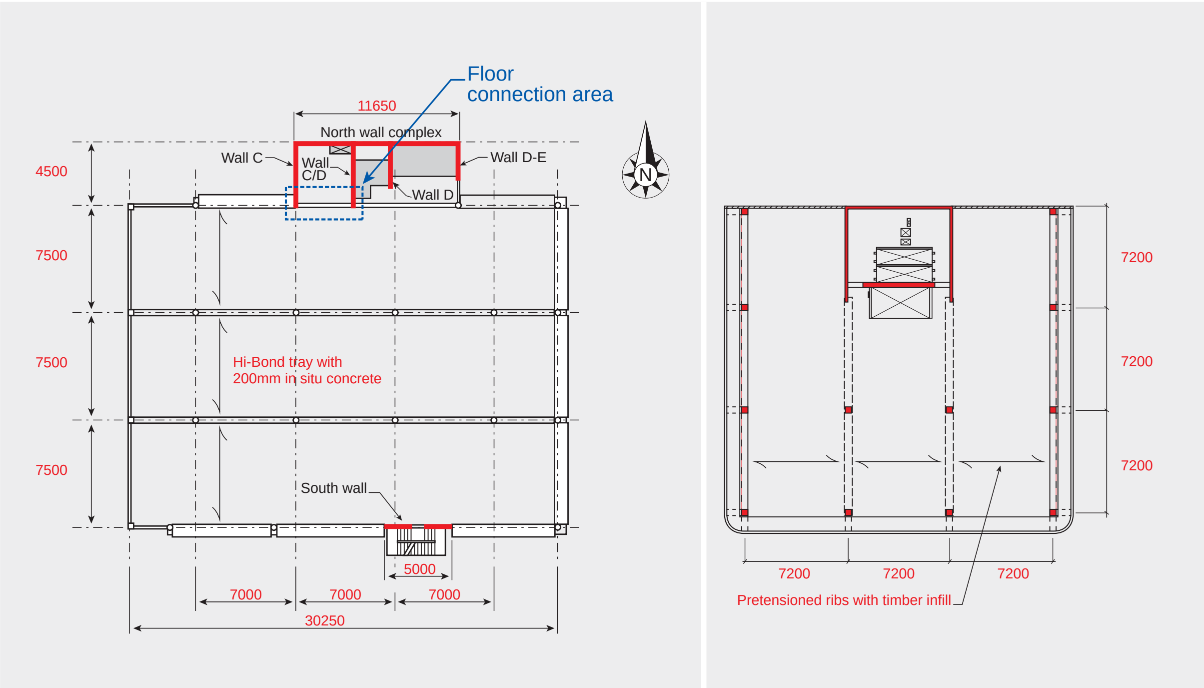
Figure 72: Seismic actions on Landsborough House due to an east-west earthquake

In Landsborough House the beams spanned in the north-south direction and were connected directly into the end of the walls, (see Figure 72). Under north-south seismic actions the inertial forces could be transferred directly into the walls. There was also continuous reinforcement from the floor going into the walls. Since the wall core was inside the building floor plate there was shear transfer capability between the slabs and the walls. Mr Henry described the floor connection to the shear wall core as a "spanner effect" with the floors encompassing the shear core on three sides. The floor reinforcing could act in tension to clamp the floor to the wall sides.

Figure 73 shows the locations of the shear walls and beams in the Landsborough House and CTV buildings. In Landsborough House the centre of mass was highly eccentric to the centre of lateral stiffness. With this arrangement the greatest inter-storey drifts occurred in the east-west direction due to the torsional response of the building. In this structure the beams spanned in the north-south direction and the rotation of the columns (due to drift in the east-west direction) was only restricted by the floor slab at each floor level. As the floor was flexible compared to the columns there was little rotational restraint provided and this reduced the structural actions induced in the columns due to inter-storey drift.

The design of the CTV building can be distinguished from Landsborough House. In the case of the CTV building, the beams spanned in the east-west direction. Modal response spectrum analysis of the CTV building shows that, like Landsborough House, significant inter-storey drifts occurred in the east-west direction due to the torsional resistance of the building. However, in this case the orientation of the beams was such that they restrained the rotation of the columns at each floor, which increased the structural actions in the columns associated with inter-storey drift.

In the CTV building the only area where inertial force from the floors could be transferred to the north wall complex was through the drag bars (installed in 1991) and by flexure, shear and direct tension in the region between the walls on lines C and C–D. The tie force was dependent on the strength of this limited area (shown in Figure 73), which was reinforced with mesh, additional 12mm diameter bars and some embedded beams bars. The arrangement used in Landsborough House was more direct and robust.



(a) CTV building

(b) Landsborough House

Figure 73: Comparison of shear wall layout and floor connection

6.2.3 Comparison of Landsborough House and CTV building

A summary technical comparison of the two buildings is given in Table 1.

Table 1: Comparison of Landsborough House and CTV buildings

Item	Landsborough	CTV
No. of levels	**8**	**6**
Central column dimensions	400mm square	400mm diameter
Confinement	10mm stps. @150c/c	6mm spiral @250 pitch
Column longitudinal reinforcement	4-H28+4-H24 base p_1 = 2.67% 4-H24 top p_1 = 1.1%	6-H20 p = 1.5%
Confinement proportion to gross area	0.43%	0.089%
Confined concrete divided by gross area	0.68	0.585
Cover to longitudinal bars	40mm	50mm
Max. axial load ratio (approx.)	0.3	0.4
Slab	Pretensioned ribs and timber infill, 75 in situ concrete	Hi-Bond with 200mm in situ concrete
Reinforcement	665 mesh and H10 @300c/c over beams	664 mesh and H12 @120c/c over beams

6.2.4 Beam-column joints

The typical beam-column joint in Landsborough House was reinforced with four 10mm diameter square ties as shown in Figure 74(a) and (b). The bars protruding out from the bottom of the precast concrete shell beams were lapped side by side in the beam-column joints with two additional 20mm bars laid in the trough of the shell beam to give reinforcement continuity right through the bottom of the joint.

The columns for the CTV building were reinforced with a 6mm diameter spiral at a 250mm pitch. The structural drawings show this spiral going through the beam-column joints. However evidence given at the hearing by Mr Graham Frost and Dr Robert Heywood suggests that no spiral reinforcement was installed in the joint zones. The CTV building typically had two 28mm bottom bars bent up and anchored in the middle of the connection zone with effectively no beam-column joint zone shear reinforcement. In comparison, the Landsborough House joint detailing would give better performance in an earthquake. The CTV joints would quickly degrade in strength with the hooked bottom bars vulnerable to pull out as described in section 6.3. The basic mechanism of shear transfer in the Landsborough House beam-column joints is shown in Figure 74(c). The shear forces lead to diagonal compression and tension stresses in the joint core, with the latter giving diagonal cracking. To prevent shear failure by diagonal tension under cyclic loading, usually along a potential corner-to-corner failure plane, both horizontal and vertical shear reinforcement is required. In Landsborough House the continuous beam bars allowed a diagonal compression strut to be mobilised across the full width of the joints. The progressive cracking along with some slip of the beam bars would have led to a loss in joint strength if there was significant cyclic inelastic demand placed on the beam or column reinforcement. However, the high stiffness of the shear core in Landsborough House protected the beam-column joints from this inelastic deformation. In contrast to this, the CTV joint zones were sensitive to strength degradation in situations where the bottom bars were subjected to tension. In this situation, due to the lack of continuous reinforcement through the joint zone, its performance was dependent on the tensile strength of the concrete, which was unreliable and, once it cracked in tension, strength degradation would occur rapidly under cyclic loading conditions.

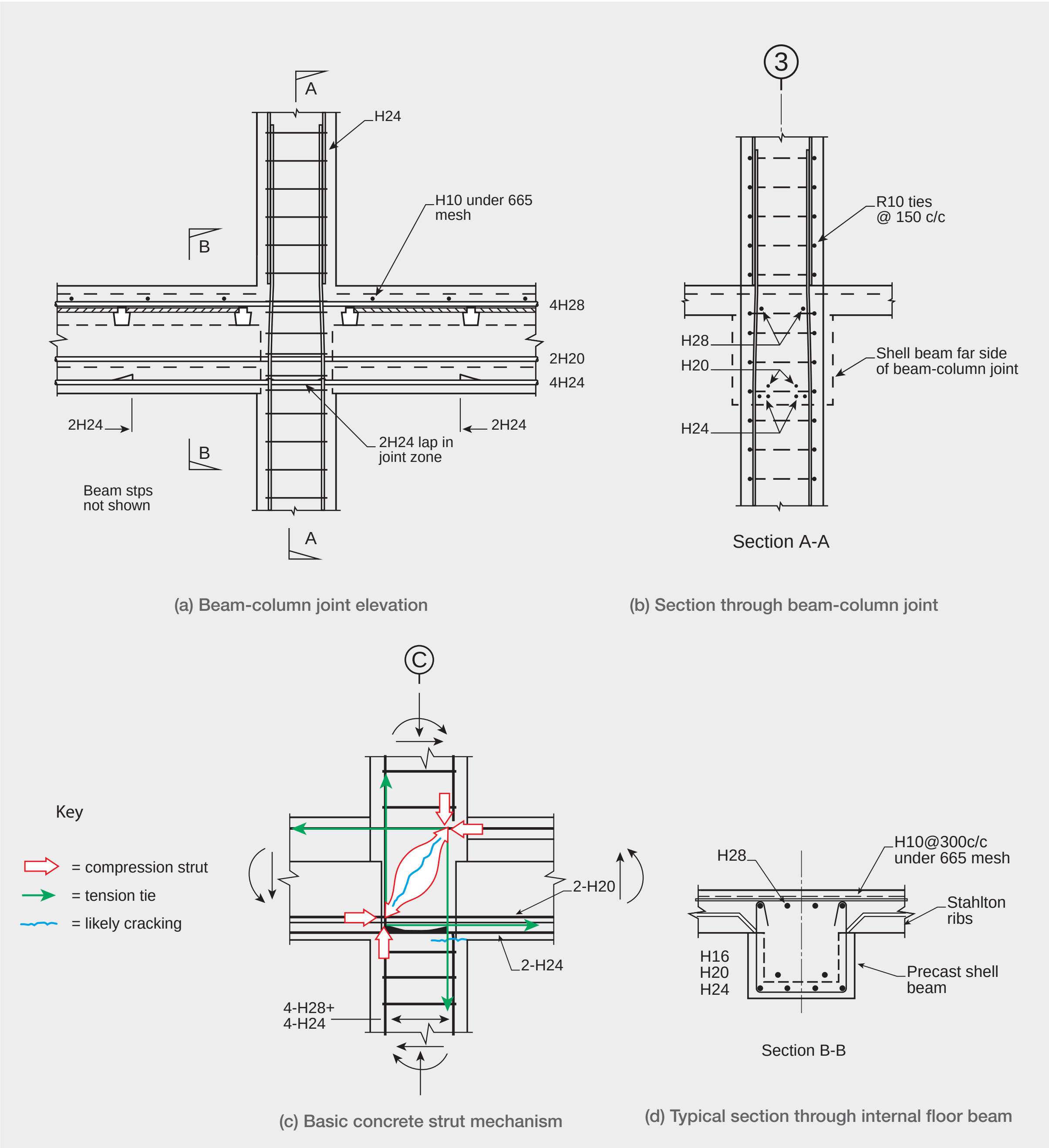


Figure 74: Beam-column joint detailing

6.2.5 Columns

One criterion for the performance of columns is the ratio of the area of confined concrete against the area of unconfined concrete. The bigger the area of unconfined concrete the greater the drop in load capacity when spalling occurs. The ratio of confined concrete to gross area of the column was 0.68 in Landsborough House and 0.585 in the CTV building. Landsborough House had a larger ratio of confined concrete, which would give a superior performance after spalling occurred.

The Landsborough House columns were confined with 10mm ties at 150mm spacing at the top and bottom of the column height, with 250mm spacing in the central region. This would not have been consistent with full ductile seismic requirements of the code (had they applied). However the detailing of the columns does show an increased reinforcement content in the potential plastic hinge regions. The confinement reinforcement in the potential plastic hinge zones increases column ductility and this enables it to sustain increased inter-storey drifts before failure. The proportion of confinement reinforcement to the gross concrete volume in the potential plastic regions was 0.43 per cent in Landsborough House. The corresponding proportion in the CTV columns was 0.089 per cent, which is one fifth of that in the Landsborough House columns. Dr Reay agreed that this made the Landsborough House columns more robust.

6.2.6 Concrete floor

In Landsborough House, precast shell beams supported the precast prestressed Stahlton ribs with timber infills and a cast in situ concrete slab on top. The CTV building had a steel Hi-Bond floor system with total depth of 200mm in situ concrete. Once the concrete hardened the floor became a composite concrete-steel member. In this type of floor, creep and shrinkage of the concrete over time results in stress redistribution and sagging of the floor between the supporting beams. In Landsborough House the creep of the prestressed beams may have had a tendency to lift the floor, although Dr Reay disagreed with that based on his practical experience.

The proportion of slab reinforcement in Landsborough House was also higher since it had a thinner concrete slab than in the CTV building. The lower proportion of reinforcement could have had adverse effects in terms of controlling floor slab cracking in the CTV building.

6.3 The structural system of the CTV building

6.3.1 General description of building

We gave a general description of the CTV building in section 1. As we noted there, the building had six main floors. There was a service floor at level 7 for the lift and other services. There was a lightweight roof at that level.

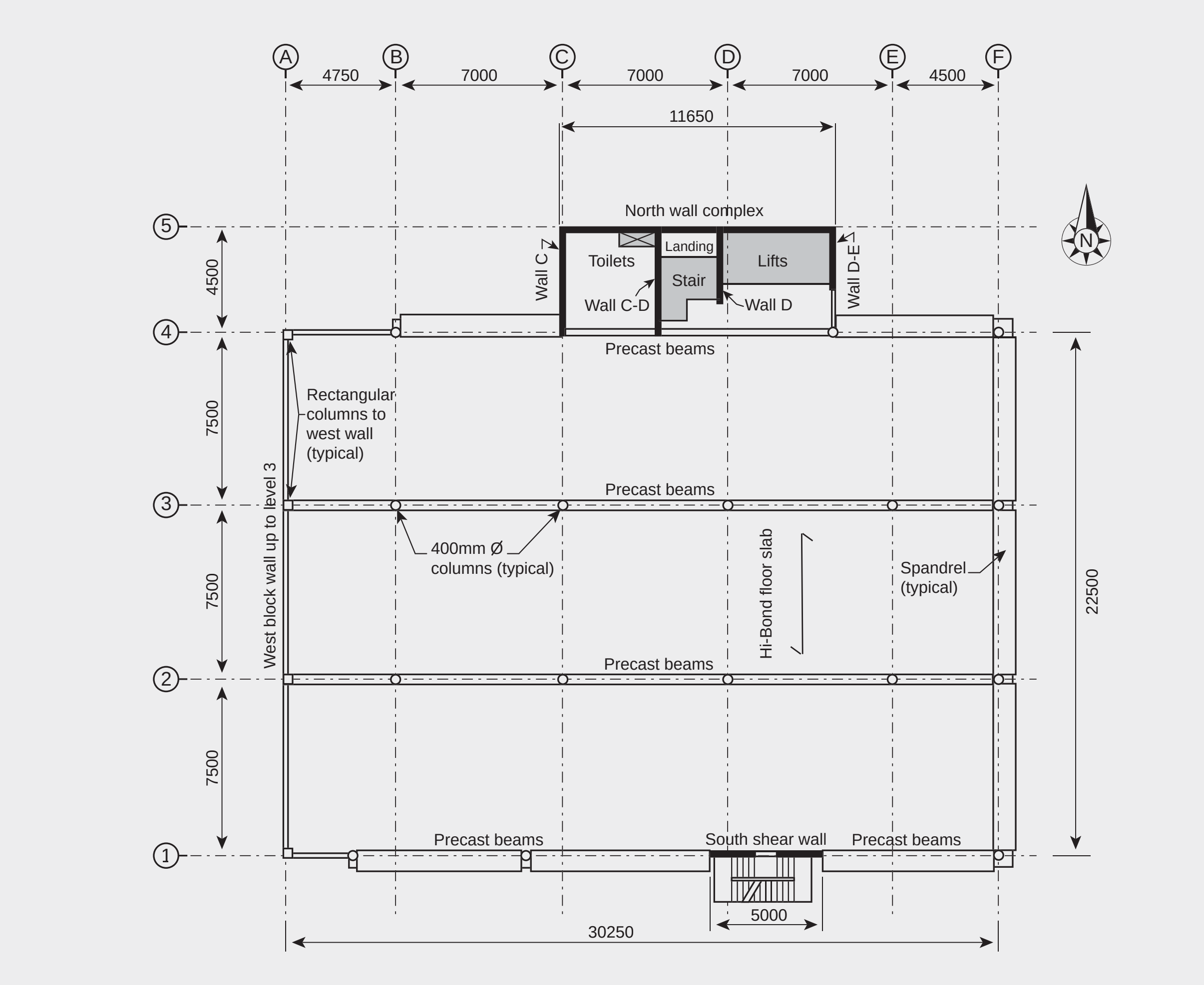


Figure 75: Floor plan for levels 3–6

Figure 75 shows a typical floor plan of the building for levels 3, 4, 5 and 6. Level 2 was similar except for a stairwell opening in the floor slab on the south side of the building. Level 1, at ground level, was a concrete floor cast on grade. The structural wall complex on the north side of the building provided the principal lateral-force-resisting elements for the forces in the north-south direction and approximately half the forces in the east-west direction. It consisted of a wall on line 5, which had a length of 11.65m, and four finger walls at right angles to the main wall that spanned between lines 5 and 4 in the case of the walls on line C and C-D, and part way to line 4 in the case of the walls on lines D and D-E. A coupled shear wall on the south side of the building in line 1 contributed to the lateral force resistance in the east-west direction

There was a block wall on the west side (line A) of the building between levels 1 and 4, which was supported on the ground floor at level 1 and by precast beams at levels 2 and 3. Above this level there were no beams on line A. The lack of damage to this wall prior to the February earthquake indicates that it was either effectively separated from the structure for the September and Boxing Day earthquakes or that it restrained the north-south inter-storey drifts sufficiently to prevent it being damaged to a noticeable extent. However, in the February earthquake it is likely that there was contact between the wall and the surrounding concrete frame.

Precast beams on lines 2 and 3 were supported by columns, and for the beams on lines 1 and 4 by columns and the structural walls. The floors were metal tray, Dimond Hi-Bond, with in situ concrete to give a total depth of 200mm. The clear span between the precast beams that supported the floors was 7.1m, which was close to 500mm in excess of the span recommended by the manufacturer. The in situ concrete was reinforced with 664 mesh. To compensate for the fact that the span length was greater than the recommended value, high strength 12mm saddle bars (380MPa) were added at 120mm centres over the beams. It is understood, though it is not shown on the drawings, that there may have been a second layer of mesh in the concrete, which was draped from above the beams to the middle span of the slab. This was to provide tensile membrane action to support the floor in the event of a fire and loss of integrity of the Hi-Bond tray.

All the columns, except those on line A and a single square 400mm column on level 1 (on grid line 4 between grid lines D and E), were circular and had a diameter of 400mm. The columns on line A were rectangular in section measuring 400mm parallel to line A and 300mm at right angles. All the columns were cast in situ.

All the columns were designed for gravity loading as it was assumed that the shear walls would limit the inter-storey drifts and prevent significant seismic actions being induced in the columns. Confinement reinforcement was kept to a minimum. For the circular columns the confinement reinforcement consisted of 6mm bar spirals at a pitch of 250mm. For the rectangular columns R10 ties were placed at 250mm centre to centre. Due to the low level of confinement reinforcement and the relatively high axial loads the columns would have had limited ductility.

6.3.2 Requested information

The Royal Commission studied a number of reports on the CTV building's structural performance during the earthquakes. During our investigation information was sought on a number of aspects that we considered had not received adequate consideration. A minute was issued dated 27 June 2012 requesting that interested parties respond to a number of questions.

The response to the minute was limited, but what was received is discussed in the sections below. Where we judged the response to be inadequate we have given the conclusions of our own analyses.

The minute from the Royal Commission asked the following questions:

South wall (line 1)

This wall appears to have been designed as a coupled shear wall.

(a) Would this wall have behaved as a coupled shear wall in the Canterbury earthquakes? In particular would the coupling beams have yielded with plastic hinges forming in each of the walls?

(b) What influence would the floors in the building have had on the behaviour of the south wall?

(c) Was there an adequate load path to transmit the inertial forces from the floors into the south wall?

(d) How do the design inertial forces between the wall and the floors compare with the corresponding design actions calculated from NZS 4203:1984 and NZS 1170.5?

North wall complex (between lines 4 and 5)

In this wall complex there are four walls which could provide lateral force resistance in the north-south direction and one wall on line 5 to provide lateral force resistance in the east-west direction.

(a) Given the lateral force resistance in the east-west direction what level of ductility would be appropriate in designing the wall and the inertial forces generated between the wall and the floors?

(b) What was the load path for the shear transfer between the floors and the wall complex?

(c) Would the wall complex warp under the action of this shear transfer? Can you account for the observed vertical cracking in the wall complex?

(d) What other structural actions are associated with shear transfer from floor into the structural wall complex?

(e) Is the detailing of the junction between the floors and the wall complex adequate to resist the shear force and associated actions?

(f) How do the predicted magnitudes of shear force transfer between the floors and the wall complex correspond to the design values found from NZS 4203:1984 and NZS 1170.5:2004?

There was one written response that dealt with design type actions for the south shear wall. Otherwise, the response to the minute was limited in extent, although some comments were made by those giving expert evidence during the hearing.

6.3.3 North wall complex

The north wall complex is shown in Figure 76. The channel-shaped form of the wall complex results in it having a flexural centre (sometimes known as the shear centre) located approximately 0.7m to the north of line 5. However this assumes that the section is fully restrained against warping, which is not the case due to the openings in the floors for the stairs and lift shaft, which are located alongside the wall. To make an allowance for this partial restraint against warping we have assumed that the effective shear centre was 0.5m north of line 5.

6.3.3.1 Assessment of north-south design actions

Some approximate calculations have been made by the Royal Commission to assess the capacity of the connections between the north wall complex and the floors. The seismic weight of each elevated floor (levels 2, 3, 4, 5 and 6) between lines 1 and 4 was assessed as 4900kN (we note this is an approximate value only). For level 6 the parts and portions lateral force coefficient from NZS 4203:1984 is close to 0.25. Under the 1984 Loading Standard (NZS 4203:1984) this coefficient could be used to calculate the design force between the floors and the lateral-force-resisting elements. Use of this value gives a total lateral design force of 1225kN for the north-south direction at line 4. The total north-south design force acting on the north wall complex consists of the 1225kN plus the additional seismic lateral force due to the seismic weight between lines 4 and 5.

Figure 76 shows the north wall complex with voids for the lift shaft and the stairs preventing effective lateral force transfer to the finger walls D and D-E. This was the situation identified by Mr John Hare of Holmes Consulting Group (HCG) in 1990, which led them to inform their client and Alan Reay Consultants Limited (ARCL) of the lack of connection for seismic forces between the floors and the north wall complex. As recounted in section 2.4, ARCL designed drag bars to enable lateral forces to be transferred to the walls D and D-E. A critical section for transfer of this lateral force is just to the south of the beam on line 4. At this location there are 19 high-strength 12mm bars (f_y 380MPa) located in the gap of 3.55m between walls C and C-D. In addition to the reinforcing bars there is 664 mesh in the floor. Allowing for a 45° dispersion of tension force through the beam on line 4, which is connected to the walls C and C-D, gives an effective width of mesh of 5.5m at the south face of the beam on line 4, (see Figure 76(a)). Using a strength reduction factor of 0.9 and basing the tensile strength on the minimum specified yield strength for the bars and the stress at the 0.2 per cent proof strain for the mesh (485MPa) gives a tensile capacity of close to 95 per cent of the design action (1225kN). A further critical section is at the location where the 12mm bars are terminated, which is close to 1.75m from the face of the beam on line 4. Allowing for further 45° dispersion of this force to this section gives an effective width of 9m, which contains 664 mesh (see Figure 76(a)). The tension capacity at this section is 723kN, which is close to 60 per cent of the design action. Due to the limited out-of-plane strength and flexibility of the wall on line 5, without the addition of the drag bars, virtually all tie force in the north-south direction would have been resisted by the walls C and C-D. Each drag bar at level 6 had a design capacity of 300kN, which gave a total capacity approximately equal to the design lateral force at this level.

Similar, though more detailed, calculations were carried out by HCG in their assessment in 1990, and separately by Mr Geoffrey Banks at ARCL in designing the retrofit drag bars. Due to the limited out-of-plane strength and flexibility of the wall on line 5, without the addition of the drag bars, virtually all tie force in the north-south direction would have been resisted by the walls C and C-D. The lateral force coefficients from the parts and portions criteria in NZS 4203:1984 reduced for the lower floor levels in the building. On this basis Mr Banks found that drag bars were not required for levels 2 and 3. For these levels it was deduced that the design forces nominally resisted by these walls could be redistributed to the walls C and C-D with the torsional component resisted by the walls on lines 1 and 5.

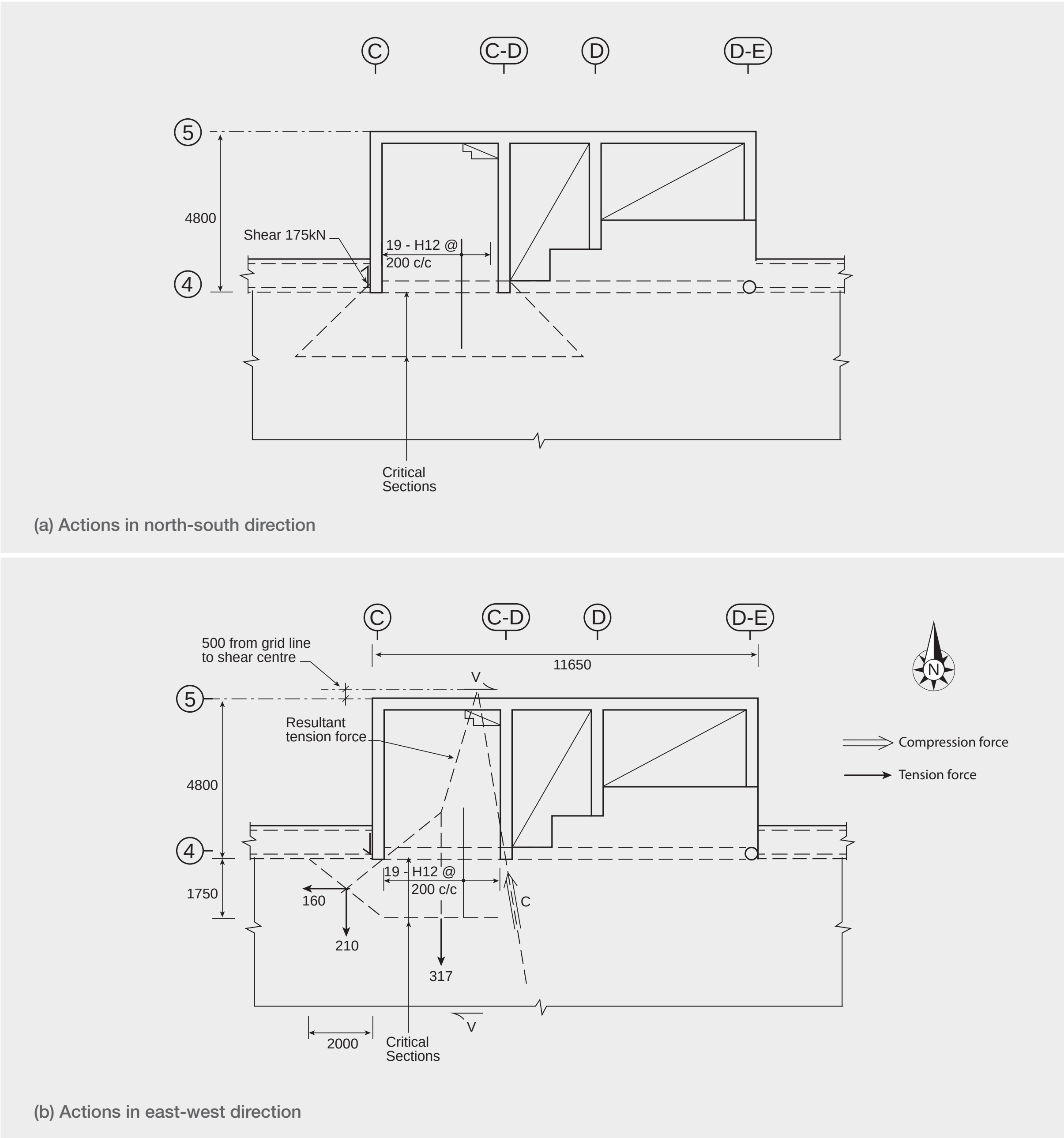


Figure 76: Connection of the floor at level 6 to the north wall complex

The second column of Table 2 has a list of the design tie forces for the different levels of the building in the north-south direction at line 4, as assessed by the Royal Commission using the parts and portions provisions in NZS 4203:1984. These values can be compared with the averaged peak forces predicted in the non-linear time history analyses[2] for the September and February earthquakes. For the September earthquake the averaged predicted peak forces from the non-linear time history analyses at each level are for the recorded ground motions at the CCCC and CBGS stations. These values are listed in column 3 of the table and they are the average of the values for the forces to the north and to the south for both ground motions, giving an average of four values. In column 4 the corresponding values have been calculated for the February earthquake. However, in this case the predicted values from the non-linear time history analyses are for the ground motion records at the CCCC, CHHC, CBGS and REHS stations and consequently they are the average of eight values.

The table shows the predicted tie forces between the floors and the north wall complex do not appreciably decrease with the height of the floor, as is implied by the design forces calculated from NZS 4203:1984. For the September earthquake the tie forces in the lower three levels are approximately 70 per cent higher than the design values. For the February earthquake the predicted tie forces in the upper two levels are approximately 50 per cent greater than the design actions and the corresponding value for the lower three levels is 160 per cent.

Table 2: Connection forces (kN) of floors to north wall complex in north-south direction

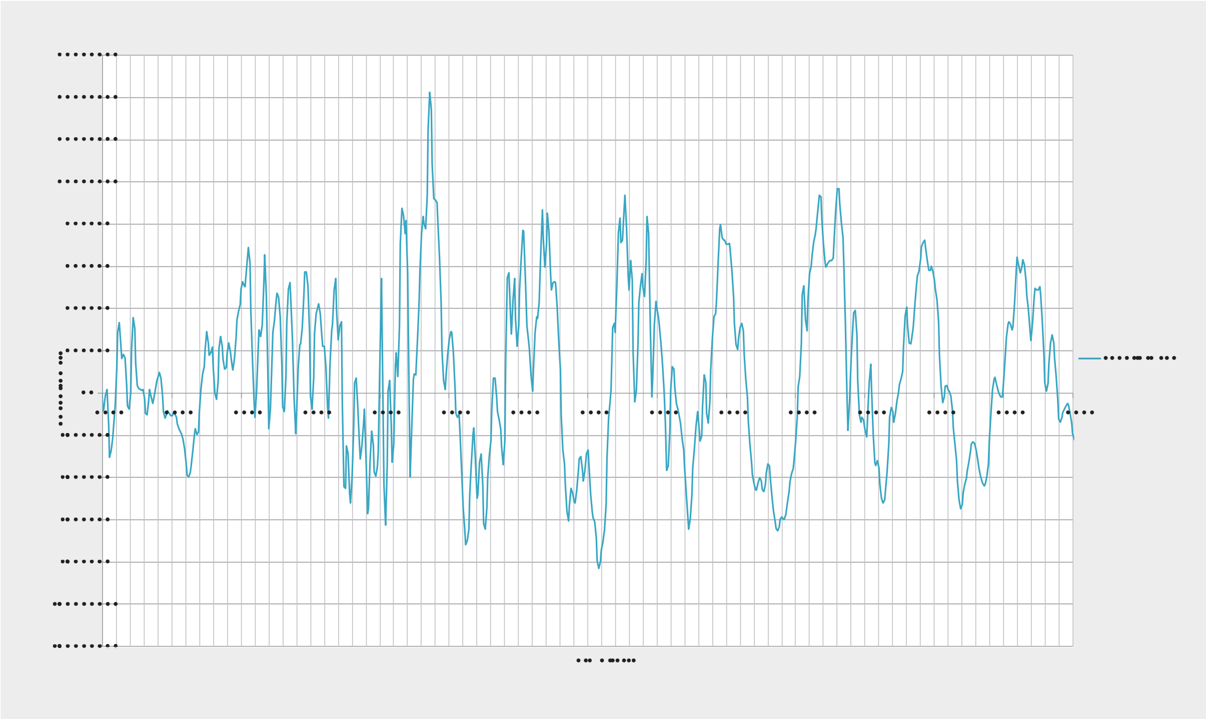
Level	Design forces NZS 4203:1984	4 September 2010	22 February 2011
6	1225	1150	1760
5	1000	1410	1570
4	750	1290	1820
3	750	1500	2150
2	750	1060	1900

Figure 77(a) shows the variation of the predicted tie forces between the level 6 floor and the north wall complex in the north-south direction during the September earthquake for the ground motion recorded at the CBGS site. From the variation of tie force with time it appears that there are two different contributions. First, there appears to be a cyclic variation that has a period of the order of one and a half seconds. Secondly, there are a large number of high frequency cycles. The fundamental period given in the non-linear time history analysis report is 1.3 seconds for vibration in the north-south direction. This suggests that the 1.5 second variation in tie force is related to the fundamental period, which may have been increased due to rocking on the foundation and/or stiffness degradation associated with inelastic deformation. The high frequency components are likely to be related to higher mode vibrations. It is evident from the figure that these higher mode components make up a considerable portion of the maximum force that is induced.

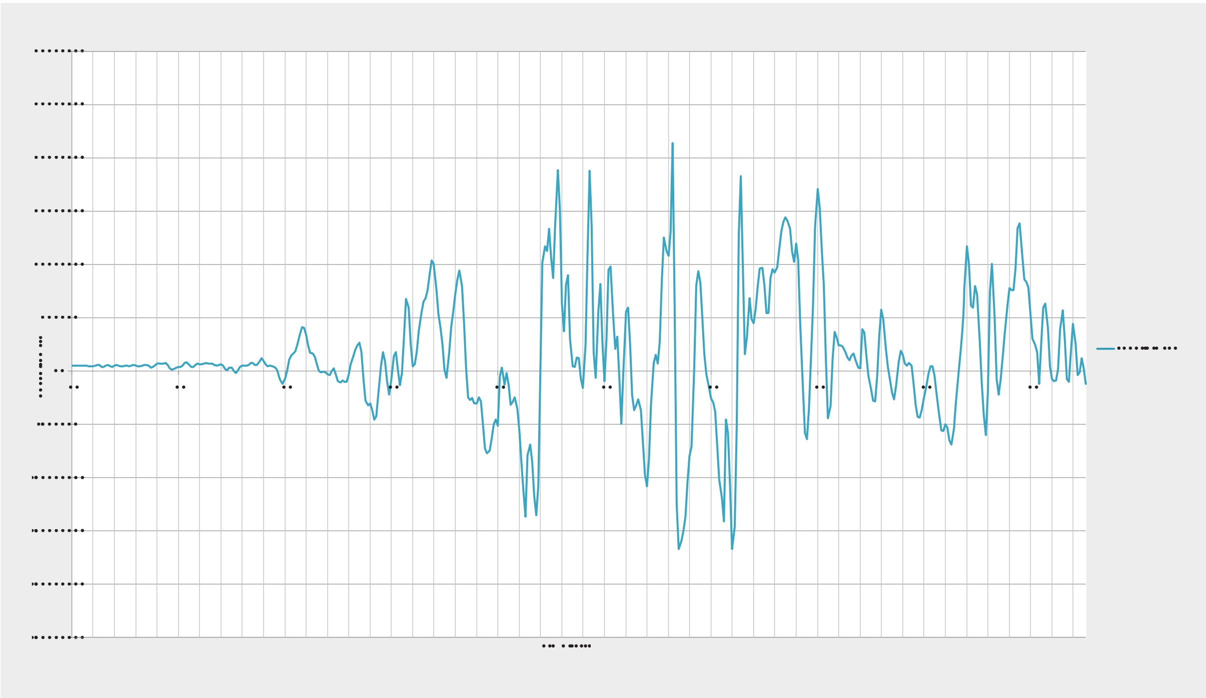
Figure 77(b) shows the corresponding variation of the tie force for the February earthquake. However, in this case the periodic variation corresponding to the fundamental period of vibration is not as clear as it was for the September earthquake. This is likely to be due to the shorter duration of the very much stronger ground motion that occurred in the February earthquake, which would have increased the inelastic deformation compared to the September earthquake. However, the record still shows that there are a large number of high frequency vibrations occurring, which add considerably to the magnitudes of peak force transfer between the floor and the wall complex. In both earthquakes the higher mode contribution to the tie forces is significant.

6.3.3.2 Assessment of east-west design actions

As with the forces in the north-south direction the voids in the floors adjacent to the north wall complex prevent any significant transfer of force from the floor to the wall on line 5 in the zone between the walls C-D and D-E. Consequently the shear transfer between lines 4 and 5 is restricted to the floor slab between walls C and C-D. Allowing for the offset of 0.1b between the centre of mass and the assumed location of the resultant lateral force gives a design shear force at line 4 between the floor on level 6 and the north wall complex of close to 600kN (where b is the width of the building normal to the direction of seismic forces). We have again assessed this force using the parts and portions clauses in NZS 4203:1984. This shear induces bending moments acting in the plane of the slab. Consequently, the floor slab in the location close to line 4 and the walls C and C-D needs to be capable of resisting simultaneously a shear force of 600kN and the associated bending moment (equal to the shear force times the distance from line 4 to the shear centre of the north wall complex). The situation, and a method of assessing the capacity of the floor slab to resist these actions, is illustrated in Figure 76(b).



(a): September earthquake



(b): February earthquake

Figure 77: Predicted tie forces between the level 6 floor slab and the north wall complex at line 4 for the CBGS ground motion records in the north-south direction in (a) the September earthquake and (b) the February earthquake

As noted previously, the effective shear centre has been assumed to be located 0.5m to the north of line 5. For any chosen free body the bending moment and shear at a critical location can be considered to be equivalent to two resultant forces balancing the shear force of 600kN in the wall, as illustrated in Figure 76(b). Equilibrium requires that these two forces intersect the force resisted by the north wall complex at a single point.

To assess the strength of the connection between the floor and wall the critical sections have to be selected. One of these is along the south side of the beam on line 4 (see Figure 76(b)). With the shear force resulting from the floor moving to the west, the wall on line 5 applies a restraining shear force to the east. The bending moment acting at line 4 induces a compression force, which would act on the end of the wall C-D (as shown in Figure 76(b)), with a tension force acting in the slab. The beam framing into the wall C on the western side has some capacity to resist lateral forces due to the dowel resistance of the top bars that pass through the wall. This dowel capacity is assessed as approximately 175kN (using the current Concrete Structures Standard, NZS 3101:2006[3]). The compression force in the slab near the end of the wall C-D would have prevented the reinforcement in the slab in this location from contributing to the tension force. To allow for this the tension capacity of the reinforcement within 0.5m of the wall C-D has been assumed to be ineffective.

Based on these assumptions the design bending moment capacity of the section at line 4 is approximately equal to 2200kNm, which corresponds to a shear force in the wall of 440kN. This is close to 70 per cent of the design value if the interaction of flexure and shear is neglected.

A separate check for shear is required at the section adjacent to the beam on line 4. The design action shear stress is found (using NZS 3101:1982[4] by dividing the shear force of 600kN by the strength reduction factor of 0.85 and by the area ($b_w d$), where the effective depth, d, is taken as 0.8h, where h is the length of slab (3.55 + 2 x 0.3) from outside to outside of the walls C and C-D, and b_w is the minimum thickness of the slab, which in this case is 150mm). The resultant shear stress is 1.42MPa. As this is not a beam, the shear resisted by the concrete should be taken as zero and reinforcement is required to carry a force of 600kN. The available reinforcement, 664 mesh and 19 x 12mm high-strength bars have a capacity of 1133kN, which exceeds the required design action. The shear stress at this level is not critical. However, as noted above, the interaction of flexure and shear reduces the flexural capacity calculated above. Consequently the design strength of 70 per cent of the design action should be taken as an upper limit.

A second critical section is close to the termination position of the 12mm bars placed over the beam on line 4. The critical section is taken at a distance of 1.75m to the south side of line 4. As the mesh to the west of line C laps into the 12mm reinforcing bars, some of the tension force carried by the mesh can be transferred to these 12mm bars. To allow for this effect the critical section is assumed to extend from the end of the line where the 12mm bars are terminated, to the junction between the slab and the beam on line 4 to a point 2m west of the wall C. This section is shown on Figure 76(b). As in the previous case it is assumed that reinforcement within 0.5m of the compression force is ineffective. Elsewhere it is assumed that mesh crossing the critical section is stressed to its design level of 485MPa (0.2 per cent proof stress). The resultant tension forces carried across the different portions of the critical section are combined to give a single resultant tension force, as shown in Figure 76(b). From the resultant forces and their angles, the magnitude of the shear force that can be resisted is found to be approximately 40 per cent of the design value of 600kN. It should be noted that this assessment ignores any contribution that may arise from the tensile capacity of the Hi-Bond tray, which may or may not contribute to the strength depending on how effectively the concrete was bonded to the tray.

Both Professors Nigel Priestley and John Mander indicated that it looked as though the floors in the location of walls C and C-D were inadequately reinforced to resist the shear and flexural actions in this region. However, they did not report on any numerical assessment that they may have made to support their deductions. Mr Banks stated that he had assessed the flexural strength. The bending moment capacity that he calculated (1800kNm) was approximately 60 per cent of the design action.

Table 3 gives the design shear forces at line 4 between the floors and the north wall complex at each of the levels for the east-west direction. These values may be compared to the averaged predicted peak shear forces found in the non-linear time history analyses[2] for actions to the east and to the west. For the September earthquake the analyses were for the CCCC and CBGS ground motion records. Hence the value recorded in the table is the average of four values. For the February earthquake analyses were made for the ground motion records from the CCCC, CBGS, REHS and CHHC sites. In this case the averaged value in the table is therefore based on eight values.

Table 3: Connection forces (kN) of floors to north wall complex in east-west direction

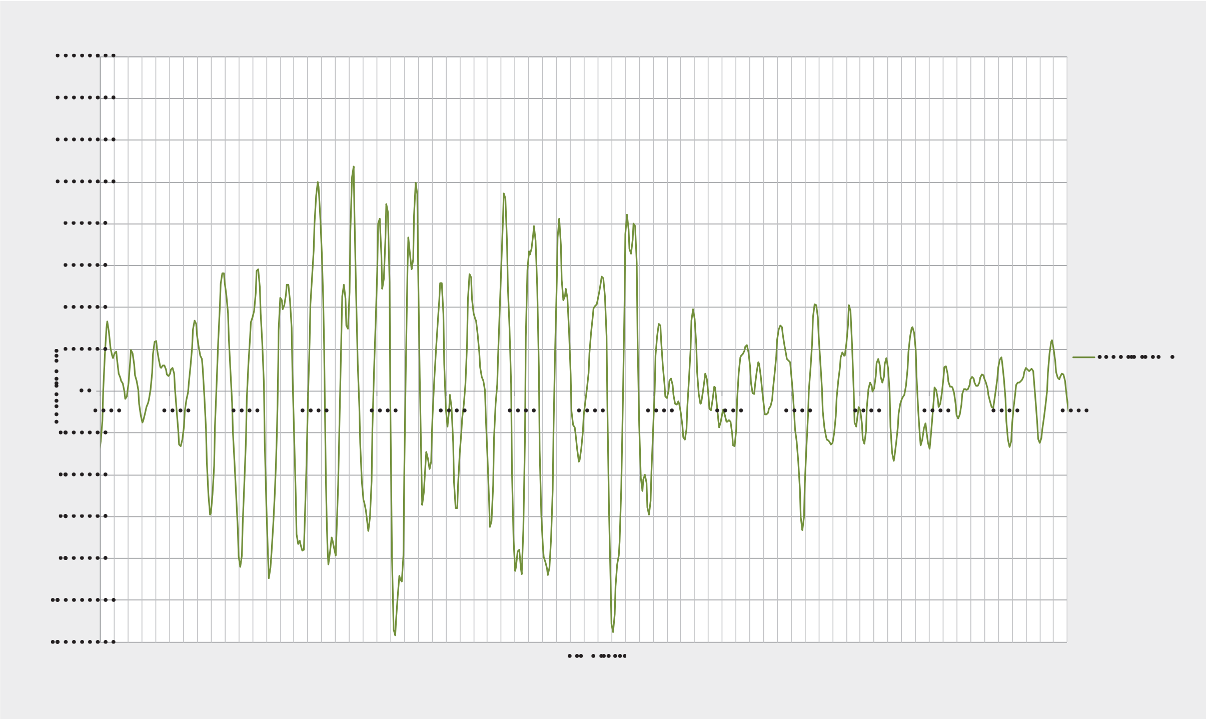
Level	Design forces NZS 4203:1984	4 September 2010	22 February 2011
6	600	1050	2170
5	490	830	1870
4	370	790	1820
3	370	720	1690
2	370	500	1310

It is clear from the table that the peak forces connecting the floors to the north wall complex for the east-west direction were considerably in excess of the design actions. Typically, the September earthquake values were about 1.75 times the design action, while the corresponding values for the February earthquake were four times the design actions.

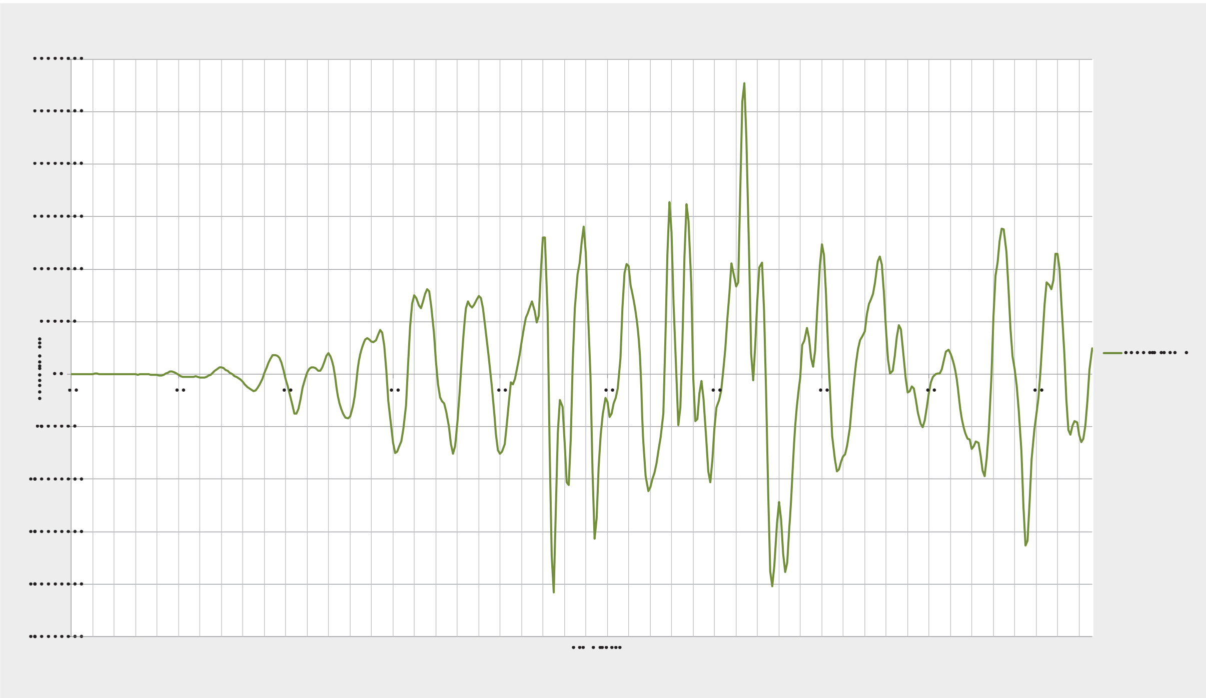
As in the north-south direction the variation in tie forces in the east-west direction for the September and February earthquakes appears to be made up of a cyclic variation in tie force, which appears to have a frequency of about 0.4 seconds, together with further high-frequency components (see Figures 78(a) and (b)). It is anticipated that the frequency of 0.4 seconds is related to the natural period of the north wall complex acting in the east-west direction.

6.3.3.3 General comments on tie forces

From Figures 77 and 78 it is apparent that the high-frequency components of the tie forces between the floors and the north wall complex make up an appreciable proportion of the maximum values. The high frequency components are associated with small displacements; consequently, limited ductile behaviour in the ties could significantly reduce the design magnitude of the tie forces. The drag bars would have had very limited ductile capacity as failure would occur either by yielding in shear of the bolts or by tensile failure of the concrete reinforced by brittle mesh. Ductile behaviour could have been obtained by connecting each of the walls C, C-D, D and D-E to the floors by three high-strength (380MPa) 20mm bars, which could have been anchored into the walls and extended into the floor almost to line 3. If this detail had been used it is probable that separation between the floor and walls would not have occurred even though the strength of these bars would be of a similar magnitude to that of the drag bars.



(a): September earthquake



(b): February earthquake

Figure 78: Predicted tie forces between the level 6 floor slab and the north wall complex at line 4 for the CBGS ground motion record in the east-west direction for the (a) September earthquake and (b) the February earthquake

6.3.3.4 Warping of north wall complex and interaction of wall D-E with the column C18 in level 6

The bending moments induced in the floor due to seismic forces acting in the east-west direction cause the wall on line 5 to rotate relative to line 4. This rotation results in warping of the wall, which is illustrated in Figure 79. This action induces out-of-plane bending moments in the wall on line 5. Lateral forces are also induced in the finger walls D and D-E and in the connections between these walls and the floors. The out-of-plane bending moments in wall 5 account for the vertical cracks which were found by Mr Graeme Smith in the wall on line 5 in the lift shaft.

Between levels 4 and 6 the floors and drag bars restrain lateral movement of the wall D-E. However, no such restraint occurs at level 7, which is at the roof level. At this level the lateral movement of the wall D-E induces lateral displacement and the associated bending moments and shear forces on column C18, see Figures 79 and 80.

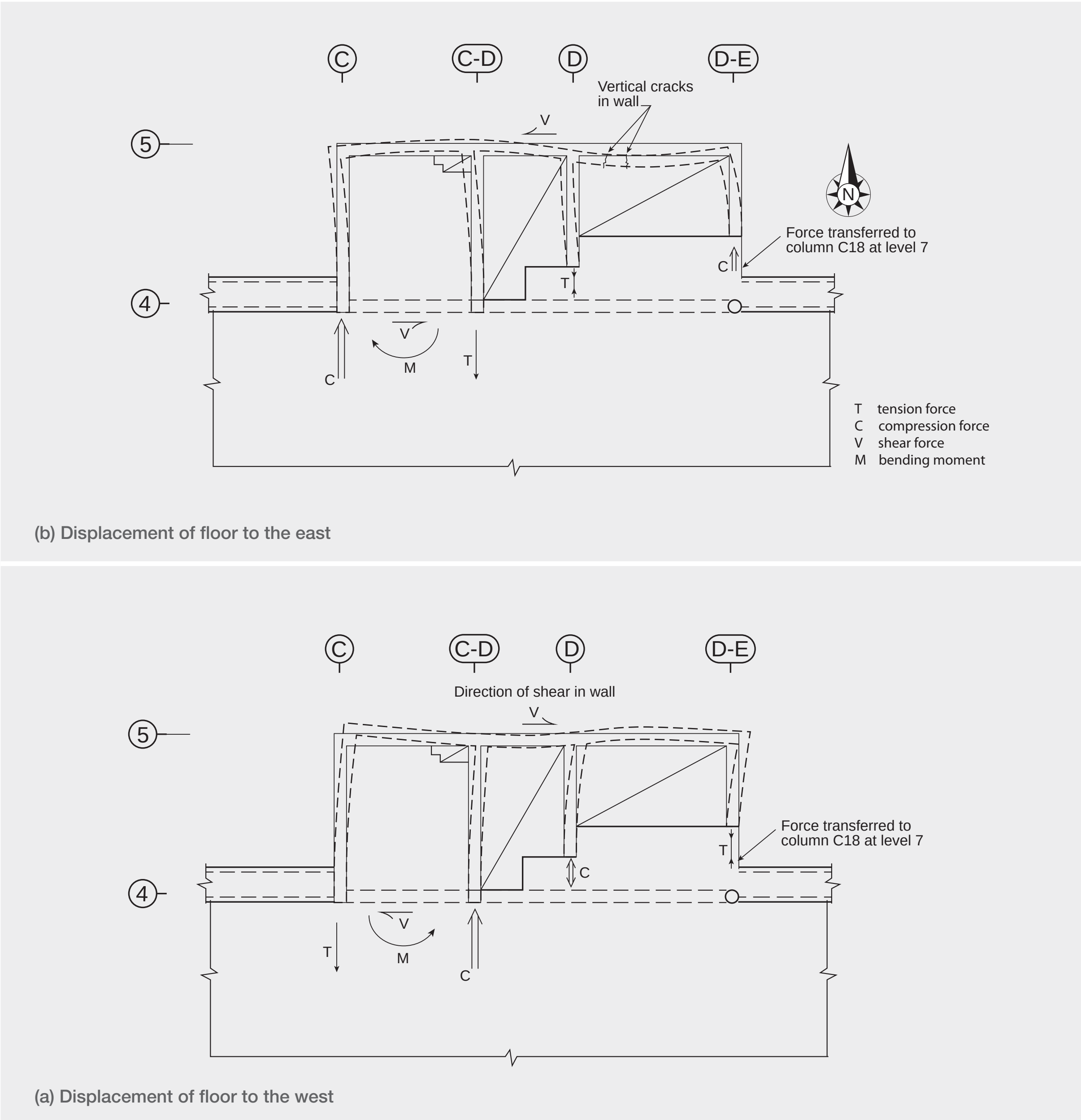


Figure 79: Warping of north wall complex

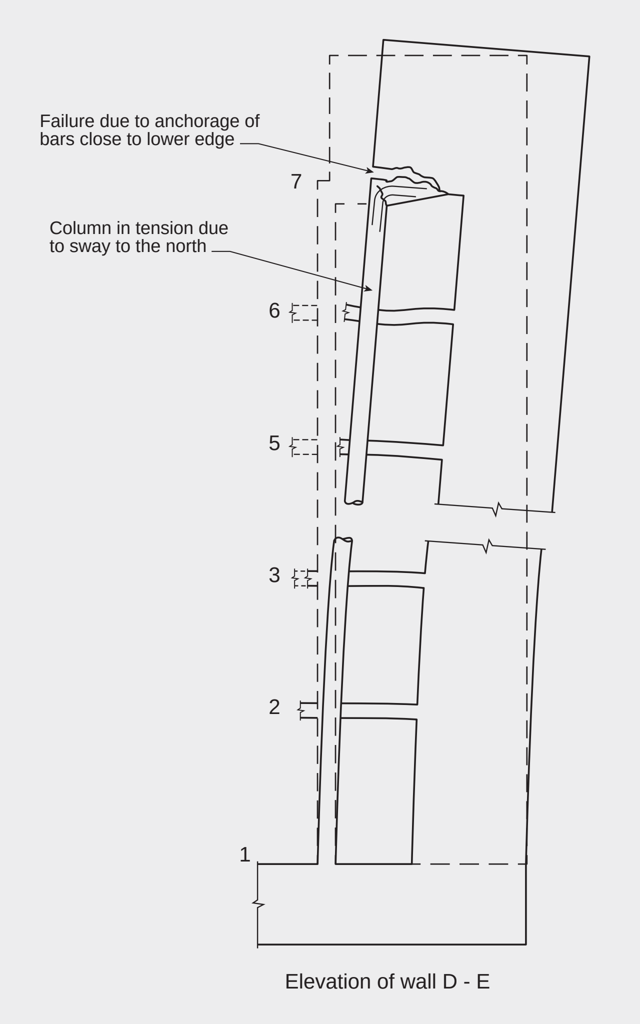


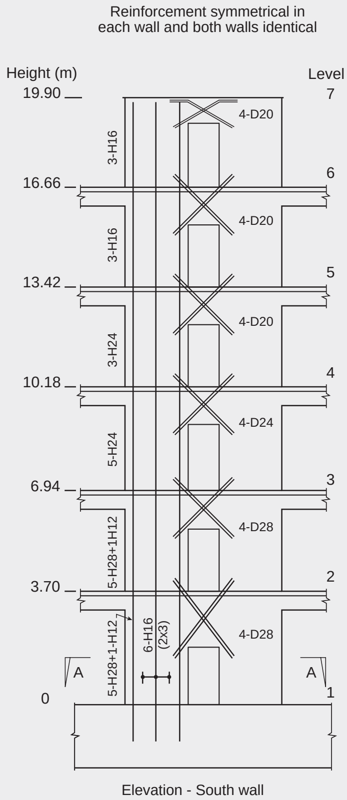
Figure 80: Deflected shape of wall D-E

The wall D-E is tied into the column C18, as shown in Figure 80. Bending of the wall D-E due to seismic actions in the north-south direction would induce axial forces in the column which, combined with the warping action described in the previous paragraph, accounts for the observed cracking in the column C18 on level 6.

The connection between the column C18 and the wall D-E was poorly detailed with the bars from the column being bent into the wall close to the lower surface of the wall at level 7 (see Figure 80). Any appreciable tension in these bars would result in the portion of wall shown in the figure being pulled out of the wall. It is likely that this was the source of the damage observed by Mr Coatsworth and Mr Leonard Pagan after the September earthquake and by Mr Peter Higgins after the Boxing day earthquake. This damage can be seen in Figures 50(a) and (b) in section 3.6.7.2 of this Volume.

6.3.4 South structural wall

The south structural wall was designed as a coupled shear wall as shown in Figure 81. The wall was nearly 20m high and it consisted of two walls each measuring 2050mm in length and 400mm in width, which were connected by coupling beams with a length of 900mm. The gap below each coupling beam was used as a doorway with a height of 2050mm, giving access to stairs for emergency egress on the outside of the building.



Actions on one wall			Coupling beam shears		
(a) Axial load (kN)	(b) Tension force (kN)	(c) Sum (a)+(b) (kN)	(d) Diagonal bars (kN)	(e) Floor slabs (kN)	(f) Sum (d)+(e) (kN)
58	(688)[2] 917[1]	746 975	346	171	517
290	(688) 917	(978) 1207	835	413	1248
522	(974) 1490	(149) 2012	1324	655	1979
754	(1318) 2178	(2072) 2932	2028	897	2925
986	(1671) 2884	(2657) 3870	2986	1139	4125
1224	(1671) 2884	(2895) 4108	4109	1381	5490

[1] Maximum possible axial tension force in one wall

[2] Maximum tension force without extensive redistribution of bending moments

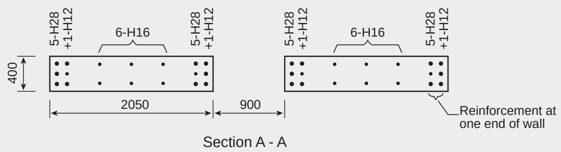


Figure 81: The coupled shear wall on line 1

Coupled structural walls are intended to dissipate seismic energy by forming plastic hinges at the base of each wall and by plastic deformation in the coupling beams. However, photographs and sketches in the Hyland materials report[5] of the wall show less damage to the coupling beams occurred than would be anticipated given that the non-linear time history analyses[2] predicted inter-storey drifts in excess of 2.5 per cent. From the Hyland materials report it is apparent that extensive yielding occurred at the base of the wall, where at least one 28mm bar necked and fractured. In addition there were significant diagonal cracks in the walls close to the base. There were also diagonal cracks in the coupling beam immediately below level 3 and in the walls between levels 2 and 3. There was little apparent damage to the coupling beams above level 3, though there were a few flexural shear cracks in the walls.

The Hyland materials report indicates that many of the beams on line 1 separated from the wall with either the bars anchoring the beam to the coupled wall being necked and fractured, or the beam and bars being pulled out completely from the wall.

The wall can only behave as a ductile coupled shear wall above the level being considered if the sum of the shear capacities of the coupling beams above the level is less than the sum of the axial loads acting above the level plus the tension force that may be resisted in one of the coupled walls at the level being considered. To check this condition the axial load acting on each wall was calculated for the dead and seismic live loads from the tributary areas. The maximum axial tension is limited by the sum of the maximum force that can be resisted by the longitudinal reinforcement in a wall.

However, for all the reinforcement to yield in tension, extensive redistribution of bending moment must take place. This level of redistribution is unlikely to occur. Without this extensive redistribution of bending moment the reinforcement on one end of the wall (see Figure 81) will be in compression or sustaining negligible tension.

The table within Figure 81 summarises calculations made by the Royal Commission on the likely performance of the south shear wall. Two conditions have been considered: first, where all the reinforcement in one wall is at yield in tension, and secondly where all the reinforcement is in yield in tension except for the reinforcement at one end of the wall. The table contains the following items:

a) The sum of the axial loads on one of the walls above the level being considered. These values were calculated from the tributary areas for the dead and seismic live load. Any change in axial load due to vertical ground motion has been neglected as the rapid increases and decreases in axial force tend to cancel each other out.

b) The maximum tension force in the wall at the level being considered based on the assumption that all the reinforcement in the wall is in tension at its design yield stress. The value in brackets is the tension force neglecting the reinforcement at one end of the wall. This represents the case where extensive moment redistribution has not occurred. For example, below level 3 the tension force for this case excludes the reinforcement group of 5-H28 + 1-H12 bars.

c) This column contains the sum of columns (a) and (b).

d) The sum of the shear forces in the coupling beams above the level being considered is given in this column. These calculations are based on the yield forces resisted by the diagonal reinforcement.

e) This column gives an assessment of the contribution to the shear resistance of the coupling beams, which arise from the floor slabs being tied into the coupled walls. This is explained in greater detail in a later paragraph.

f) This column contains the sum of the values in columns (d) and (e) to give the resultant sum of coupling beam shear forces above the level being considered.

As noted previously, the coupling beam cannot yield to any appreciable extent at a level if the value in column (f) exceeds the corresponding lower value in column (c), and significant yielding in the coupling beam is unlikely if the value in column (f) exceeds the bracketed value in column (c). It has been, and still is, standard practice to calculate the shear capacity of diagonally reinforced coupling beams from the forces that can be carried by the diagonal bars acting at yield in tension and compression. With the two sets of diagonal bars the diagonal tension and compression forces are equal and consequently no axial load is assumed to act on the coupling beam. However, this is often not the case.

As shown in Figure 82, once flexural cracks form in the coupling beam elongation occurs. If floors are connected directly or indirectly to the coupled walls they apply lateral restraint to the walls. The restraining tension force induced in each floor applies a corresponding compression force in the coupling beam. Professor Mander agreed that this action would occur. As illustrated in Figure 82(b), this increases the shear capacity of the coupling beam, with the axial force increasing the diagonal compression force in the coupling beam.

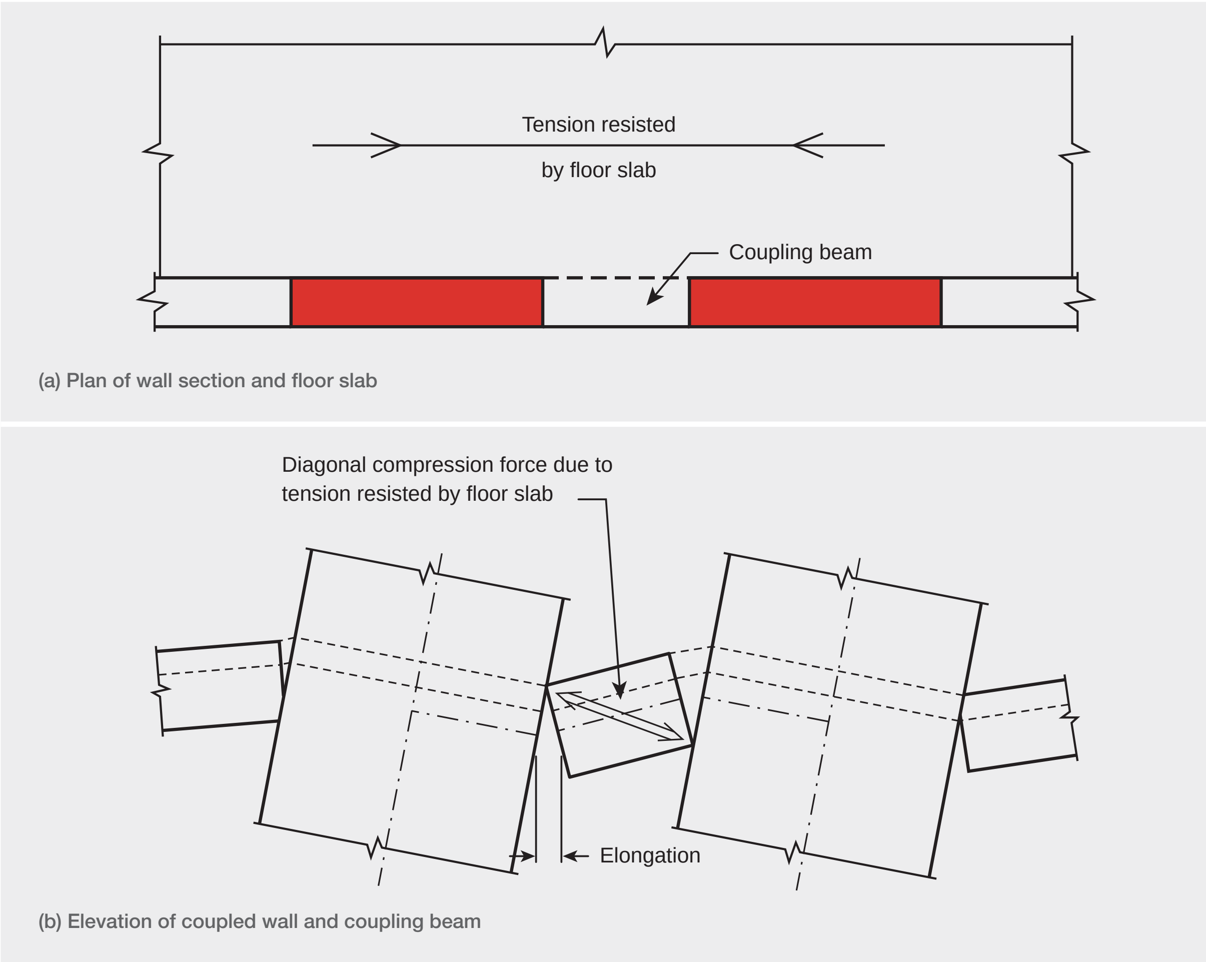


Figure 82: Elongation of coupling beam and interaction with floor slabs

The slab in the east-west direction was reinforced by 664 mesh. Due to this low reinforcement content, when the floor was subjected to axial tension, cracking would have been limited to a single crack, with the reinforcement yielding at this one section. Consequently, due to tension stiffening of the concrete the floor would have acted as a stiff tie with a lateral tie force limited by the yield capacity of the 664 mesh. To assess the significance of the restraint force on the coupled wall it has been assumed that the tie force in the slab is limited to the tension capacity of the mesh in half of the 7.5m span of the slab between lines 1 and 2 (a distance of 3.75m) stressed to its 0.2 per cent proof stress of 485MPa. Using this force, the added shear capacity of the coupling beams can be assessed by assuming that the diagonal force due to axial load forms at the same angle as the diagonal compression force in the coupling beams.

When a check was made for these conditions at the base of the wall it was found that the sum of the shear capacities of the coupling beams was marginally higher than the axial load and the tension capacity of all the longitudinal reinforcement in the wall. This indicated that below level 2 the coupled wall would act as a single unit. The damage at this level is consistent with this prediction. However, when the criterion was applied at level 2 and above it indicated that coupling beams might yield, but only if extensive redistribution of bending moment occurred.

We conclude that extensive yielding of the coupling beams would not have occurred and the coupled wall would have acted predominantly as a single unit. A limited amount of yielding may have occurred in the coupling beams at levels 3 and 4 and in higher mode actions, but this yielding would have been limited. The consequence of this is that the wall would have been stronger than assumed in the design but it was less able to dissipate energy than implied by NZS 3101:1982. The increased energy dissipation in coupled walls is recognised in the Standard by the use of the structural type factor of 0.8 instead of 1.0 for a ductile wall. In retrospect, the use of the 0.8 was not appropriate for this case. However, there were no criteria in the design standards and text books at the time that would have indicated that the coupling beam strengths were too high.

Our previous Recommendation 47 in Volume 2 is for structural designers to develop a greater awareness of the interaction between elements due to elongation, and for guidance on these matters to be given in the commentary to the Standard. We note that the potential restraining effect of floor slabs on coupled shear walls, as described above, is an example of such an interaction.

6.3.4.1 Assessment of south shear wall design actions

Table 4 lists the lateral design tie forces between the floors to the south shear wall. These values have been assessed by the Royal Commission using the parts and portions section of NZS 4203:1984. As in the case of the north wall complex, the seismic weight of the floor from line 4 to line 1 was assessed as 4900kN. Allowing for the 0.1b offset, which includes an allowance for accidental torsion, 68 per cent of the lateral force due to this mass would act on the south shear wall. The forces relate to the tie actions between the south shear wall and the floor which act across the interface between the wall and the floor. Any contribution due to the weight of the floor, between lines 4 and 5 would have been minimal due to its closeness to the wall on line 5 and the torsional resistance of the walls C, C-D, D and D-E.

The peak connection forces from the non-linear time history analyses[2] are listed in the table in the third and fourth columns. These values were found following the steps described for the corresponding actions for the north wall complex.

The predicted tie forces for the September earthquake are on average 10 per cent higher than the design actions. However, unlike the design actions and the corresponding values for the north wall there is no significant decrease in magnitude with the height of the floor in the building. The corresponding values for the February earthquake are on average 90 per cent higher than the design forces. The predicted magnitudes of the average peak tie forces between the floors and the south shear wall increased as the height of the floor in the building reduced. This is opposite to the trend in design actions and predicted tie forces for the north wall complex. While the parts and portions design forces are higher for the south shear wall than for the north wall complex the predicted values from the non-linear time history analyses are smaller. This is almost certainly related to the longer natural period of vibration of the south shear wall compared to the north wall complex.

Table 4: Connection forces (kN) of floors to the south wall

Level	Design forces NZS 4203:1984	4 September 2010	22 February 2011
6	840	630	1030
5	680	770	1160
4	510	670	1080
3	510	700	1210
2	510	620	1410

6.3.4.2 Connection of floors to south shear wall

In the 5m length of the south shear wall at each floor level there were five metres of 664 mesh and eight H12mm bars to tie the floor into the wall. The design of this interface could be based on shear friction as detailed in NZS 3101:1982 with a friction coefficient of 1.4. The total tension capacity of the reinforcement between the wall and each floor based on the design yield strength of the reinforcing (f_y 380MPa) bars and the 0.2 per cent proof stress in the mesh (485MPa) is 790kN. With the shear friction coefficient of 1.4 and the strength reduction factor of 0.85 the design interface shear strength is 940kN. This value exceeds the maximum design value. If, instead, the critical section is taken in the slab where the 12mm bars are terminated, the strength over the five metre length decreases to 45 per cent of the maximum value. However, adequate additional capacity can be developed by transferring the shear force to the perimeter beams on line 1 provided these beams are adequately tied into the coupled wall.

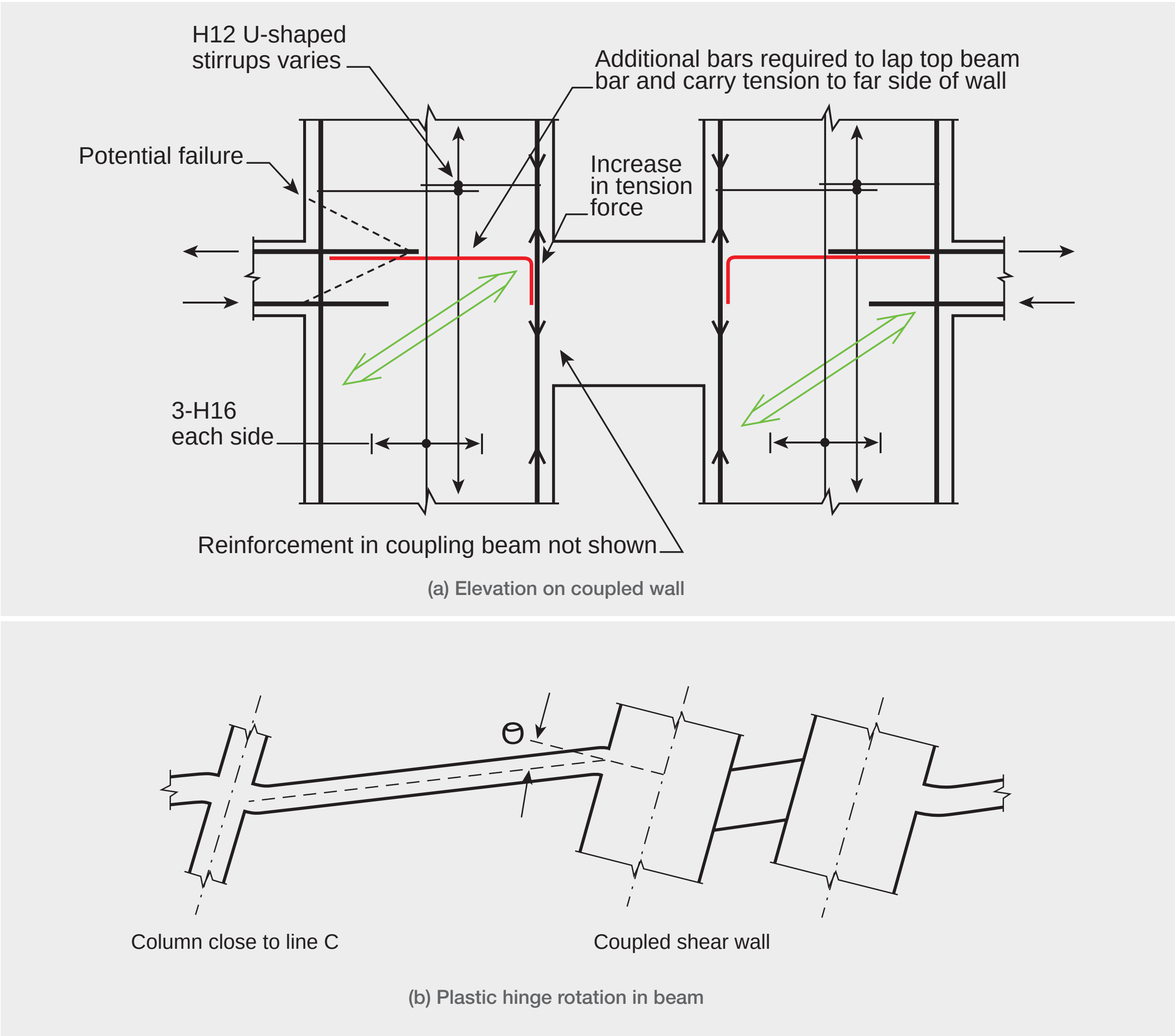


Figure 83: Coupled shear wall and anchorage of beams framing into the wall

The beams on each side of the coupled wall on line 1 are tied into the wall by five 24mm bars, with three at the top of the beam and two at the bottom. The design yield strength of this reinforcement is 380MPa. The span of the beams between the end of the coupled wall and the adjacent columns on line 1 is 7.5m. The flexural strength just meets the gravity load case (of 1.4D and 1.7L). The development length for the top bars meets the specified length given by NZS 3101:1982. However, the bars are anchored in the wall near the location where they entered the wall. The tension force carried by the reinforcement should be carried to the far side of the wall from where it entered the wall. This arrangement enables the tension force to be anchored by diagonal compression forces in the concrete and a vertical force in the wall resulting from a change in the magnitude of either the flexural tension or flexural compression force, as illustrated in Figure 83(a). An alternative to extending the beam bars is to add additional reinforcement, which laps the beam bars and enables the tension force to be carried to the far side of the wall. Without this additional reinforcement there is the possibility that the beam bars will be pulled out of the concrete. Where it is anchored in the wall the beam reinforcement will generate tensile stresses in concrete, which may be expected to initiate diagonal cracks in the concrete.

Figure 83(b) shows the deformed shape of the wall and beams framing into it. The flexural rotation of the coupled wall increases the plastic rotation induced at the end of the beam by about 40 per cent. This added plastic hinge rotation occurs due to the lack of local elastic rotation in the wall relative to the columns, and the increased vertical movement of the end of the beam due to the flexural rotation of the wall. The increased plastic hinge rotation, which is shown in Figure 83, adds to the anchorage problems of the beam bars in the wall.

There is one further problem that arises at the junction of the wall and precast beams. The beams were detailed as 960mm wide and the corresponding wall dimension was 400mm. As the inside faces of the beams and wall were in line there is an eccentricity of 280mm between the two of them, which would result in local torsional actions being induced in the wall and lateral bending in the beam. The high plastic hinge rotations induced with east-west inter-storey drifts and the local twisting action may have reduced the anchorage strength of the bars in the wall. This may have contributed to the observed separation of the beams and coupled shear wall in its collapsed state.

6.3.5 Gravity load system

Typical details of the beams and floor slab (which was constructed by casting 200mm of concrete reinforced with 664 mesh on a Dimond Hi-Bond metal tray) are shown in Figure 84. The clear span of the floors was 7.1m, which, as we have earlier noted, was approximately 500mm in excess of the recommended maximum span (Dimond Hi-Bond technical literature). To compensate for the extra 500mm, 12mm high strength bars were placed over the beams at 120mm centres. There was no direct reinforcement tie between the beams and the floor slab and consequently, in the event of partial loss of support, the beam and associated floor could separate. This detail is not robust and it is possible that this contributed to the progressive collapse of the building.

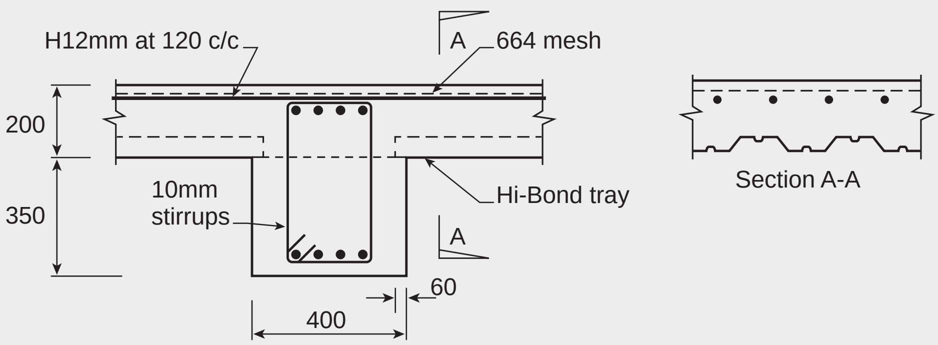


Figure 84: Typical details of floor slab and support beams

Precast beams were supported on cast in situ columns as shown in Figure 85. The columns on lines B, C, D, E and F were circular in section with a diameter of 400mm, and they were reinforced by six high-strength 20mm bars confined with 6mm spiral at a pitch of 250mm. The columns on line A were rectangular in section with dimensions of 400 x 300mm with the 400mm dimension parallel to line A. Ties in these columns consisted of 10mm bars at 250mm centres.

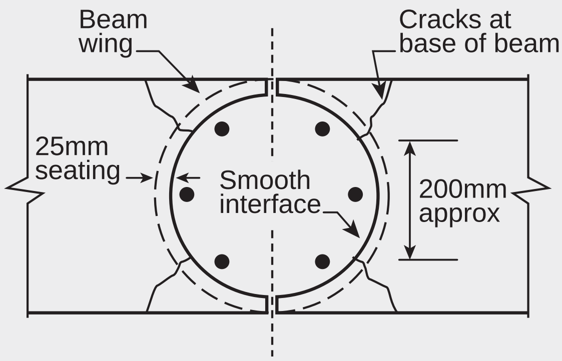


Figure 85: Plan of precast beams supported on an internal column

The ends of the precast beams were shaped to enable them to be supported on the circular columns as simply supported members during construction, as shown in Figure 85. The part of the beam between the circular shape for the column and the edge of the beam is referred to as a wing. There was no reinforcement to tie the wings into the beams. With this arrangement the beams had a 25mm support length on cover concrete. In most of the beams the bottom longitudinal reinforcement consisted of four high-strength 28mm bars. Two of these bars were terminated just short of the columns and the other two were hooked so that, when the columns were cast, they were anchored in the in situ concrete. These details are shown in Figures 87, 88, 89 and 90. In some of the shorter beams a lesser amount of reinforcement was used in the precast beams. However, in all cases the bars that extended into the beam-column joint zone were hooked in the in situ concrete.

As discussed in section 5.1, inspection of the CTV site after the collapse, and of the building debris at the Burwood landfill where it was stored, showed that in all cases the wings of the precast beams had broken off (see Figures 85 and 86). Inspection of the ends of the beams showed that the interface between the precast beam and in situ concrete in the column was smooth, with no adhesion between the precast and in situ concretes (see Figure 86). Mr Frost suggested that the wings could have broken off due to radial pressure between the column and the end of the beam (see section 5.1.3.3). Cracks may have been initiated by radial pressure between the beam and column due to negative moments generating a flexural compression, applying a compression force across the interface. Alternatively, thermal expansion of the concrete associated with heat of hydration when the column was cast, or the Poisson expansion of the column concrete, associated with the axial load and creep, may have generated pressure that initiated the cracks.

Photographs show that typically when the beam wings had fallen off, the remaining interface between the column and beam had a width of about 200mm.

Figure 86: Photo of end of beam

Figure 87 shows the structural details and potential failure mechanisms associated with an internal beam column support. Parts (a), (b) and (c) of the figure show a plan view and a sectional elevation of the joint zone and a beam section. Of the four 28mm bars at the bottom of each beam only two extend into the joint zone. This reduction in reinforcement close to the column face concentrates any positive moment flexural tension force into two bars which, as detailed, have just under 60 per cent of the development length required for full strength. With this arrangement any positive moment inelastic deformation in the beam could lead to rapid strength degradation of the joint zone.

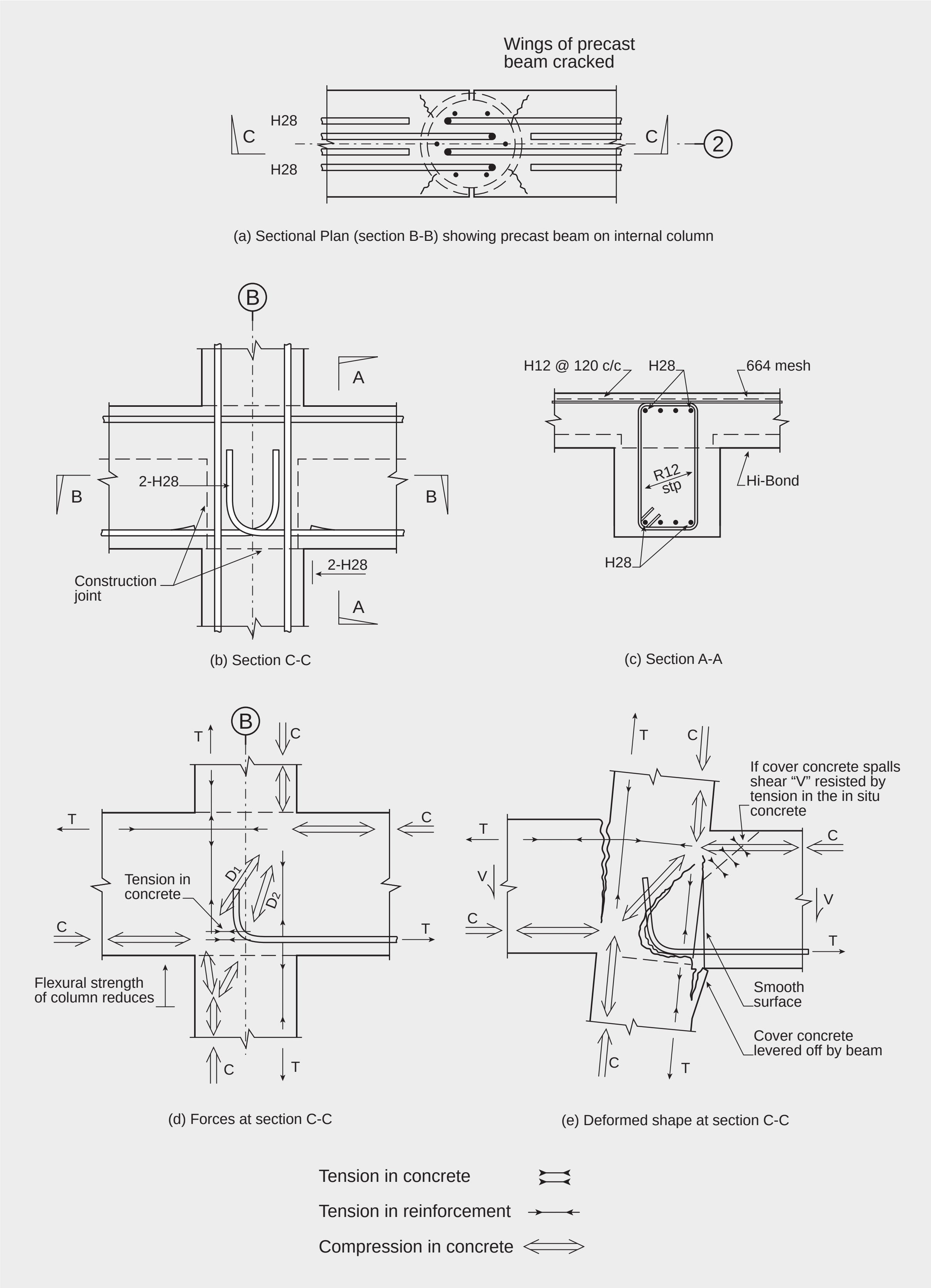
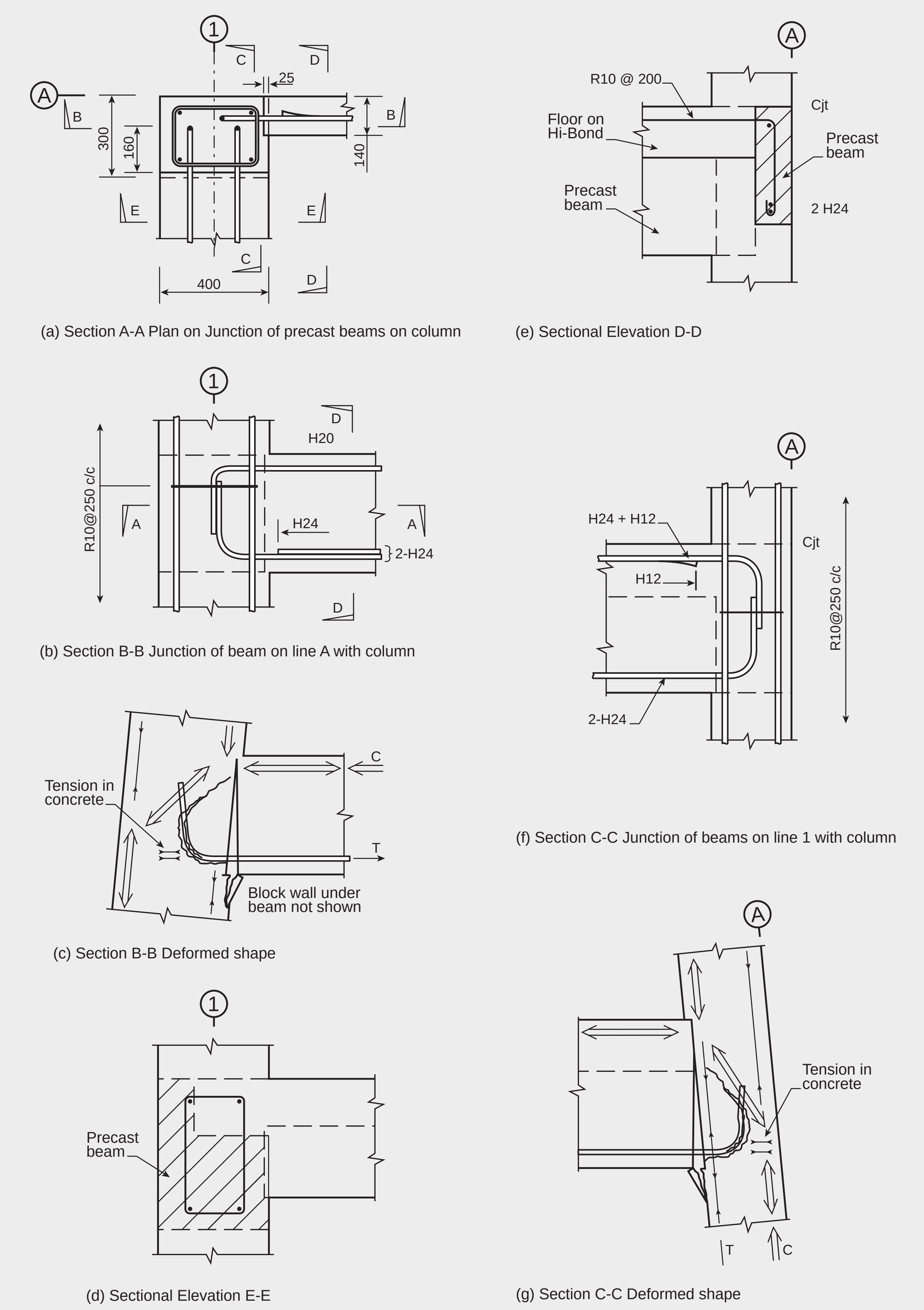


Figure 87: Junction of internal beams and column

Part (d) of Figure 87 shows the track of the forces through the joint zone and the adjacent regions of the beams and columns. Any flexural tension force in the right hand side beam acts on the hooked bars, which extend to the mid-zone region of the joint. Any tension in this bar is resisted by tension in the concrete behind the bar. Once a crack forms due to this tension force the diagonal compression force, shown as D_2 in the figure, can develop from the upper compressed corner of the joint into the hook. To satisfy equilibrium requirements a vertical force is required from the lower column to balance the vertical component of the force D_2. To provide this force the centroid of the compression force in the lower column must move towards the centre of the column (see part (d) of the figure). This results in a reduction of the flexural strength of the column, close to the interface with the beam soffit, of the order of 10–20 per cent. As a result, any inelastic deformation in the column is confined to the joint zone and the immediate vicinity of the column interface with the beam soffit. In effect the joint zone is weaker than the column. Professor Mander came to the same conclusion. As a consequence, the joint zones could be expected to degrade in strength and stiffness before the columns developed plastic hinges.

Part (e) of Figure 87 shows two potential failure mechanisms for the internal beam-column joints.

1. Once the concrete behind the hooked bar cracks in tension there is little to stop the hooked bar pulling out of the concrete as the diagonal compression force “D_2” forms at an angle that is too steep to be effective. The anchorage failure of the hooked bar is associated with the development of a wide crack through the joint zone as shown in part (d) of the figure. This degrades the strength of the joint zone and a reversal of actions could be expected to destroy the joint zone.
2. The beam is supported on a 25mm width of cover concrete. If the wings have broken off, as described previously, the effective bearing area is on a width of 25mm and a length of approximately 200mm (see Figure 85). The gravity load due to self-weight of the slab and beam, the added dead load and a nominal live load results in a contact force of approximately 160kN. The resultant contact average stress due to this load is close to 30MPa, which is at a level close to the failure load. This stress level could have been significantly increased by vertical seismic ground motion. If a positive moment acts on the beam a crack could open up at the smooth interface between the beam and the column. The opening up of this crack to even a small width may result in the cover concrete being levered off from the column, as shown in part (e) of Figure 87. As no shear can be resisted across the smooth crack the shear force is transferred to dowel action of the hooked bars at the bottom of the beam, and to tensile resistance in the in situ slab concrete at the top of the beam. Appreciable deformation is required to develop dowel action and, as one of the bars is relatively close to the free edge of the column, it is unlikely that appreciable dowel action could be sustained. The shear resistance provided by tension in the slab concrete is a brittle action. Consequently, the capacity of dowel action and the tension capacity of the concrete in the slab cannot be added together. The shear force in the beam is resisted by the web, which has a width of 400mm. If the shear force is resisted by tension in the slab an effective width of slab is likely to be of the order of 500mm, which is 100mm wider than the width of the web. Four 28mm top bars pass though this region, reducing the effective width of slab that could resist the tension to about 400mm. Based on this width the diagonal tension stress in the concrete required to resist the gravity load (of 160kN) is of the order of 2MPa. The design direct tensile strength given in NZS 3101:2006 is 1.8MPa for 25MPa concrete and it is used with a strength reduction factor of 0.6. Consequently a direct tensile strength of the order of 2MPa is close to a condition where failure would be expected to occur. If allowance is made for vertical seismic ground motion, the tensile stresses are likely to increase into the range where failure would be expected, and this would result in collapse of the beam. Mr Frost postulated a similar collapse mechanism.



(a) Section A-A Plan on Junction of precast beams on column

(e) Sectional Elevation D-D

(b) Section B-B Junction of beam on line A with column

(f) Section C-C Junction of beams on line 1 with column

(c) Section B-B Deformed shape

(d) Sectional Elevation E-E

(g) Section C-C Deformed shape

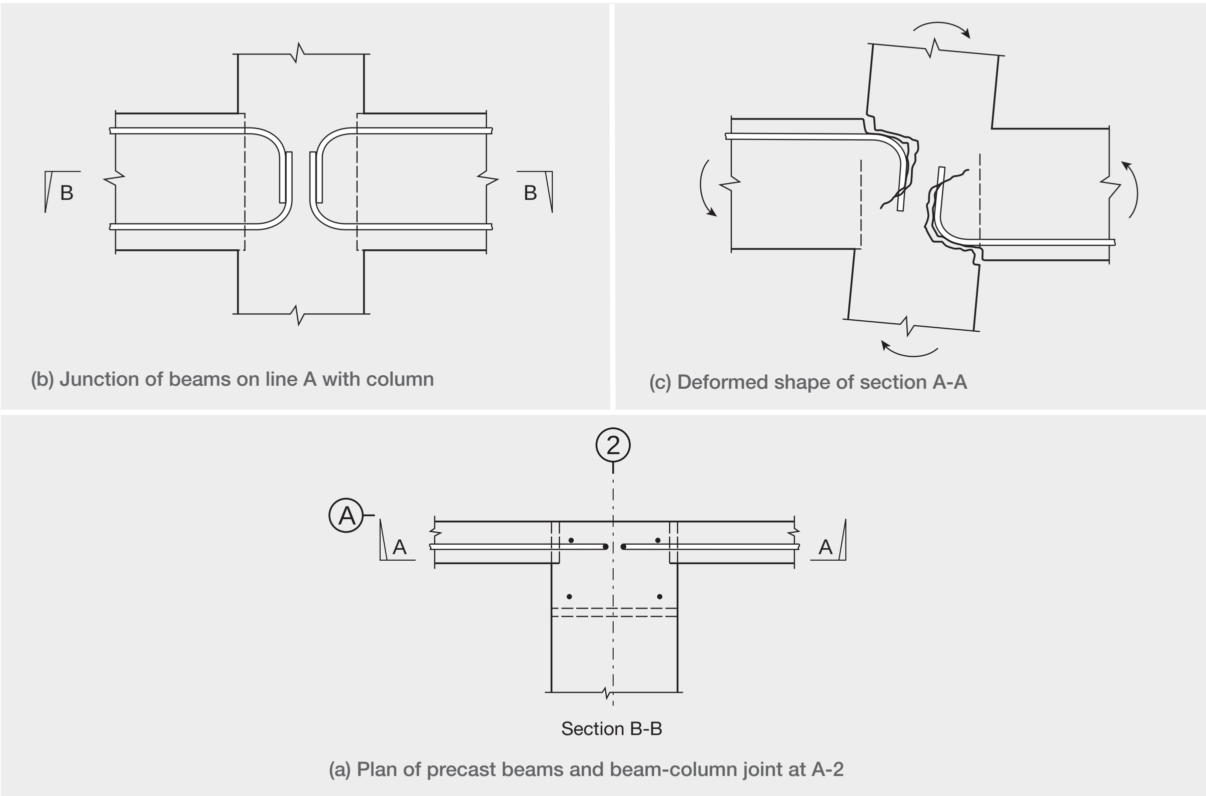
Figure 88: Junction of beam and column at south-east corner of building

Figure 88 shows the details and structural actions in the beam-column joints in the south-west corner of the building, which was at the intersection of lines A and 1. Many of the structural details and associated actions illustrated in this figure apply to the other beam column joints on line A.

Figure 88(a) shows a sectional plan of the intersection of the precast beams with the column. The precast beams on line A were associated with a concrete block wall, which extended to level 4 (above this level there were no precast beams on this line). The drawings show that concrete blocks were tied into the beams by starter reinforcement above the beams on levels 2 and 3; by reinforcement which extended from anchors mounted in the soffits of the precast beams at levels 2 and 3; and from the soffit of the slab at level 4 into the concrete blocks. It is not clear how this reinforcement was placed. The top layer of blocks was not filled with concrete to allow relative lateral movements to occur between the top of the walls and the beams.

The lack of damage to the block wall as a result of the September earthquake indicated either that it was effectively isolated from the structural members for this event or that it restrained movement in the north-south direction. However, in the February earthquake greater inter-storey drifts were induced and the extent to which the wall on line A would have restrained inter-storey drifts before collapse occurred cannot be established with any level of certainty.

Figure 88(b) shows the reinforcement details of the intersection of the precast beams at levels 2 and 3 and the corner column. One high-strength 24mm bar extended from the bottom of the beam into the column where it was anchored by a hook just short of the mid-section of the column. This gave a development length of approaching 60 per cent of that required by NZS 3101:1982. The top reinforcement consisted of one 20mm high-strength bar. In addition there was some limited tensile capacity provided by the mesh reinforcement in the slab, which was tied into the beam by plain round 10mm bars (f_y 275MPa) at 200mm centres. Part (c) of the figure shows the seismic actions and potential failure mode due to drift in the north-south direction. Parts (d) and (e) of the figure show details of the beam sections which are supported by the column. Parts (f) and (g) show the reinforcement details and failure mode at the intersection of the beam in the corner column for seismic actions associated with inter-storey drift in the east-west direction. When significant flexural tension is applied to either the top or bottom reinforcement in the beam, the concrete behind the bar hooks is subjected to tension and it will crack allowing the hooked bar to pull out of the concrete. Clearly there is the potential for either or both of the beams on lines 1 and A to tear the joint zone apart.



(b) Junction of beams on line A with column

(c) Deformed shape of section A-A

(a) Plan of precast beams and beam-column joint at A-2

Figure 89: Junction of beams on lines 2 and 3 with external column on line F

Figure 89(a) shows the intersection of the beams on line A at levels 2 and 3 with the beams on lines 2 and 3. Single hooked bars in the precast beams in line A extend to 15mm short of the column centre-line as shown in part (b) of the figure. The bottom bar in each beam has a diameter of 24mm and the top bar 20mm. Under the action of sway in the north-south direction negative moments are induced in one of the beams and positive moments in the other. Tension would have been induced in the concrete immediately behind the bottom bar, which is resisting a positive moment, and behind the top bar that is resisting a negative moment. The concrete would crack and the joint zone ties of R10 bars at 250mm centres, giving a maximum number of two ties, would not have been anywhere near adequate to prevent the hooked bars pulling the joint zone apart, as shown in part (c) of the figure. Rapid strength degradation would occur. The only feature that could have prevented collapse of these joint zones in the north-south direction would have been sufficient strength and stiffness in the concrete block wall to prevent appreciable inter-storey drift from developing at the columns on line 1. However, we consider that is unlikely to have been the case. It may also be noted that the opening up of the cracks in the joint zone due to sway in the north-south direction would have caused elongation, which would have added to the deformations imposed on neighbouring beam-column joint zones.

The situation described above is particularly critical as the strength of the columns is considerably greater than that of the beams and this would have forced inelastic deformation into the beams. The actions in the joint zone associated with the beams on lines 2 and 3 for inter-storey drifts in the east-west direction are very similar to those described in Figure 88.

Figure 90 shows the structural details and structural actions at the beam-column joints on line F at the intersections with grid lines 2 and 3. Part (a) of the figure shows a plan of the reinforcement in the precast beams and how this reinforcement is anchored into the column. All the longitudinal reinforcement in the bottom of the beams on line F is terminated short of the column centre line. Parts (b) and (c) in the Figure show cross-sections, which identify the locations of the principal longitudinal reinforcement in the beams. The top reinforcement along line F is continuous over the column.

The beam bars from the beams on line 2 and 3 were extended through to near the far side of the column (see Figure 90(b)). With this arrangement the joint zone should be capable of sustaining a few cycles of inelastic loading without loss of axial loading capacity for sway in the east-west direction, provided the joint zone is not degraded by actions associated with north-south sway.

The precast beams on line F were placed so that there was a 20mm gap between the ends of the two beams (see Figure 90(a)). This gap, whether filled with concrete or not, would have acted as a crack initiator, with the crack extending through the column centre-line as shown in part (d) of the figure. Such a crack could be initiated by wind or seismic actions, or it could have been present from shortly after construction due to shrinkage of the concrete due to thermal contraction associated with heat of hydration.

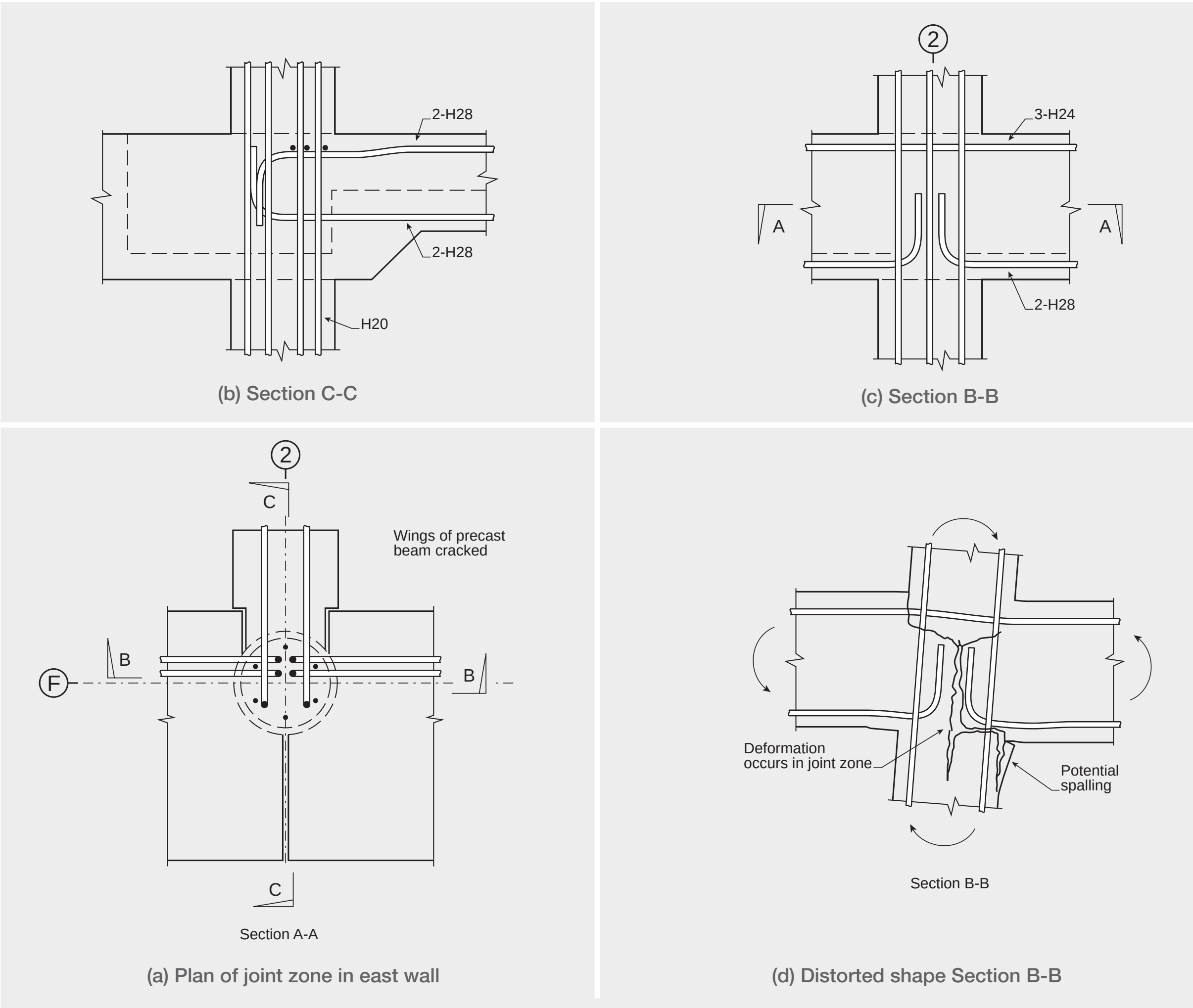


Figure 90: Intermediate beam-column joints on line F

Any tension force resisted by reinforcement in the bottom of the beam could be expected to extend the crack between the ends of the precast units into the column centre line. It may be noted that the column acts to lever the two beams apart and in the process creates a wide crack in the joint zone. This would result in strength degradation and the bars pulling the joint zone apart, as illustrated in Figure 90(d).

6.3.6 Answers to questions sent out to technical witnesses in the minute of 27 June 2012

As mentioned in section 6.3.2 experts giving evidence to the Royal Commission on the collapse of the building were requested to consider a number of questions related to the structural performance of the north wall complex and the south shear wall. There was a limited response to these questions. However, in the assessment of the structural performance of the building several of the aspects were assessed by the Royal Commission. This section summarises the responses received and the additional conclusions we have arrived at due to our own study of the issues. In several cases more detailed information on these issues may be found in the discussion already set out above in this section.

6.3.6.1 Questions related to the south shear wall

a) *Would this wall have behaved as a coupled wall during the Canterbury earthquakes? In particular would the coupling beams have yielded with plastic hinges forming in each of the walls?*

During the hearing Professor Priestley expressed the view that the wall would have behaved as a coupled wall as intended. The written statement from Dr Hyland supported this view. Dr Arthur O'Leary, who was called by the CCC, said that he had checked the relative shear strengths of the beams and the flexural strength of the walls and concluded that the coupling beams were too strong to permit appreciable plastic deformation from developing in the coupling beams. This would have resulted in the coupled wall acting as a single unit.

The conclusions from our study are that the wall would have predominately acted as a single unit with only limited yielding occurring in the coupling beams.

b) *What influence would the floors in the building have had on the behaviour of the south wall?*

Our conclusion is that elongation of the coupling beams, which would have been initiated when flexural cracks formed in the beams, would have been partially restrained by the floor slabs. This restraining action would have significantly increased the shear capacity of the coupling beams.

Professor Mander agreed that restraint from the floor slabs would increase the shear resistance of the coupling beams. This is an issue that requires further research.

c) *Was there an adequate load path to transmit the inertial forces from the floors into the south wall?*

Our study indicates that there was an adequate load path for the design actions found using NZS 4203:1984, but there may have been an issue with the connection of the beams on line 1 with the coupled walls.

d) *How do the design inertial forces between the wall and the floors compare with the corresponding design actions calculated from NZS 4203:1984 and NZS 1170.5:2004[6]?*

The transfer forces in the September earthquake were generally comparable to the design actions found using values NZS 4203:1984. However, the corresponding values for the February earthquake were significantly greater than the design values. NZS 1170.5:2004 does not give any guidance on how these transfer forces can be assessed. This aspect needs to be studied and appropriate code clauses added to the design standard. This is one aspect of our previous Recommendation 36, which suggests that design actions for floors need to be more clearly identified in NZS 1170.5:2004.

6.3.6.2 Questions related to the north wall complex

a) *Given the lateral force resistance in the east-west direction what level of ductility would be appropriate in designing the wall and the inertial forces generated between the wall and the floors?*

No response was received on this question. We note that the stiffness of the north wall complex in the east–west direction was of the order of 20 times that of the south shear wall and the strength of the north wall complex was also considerably greater than that of the south shear wall. There is a complex interaction between the lateral stiffness and strength of these two elements and the torsional inertia of the floors that connect them. We note that the predicted tie forces between the floors and the walls were much higher for the north wall complex than for the south shear wall. Further guidance on how these actions should be allowed for in design is desirable.

b) *What was the load path for the shear transfer between the floors and the wall complex?*

The drag bars were added to accommodate seismic actions in the north-south direction and this work has been considered in detail in section 2.4 of this Volume. However, the seismic actions in the east-west direction are of particular concern. The only viable load path for shear transfer for this action was in the floor in the bay between walls C and C-D. The shear force in this bay creates an in plane bending in the floor. Both Professors Priestley and Mander commented that the reinforcement in the floor looked as though it would not be sufficient to resist the combined flexural shear actions. Mr Banks said in evidence that he had recently assessed the flexural strength of the floor and found it was only 60 per cent of that required to resist the design actions calculated using the parts and portions clauses of NZS 4203:1984. The 60 per cent figure was in reasonable agreement with corresponding calculations made by us. We note that the design actions found using NZS 4203:1984 were appreciably smaller that the corresponding actions predicted by the non-linear time history analyses. We also note that no guidance is given in the Earthquake Actions Standard, NZS 1170.5:2004, for the tie forces required between lateral-force-resisting elements and floors.

c) *Would the wall complex warp under the action of this shear transfer? Can you account for the observed vertical cracking in the wall complex?*

Professor Mander and Dr O'Leary agreed that the north wall would warp. Professor Mander went on to agree that this action could account for the vertical cracks in wall 5 in the lift shaft, which were observed by Mr Smith after the Boxing Day earthquake.

d) *What other structural actions are associated with shear transfer from the floor into the structural [north] wall complex?*

There were no responses to this question. The non-linear time history analyses[2] indicated that structural actions in the critical region of floor for the transfer of tie forces between the floor and wall complex included in-plane actions of direct tension, flexure and shear, and out-of-plane actions of flexure. The latter action arose due to the vertical deformation of the finger walls (C, C-D, D and D-E) associated with flexure in the north-south direction and possible rocking on the foundation.

e) *Is the detailing of the junction between the floors and the wall complex adequate to resist the shear force and associated actions?*

As noted in b) above the detailing was not adequate to resist the design actions found using the parts and portions clauses in NZS 4203:1984. The actions predicted by the non-linear time history analyses were much greater than the design actions demonstrating that the detailing was not adequate. The lack of ductility of the drag bars and the 664 mesh in the floor added to the inadequacy of the connections.

f) *How do the predicted magnitudes of shear force transfer between the floors and the wall complex correspond to the design values found from NZS 4203:1984 and NZS 1170.5:2004?*

The design shear force transfer forces in the September earthquake were typically 1.75 times the design values found using NZS 4203:1984, and the corresponding values for the February earthquake were typically four times the design values. No guidance is given in NZS 1170:2004 on how to find the design forces between floors and lateral-force-resisting elements.

6.3.7 Conclusions

The design calculations for Landsborough House were used as a guide for the design calculations for the CTV building. Both structures were eccentric in that there was an appreciable distance between the centre of lateral stiffness and the centre of mass, which would induce torsional actions in the building during an earthquake. Due to the high eccentricity the modal response spectrum method of analysis was used in the design of both buildings. However, there was a difference in approach. Mr Henry, who designed Landsborough House, interpreted the modal analysis conservatively while Mr Harding reduced the design actions to the minimum level required by the NZS 4203:1984. Having reviewed Mr Harding's design calculations for the purposes of the Royal Commission hearing, Mr Henry concluded that Mr Harding appeared to have made an error in scaling the modal response spectrum that led to the design being based on 80 per cent of the equivalent static analysis results. As we do not have the input or output from the modal analysis we cannot be absolutely sure that the values used by Mr Harding were in error. However, on the balance of probabilities we agree with Mr Henry's conclusions. We also consider that the likely 10 to 20 per cent reduction in design strength below that required by the Standard would not have had a significant influence on the collapse of the building.

There is a major question in relation to the deflection calculations for the CTV building. The fundamental period was found by Mr Harding to be 1.06 seconds in both the north-south and east-west directions. However, the deflections in the east-west direction are of the order of half those in the north-south direction. Displacement spectra indicate that they should be similar in magnitude. This observation supports Mr Henry's deduction that Mr Harding based his design inter-storey drifts on values related to the centre of mass and he failed to allow for the increase in drift due to torsional rotation of the building. This led to an underestimate of the inter-storey drifts of the columns on lines 1 and 2 in the building. This error had implications for the seismic performance of the building, as the magnitude of inter-storey drift was a factor which determined the type of detailing required for columns and beam-column joints (as discussed in section 8.1.7).

There are major weaknesses in all the beam-column joints in the building. These arose from the longitudinal reinforcement in the bottom of the precast beams, and in some cases the longitudinal reinforcement in the top of the beams, being anchored into the beam column joint zones by 90° hooks. The hooks were located in the mid-region of the beam-column joint zones. With this arrangement there was no effective lap length between the longitudinal bars. The consequence of this was that when the longitudinal bars were subjected to tension, tensile stresses were induced in the concrete. While the tensile strength of the concrete might have enabled the bars to resist about a quarter of their design strength before it cracked, once this cracking occurred strength degradation would have been rapid as the anchorage of the bars failed and concrete was pulled out of the joint zones by the tension forces in the reinforcement. In short, the failure of the beam-column joints would have been brittle in character. It is unlikely that visible cracks would have been apparent for more than an instant or two before failure, partly because gravity loading would have acted to close the cracks. The exception to this situation was in the beam-column joints where the beams in lines 1, 2, 3 and 4 met the columns on line A. At these locations gravity loads and seismic actions could both act to induce tension in the top beam bars and consequently, gravity loading would not have tended to close these cracks.

The error in the design of the beam-column joint zones occurred due to the designer not tracking the load path through the beam-column joints. If he had done this he should have noted that the integrity of the joint zone depended on the tensile strength of concrete, which is not acceptable as it is unreliable and tensile failure of the concrete leads to a brittle failure.

From the design calculations and the structural drawings for the CTV building it is clear that the floors were inadequately tied into the north wall complex. Mr Harding based his calculations for the required tie forces on the equivalent static analysis design forces, which gave values that were less than half those required by NZS 4203:1984. In addition he failed to allow for the in plane bending moments associated with the in plane shear forces for seismic actions in the east-west direction. The net result was that the floors were inadequately connected to the north wall complex. This was for two reasons. First, Mr Harding failed to correctly calculate the required tie forces between the floors and the north wall complex. Secondly, he failed to track the load path, which involved critical shear, flexure and direct tension forces in the floor close to where they connected to the north wall complex. This led to the connection between the floors and the north wall complex being considerably weaker than required by the then current design standards.

We have concluded that the four features in the CTV building which were the major contributors to the collapse in the February earthquake were:

1. The failure to adequately design the beam-column joint zones;
2. The failure to provide adequate strength between the floors and the north wall complex;
3. Inadequate confinement of the columns; and
4. The failure to identify clearly the need to roughen the interface between the ends of the precast beams and the in situ concrete in the columns.

6.3.8 The assessment of other buildings with potential structural weaknesses

It is important to identify other buildings in New Zealand that have characteristics that might lead to their collapse in a major earthquake, so that appropriate steps can be taken to reduce the potential hazard posed by these structures.

In any detailed assessment of a building, it is important to identify the load paths through the structure. The equivalent static and/or pushover analyses may be of assistance in identifying load paths associated with first mode type actions. However, it is also important to identify local load paths associated with higher mode actions that can contribute significantly to the forces required to hold parts and portions of structures:

a) to the lateral force resisting elements; and
b) to the forces between floors and lateral force resisting elements (see section 6.3.3).

The individual structural elements (beams, columns, walls, and braced frames) and the connections between them should be examined to identify the load paths through them and any inherent weak or potential non-ductile failure mechanism.

Where appropriate, non-linear time history analyses may be carried out to identify the likely magnitude of actions that the different elements may need to sustain in the event of a major earthquake. However, these analyses should not be undertaken before steps have been taken to identify the critical zones as described above. The knowledge gained from the preliminary assessment of the different structural elements enables the structural engineer to focus on these critical locations and not get lost in the potentially massive output from such analyses. In interpreting the analytical results it is essential that allowance is made for the approximations inherent in the modelling. This is particularly important in situations where non standard details are used, such as arose in the beam-column joints of the CTV building (see section 6.3.5).

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 that 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.

References

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

2. Compusoft Engineering. (2012). *CTV Building: Non-Linear Time History Analysis (NLTHA) Report* (Ref: 12026-00). Christchurch, New Zealand: Canterbury Earthquakes Royal Commission.

3. NZS 3101:2006. *Concrete Structures Standard,* Standards New Zealand.

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

5. Hyland, C. (2012). *CTV Building Site Examination and Materials Tests for Department of Building and Housing: 16th January 2012*. Wellington, New Zealand: Department of Building and Housing.

6. NZS 1170.5:2004. *Structural Design Actions, Part 5 – Earthquake Actions – New Zealand*, Standards New Zealand.

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

Section 7:
The collapse

7.1 Introduction

The Terms of Reference require the Royal Commission to consider why the CTV building failed severely.

The north wall complex and south shear wall were designed to be the primary earthquake load-resisting elements in the building. The columns were designed to provide support to gravity loads on the floor slabs.

The north wall complex did not fail as such; it remained standing after the collapse and was eventually deconstructed. However, it failed to perform its intended function of resisting earthquake loads on the building because the floor slabs detached from it. The south shear wall similarly failed in that the floor slabs detached from it and it toppled over, coming to rest on the remains of the building. The columns also failed to perform their function of providing gravity support for the floor slabs, although this failure may have been related to the failure of the joints between the columns and the beams.

It is clear that the building failed in a number of ways and that the consequences of these failures were as severe as they could possibly have been in terms of the safety of those in the building.

A number of expert witnesses who gave evidence to the Royal Commission put forward possible collapse scenarios. In most cases, an initiating point of failure was identified, as were the consequences of that failure. Some of the scenarios had common features while others were markedly different. However each of them necessarily incorporated the disconnection of the diaphragm connections at the north wall complex and south shear wall and the failure of the columns and/or beam-column joints.

The experts variously identified failures of the following parts of the building as possible collapse initiators:

- buckling or crushing of the columns, either internally or externally;
- disintegration of the beam-column joints;
- disconnection of the floor slabs from the north wall complex;
- disconnection of the floor slabs from the south shear wall; and
- disconnection of an interior beam from a west wall column.

In some of the proposed scenarios, the initiator resulted in the failure of other critical parts of the building. However, some experts gave evidence that the point of failure identified would have been sufficient to cause complete collapse.

A number of sources of evidence were available to those experts, and to the Royal Commission, to consider possible collapse scenarios:

- the evidence of witnesses to the collapse, including those who were in the building at the time;
- photographs of the building following collapse;
- evidence about the state of elements of the building following collapse; and
- the results of computer analyses.

Each of the possible collapse scenarios will be described and considered below.

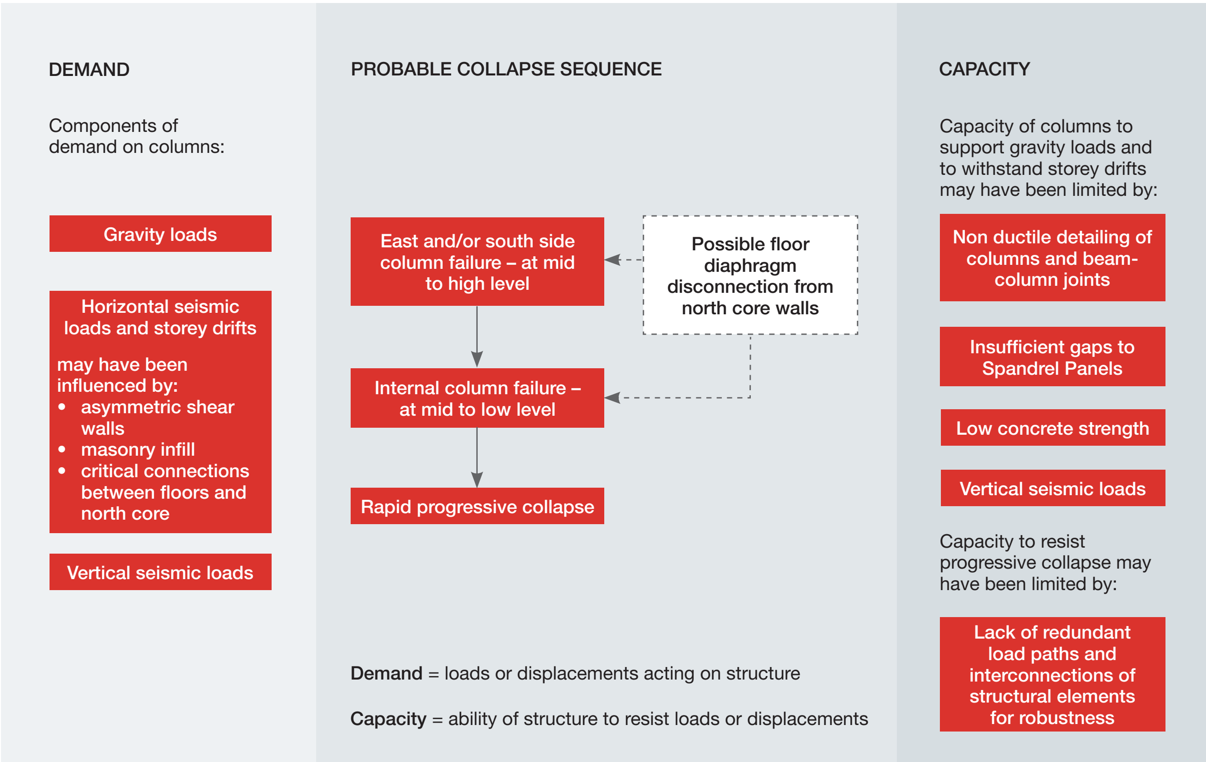


Figure 91: Collapse sequence flowchart produced in the Hyland/Smith[1] report

7.2 Possible scenarios

7.2.1 The Hyland/Smith report scenarios

Dr Clark Hyland and Mr Ashley Smith evaluated various collapse scenarios for the purpose of identifying, if possible, the most likely. They produced a flowchart to illustrate the key considerations they used (see Figure 91).

The failure of one or more columns was central to their approach in each of their scenarios. They considered that collapse was almost certainly initiated by failure of a column when the lateral displacement of the building was more than the column could sustain. A comparison of “demand” (loads and displacements imposed on columns due to gravity and earthquake actions) and “capacity” (the strength and deformation capacity of critical columns) lay at the heart of their assessment.

As their flowchart illustrates, column failure was seen as the initiator of the collapse, with a possibility of contribution from the disconnection of the floor slabs from the north wall complex.

Four possible scenarios were identified.

7.2.1.1 Scenario 1, collapse initiated by columns in line 1 or F

As the building moved in the earthquake, the columns on line 1 and line F would have been exposed to the highest inter-storey drifts in the structure. This would have led to a failure of the line F columns in the second to fifth storeys, possibly exacerbated by interaction with the adjacent spandrel panels.

With the loss of the load-carrying capacity of the columns on line F, the interior columns on lines 2 and 3 at a mid to low level would have become overloaded. The slabs and beams they supported would have pulled downwards and northwards on the south shear wall and frame on line 1. The slabs and beams connected into the columns at line A would have pulled downwards and inwards on those columns, which could explain the beam-column joints pulling out in some locations. The upper levels and roof above the column failure on line F could have dropped as a unit, with a slight lean to the east.

Dr Hyland and Mr Smith described this in their report as their “preferred collapse scenario”, given that some witnesses saw the collapse start in the upper third of the building, a witness reported a slight tilt to the east and debris was observed on Madras Street after the collapse. This scenario was represented in three figures in their report (see Figure 92):

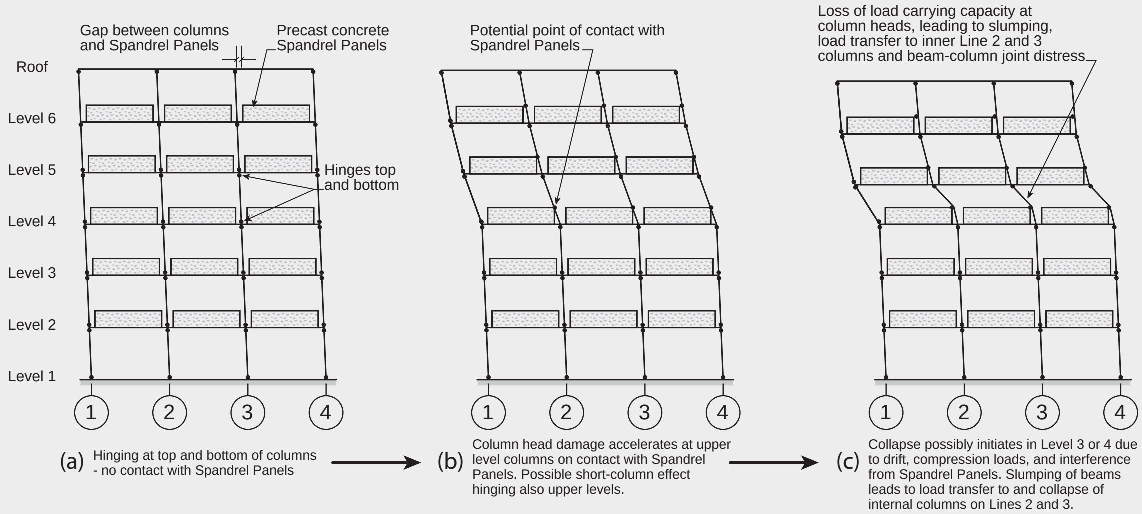


Figure 17: Possible collapse sequence along Line F as inter-storey drifts reach critical levels and columns begin to fail from lack of displacement capability or from additional damage caused through contact with precast concrete Spandrel Panels. Displacements and damage are greatest in the upper levels, but inelastic drift capacity less in the lower levels. Also change in torsional stiffness at Level 4 due to the Line A masonry infill wall stopping at that level may have contributed to collapse appearing to initiate above Level 4.

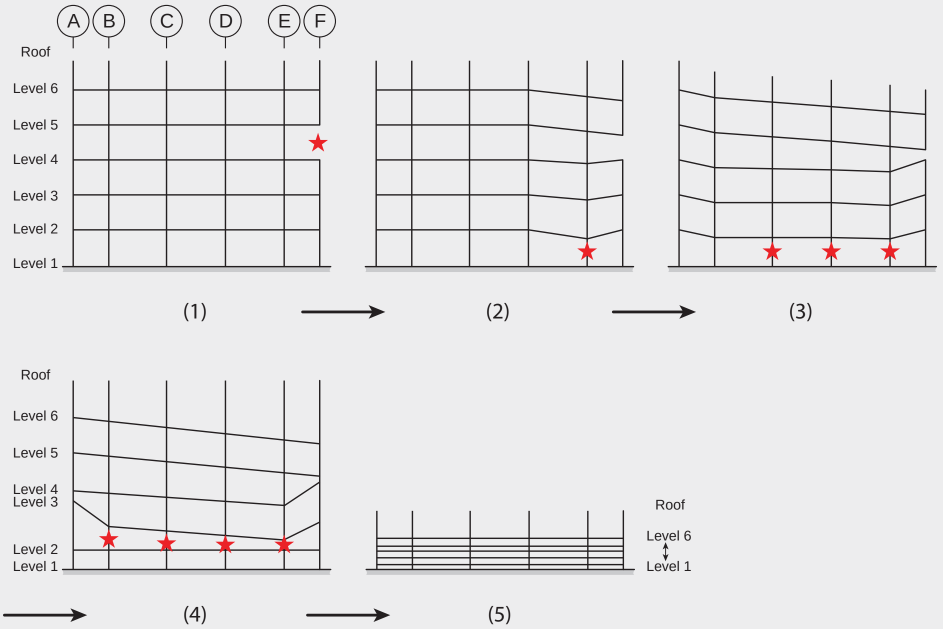


Figure 19: Possible progression of collapse from loss of column capacity on Line F is shown sequentially as follows: (1) Collapse of Line F columns above Level 4 leads to extra floor area being supported of columns on Line E; (2) The Line E columns begin to collapse under the extra load; (3) As the Line E columns sink additional floor area becomes supported on the Line D columns which in turn begin to collapse, causing an eastward tilt in the upper levels; (4) The upper levels then hit the Level 4 Line F; (5) The collapse completes with all floors laying on top of each other. Note that collapse is also spreading in the north-south direction simultaneously to this as shown in Figure 19.

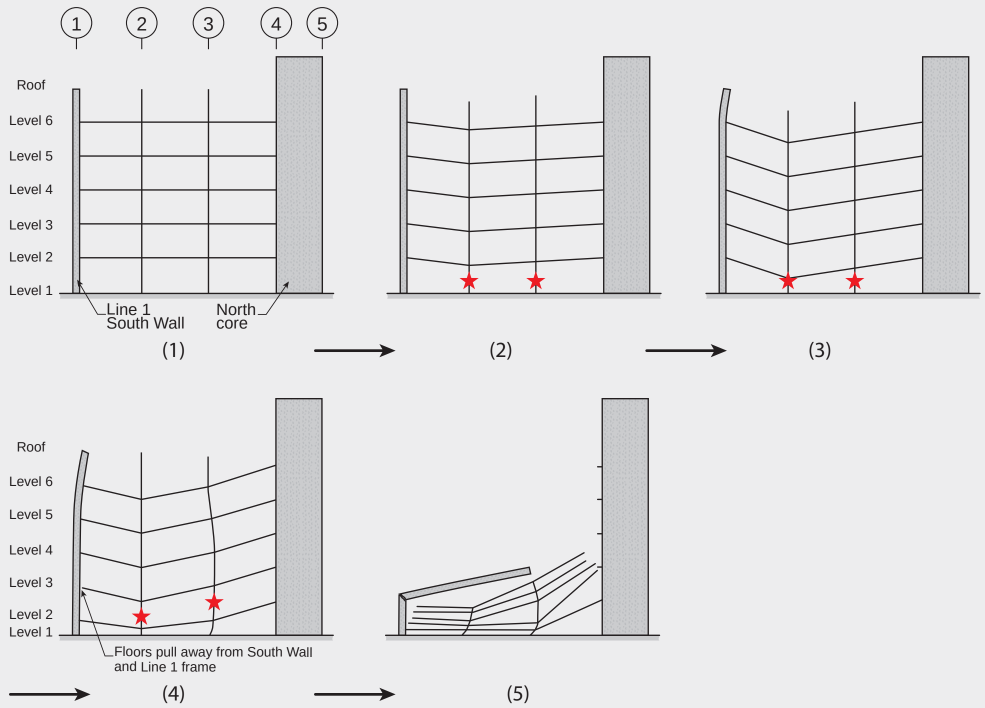


Figure 20: Possible progression of collapse on a north-south section through the building simultaneous with that shown progressing westwards in Figure 19 (1) Initial condition; (2) Line 2 begins to subside; (3) As line 2 subsides further the stiffer and stronger North Core pulls the collapsing floors towards it; (4) The South Wall is pulled northwards; (5) The slabs pull away from the North Core eventually lying diagonally against it and the South Wall is pulled down onto the collapsed building.

Figure 92: Figures 17, 19 and 20 from the Hyland/Smith report illustrating their preferred collapse scenario

7.2.1.2 Scenario 2, collapse initiated by failure of internal column on line 2 or 3

In this scenario, collapse was initiated by the failure of one of the most highly loaded internal columns on line 2 or line 3, following which the floor would have sunk and the slabs would have been forced into catenary behaviour[2]. The structure would have progressively collapsed onto itself.

Dr Hyland and Mr Smith said in their report that this is a credible possibility that cannot be discounted but that it “may not be totally consistent with the observation of an eastward tilt as the upper levels fell as a unit and the slight eastward throw of debris into Madras Street. The isolated internal column collapse initiation would perhaps have been more likely to have resulted in an even more concentric debris pile on the site than what was observed”.

However, Mr Rob Jury, who was part of the DBH Expert Panel, gave evidence of his opinion that this scenario was more likely than Scenario 1 and that the initiator of the collapse may have been an internal column on level 1. He considered that the second Compusoft Engineering Limited[3] (Compusoft) NLTHA results provided a stronger case for this than the earlier analysis.

7.2.1.3 Scenario 3, collapse initiated by disconnection of floors at levels 2 and 3 from north wall complex

The diaphragm connections at levels 2 and 3 of the north wall complex (at which there were no drag bars) detached due to potentially high in-plane flexural demands. The effect of this detachment would have been to overload the columns on levels 1, 2 or 3 by imposing greater lateral displacement due to the loss of restraint from the north wall complex.

However, Dr Hyland and Mr Smith concluded that diaphragm disconnection at levels 2 and 3 was not entirely consistent with the collapse evidence and less likely than their first two scenarios. They referred to evidence of the floor slab lying against the north wall complex in a manner indicating it had lost support at the line 3 end of the slab rather than at the north wall complex (see Figure 93).

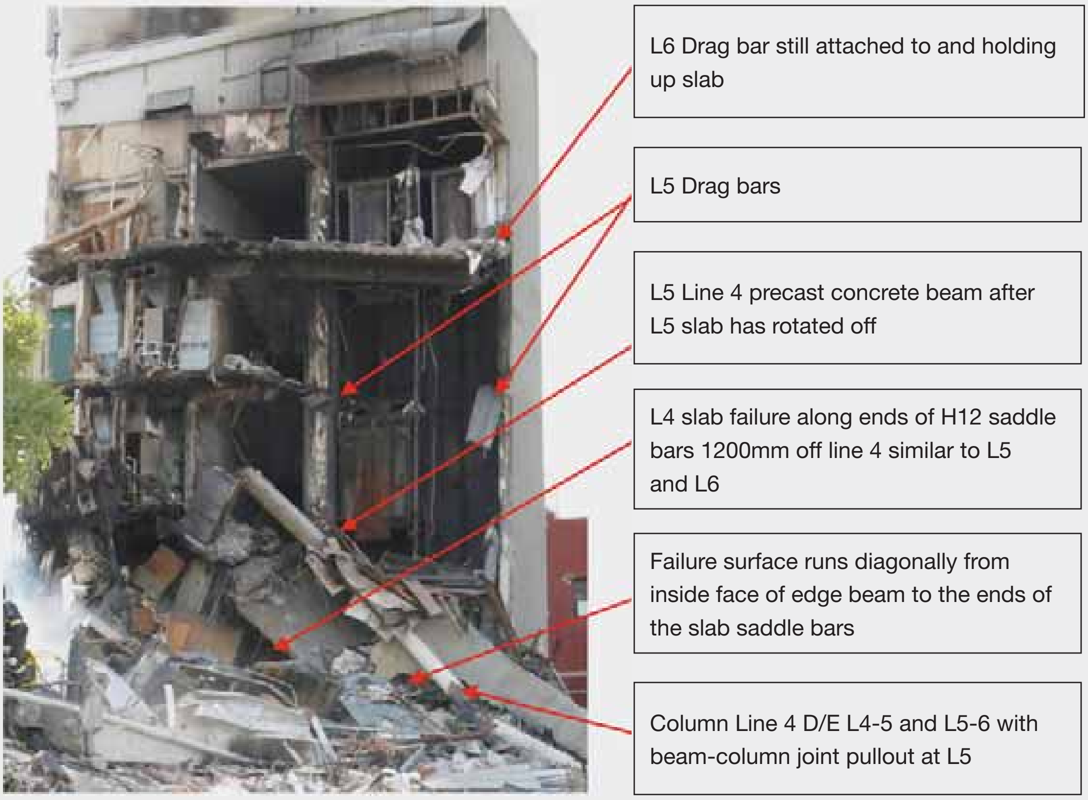
Figure 93: Figure 95 from the Hyland/Smith report showing the slabs lying against the north wall complex

7.2.1.4 Scenario 4, collapse initiated by disconnection of drag bars from levels 4, 5 or 6

This scenario is similar to Scenario 3 but also includes failure of the drag bars and adjacent slab. The disconnection from the north wall complex at levels 4, 5 or 6 would lead to column failure. This might have been compounded by the effects of east-west foundation rocking and uplift of the slab/wall connection due to northwards displacement.

Rocking or tensile extension of the south face of the north wall complex as part of the north-south response may have initiated failure and detachment of the floor slabs due to a combination of in-plane and out-of-plane diaphragm actions. With the detachment the inter-storey drifts would increase resulting in failure of the columns.

Dr Hyland and Mr Smith concluded that this scenario was not entirely consistent with the collapse evidence. Photographs of the north wall complex showed that the levels 4, 5 and 6 slabs may not have failed initially at the drag bars (see Figure 94). For these reasons they believed it was less likely than the previous three scenarios.



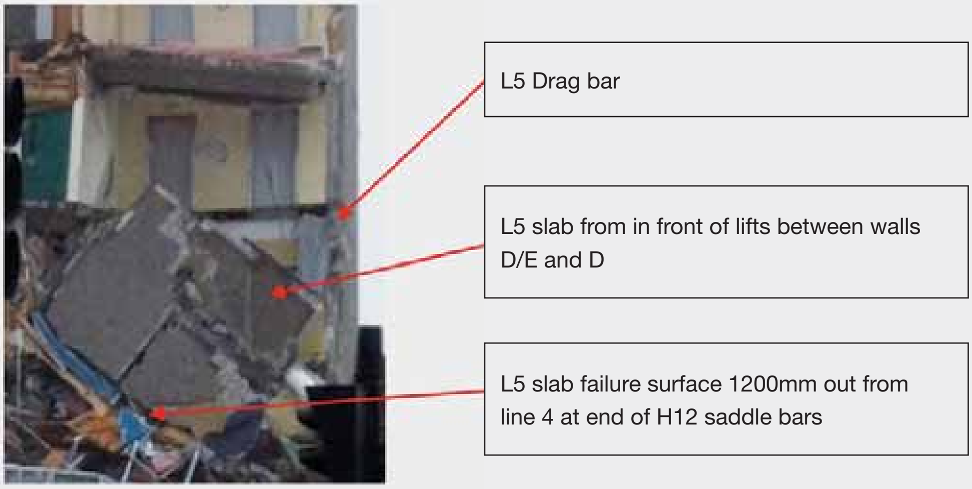


Figure 94: Figures 163 and 164 from the Hyland/Smith report showing diaphragm failure at the north wall complex

7.2.1.5 Discussion on collapse scenarios 1, 2, 3 and 4 from Dr Hyland and Mr Smith

All four of the collapse scenarios proposed by Dr Hyland and Mr Smith are based on the premise that a column or columns initiated failure due to excessive drift being developed during the earthquake. They identified columns on the east and south sides of the building as the ones most likely to have initiated failure.

The limiting inter-storey drifts the columns could sustain were calculated by Dr Hyland using a Cumbia software program to calculate moment curvature relationships for reinforced concrete members subjected to axial load. The analysis is based on the assumption that plane sections remain plane. The failure criterion for lightly confined columns, such as those used in the CTV building, was taken as the actions sustained when a limiting strain of 0.004 was reached in the concrete. A limiting strain of 0.004 is widely assumed to be the value at which spalling of concrete commences. In practice this critical strain level varies and it is generally in the range of 0.003 to 0.007. For lightly confined columns, such as those in the CTV building, spalling of the concrete could be expected to result in a loss of flexural strength. However, a reduction in flexural strength does not necessarily result in failure of the axial load-carrying capacity, provided there is some other load path that can resist P-delta forces, which would be the case provided there is some connection between the floors and the walls.

We note that it is difficult to identify a critical displacement that will cause an axial load failure and this displacement may have been underestimated by the authors. Our concerns about the Hyland/Smith analysis include their use of low concrete strengths, the assumption that plane sections remain plane, the assumption about the stiffness properties of the columns and the approximation of the plastic hinge length.

Dr Hyland and Mr Smith note in their report:

> …failure may not have occurred at the drag bar connections to the North Core at levels 4, 5 and 6 prior to slab pulling away. The slabs at levels 5 and 6 were seen to have hung up on the North Core with their line 3 ends resting on the ground after the collapse as seen in Figure 95, (which is reproduced in Figure 93). This would not be expected to have occurred if they had first lost their support adjacent to the North Core. It is concluded that the slab failures at level 4, 5 and 6 had most likely occurred due to the floors losing their supports along lines 2 and 3 as those columns collapsed.[4]

We note that Figure 95 in the Hyland/Smith report, which is reproduced in Figure 93, shows that the floors of levels 5 and 6 are leaning against the north wall complex opposite walls C, C-D and D but not to the east of this. This could be explained by failure of the drag bars and by partial disconnection between the floors and the wall in the region between the finger walls of C and D, with complete disconnection occurring when the slabs collapsed as a result of the failure of the columns on lines 2 and 3.

In Scenario 1, Dr Hyland and Mr Smith identified a column on line F, the east side of the building, as the most likely location of the critical column that initiated the collapse. In Scenario 2 they indicated that one of the heavily loaded internal columns on lines 2 or 3 may have failed due to excessive axial load. This deduction was in part based on the low concrete strengths that they had measured in core test samples taken from the remains of the columns after collapse. Later evidence indicated that these test results were in all probability on the low side and not representative of the actual concrete strength in the columns. We have discussed this in section 2.3.4.

As previously noted the third scenario postulated by Dr Hyland and Mr Smith was failure due to total disconnection between the floors and the north wall complex. As with the previous cases this is a possibility, but clearly not one that the authors felt was likely to have occurred. Scenario 4 is similar to the third but involves the additional failure of the drag bars which were at the higher levels. Again the authors clearly felt this was unlikely.

The Royal Commission accepts that all four collapse scenarios described by Dr Hyland and Mr Smith are possible.

7.2.2 Professor's Priestley's scenario

Professor Nigel Priestley gave evidence that the columns on line F were unlikely to have acted as the failure initiator. He considered it more likely that failure of the connections between the diaphragms and the north wall complex would have occurred early in the building's response to the February earthquake. It was, in his view, "entirely possible" that partial disconnection in that location had already occurred during the September earthquake. Failure of the connection in the February earthquake would have increased the inter-storey drifts and this would have caused distress to a number of beam-column joints. Professor Priestley indicated that the beam-column joints were weaker than the columns. As a result, yielding of the column reinforcement would have been largely confined to the beam-column region. The consequent spalling of concrete from the bottom of the joint zone would reduce the capacity of columns to support vertical loads and horizontal displacements. The failure of the connection to the north wall complex would result in an increase in displacement demands to the columns and the failure of internal columns. It was Professor Priestley's view that the "failure of internal columns due to the combination of large displacements, spalling of concrete and high vertical loads (including vertical acceleration effects) would result in explosive failures of the columns and the beam-column joints".

7.2.2.1 Discussion

We note that early failure of the connections between the floors and the north wall complex features in several scenarios. We consider that early failure of the drag bars is likely. From the photographs after the collapse it appears that the failure of the drag bars was the result of either the anchors between the steel angle and concrete slab failing or the failure of the reinforced concrete slab in which the drag bars were anchored. The slab was lightly reinforced with non-ductile mesh. Failure of the drag bar anchors or of the slab would be brittle as the connection had little ductility. Consequently it is likely that the drag bars would have failed in the first few seconds of the intense ground shaking in the February earthquake (see section 6.3.3). As suggested by Professor Priestley it is possible that some of the drag bars disconnected in the September earthquake.

Failure at the interface of a column at the lower surface of a beam-column joint due to spalling of the concrete is supported in other scenarios and we accept that it is a likely cause of collapse

7.2.3 Mr. Holmes' scenario

Mr William T. Holmes proposed a "refined collapse scenario" in his peer review[5] of the Hyland/Smith report into the collapse of the CTV building. He considered that a global collapse mechanism was caused more by the degradation of the beam-column joints than by column hinging. He said that joint degradation would have been "sudden and complete" and would have lead to a global collapse far more directly than column hinging because all moment capacity in the joint would be lost, gravity support for the columns would be lost and the joint could come apart, leaving only the weak floor topping to hold the floor plates together.

Mr Holmes referred to photographic evidence of almost universal joint failure and to the evidence of Mr Graham Frost, who noted that there were no intact beam-column joints to be found on the site after the collapse.

Mr Holmes said that failure of the beam-column joints would also lead to a more direct vertical collapse, while failure of the columns due to the formation of plastic hinges would have led to greater lateral sway as the floors collapsed. He noted the vertical collapse mode was consistent with eyewitness accounts and with the "folding over of the front [south] coupled shear wall into the centre of the building with only a slight tilt towards the east (see Figure 95). It was also consistent with several of the floor slabs leaning against the tower indicating that line 3 probably collapsed before these slabs lost vertical support from the tower [north wall complex] along line 4".

Mr Holmes indicated that the beam-column joints were all potentially susceptible to joint degradation. However, he identified the beam-column joints on line A at the intersection with the beams on lines 2 and 3 as being particularly susceptible to rapid degradation due to the way the beam bars were hooked into the beam-column joints (see Figures 88 and 89 in section 6.3.5). He quoted the account of Mr Leonard Fortune (eyewitness 16 in the Hyland/Smith report) who saw the column at A-1 kick out as though it had buckled from what is believed to have been level 4 before it fell narrowly missing him. Mr. Holmes' interpretation of that account is that there was a complete loss of strength of the joint zone, leading to a buckling-type failure of the column over two storeys.

Mr. Holmes noted that there was a lack of apparent yielding in the north wall and this was not consistent with the predicted drifts of the wall. From this he concluded that partial or complete separation of some of the floors would have occurred at an early stage in the earthquake. This would have led to increased drifts of the columns and greater structural actions on the beam-column joint zones.

Figure 95: Aerial photograph referred to by Mr Holmes

Mr Holmes said that if the columns in the building had been better detailed, but the beam-column joints had not, the building would probably still have collapsed. However, if the beam-column joints were improved, both for shear and confinement so as to better tie the beams to the columns, the building may not have collapsed so completely, provided that the lateral loads were adequately transferred to both the north wall complex and the south shear wall.

7.2.3.1 Discussion

We consider that Mr Holmes' scenario highlights some of the critical weaknesses in the building and that failure of the beam-column joints accompanied by partial disconnection of the floor slabs from the north wall complex was a very likely cause of the collapse.

7.2.4 Professor Mander's scenarios

Professor John Mander described three alternative gravity dominated collapse scenarios, namely:

- a collapse mechanism under east-west shaking;
- a collapse mechanism under north-south shaking; and
- a collapse mechanism under northward displacement.

7.2.4.1 Collapse mechanism under east-west shaking

The inter-storey drifts in the east-west direction predicted from in the non-linear time history analyses carried out by Compusoft[3] were of the order of three per cent. Such displacements would have generated negative and positive moments in the beams on lines 2 and 3 where they were framed into the columns on line A. As illustrated in stage 1 in Figure 96, these bending moments would have generated cracks in the columns behind the hooked beam bars (see also section 6.3.5).

Professor Mander postulated that the positive moment rotation would have caused the concrete below the beam to spall, allowing the beam to drop (as shown in stage 2 in Figure 96) and become wedged between the columns on lines A and B. This action would have partially unloaded the support and transferred gravity loads to the adjacent column or columns on line B. Professor Mander proposed that, as the inter-storey drift decreased, the wedging action of the beam would have pushed the row of columns to the west as illustrated in stages 3 and 4 in Figures 97 and 98.

Professor Mander estimated that, including vertical load effects, there was an increase in axial load on the level 2 columns of 400kN, leading to the incipient collapse mechanism shown in stage 4 of Figure 97. He calculated that differential drifts of 1.15–1.3 per cent, depending on the concrete strength, would have been sufficient to cause a column or columns to fail. The final collapse mechanism is shown in Figure 98. Professor Mander noted that this mechanism is consistent with some of the eyewitness accounts.

Sequence

Stage 1
The building sways to the west with a large velocity pulse. The E-W beams on column lines 2 and 3 at the west wall are required to form large negative moments that cause the joint core concrete and the beam-soffit cover concrete to crush.

Stage 2
During the next half-pulse the building lurches eastward. The beam along line 2 and 3 pull away from the west wall and their line A column seats to form the alternating positive movement. The crushed cover concrete from the previous reversal spalls off and the beam slumps down a little, with a partial or full loss of seating. Due to the loss of seating at the support line A there is a transfer of the previous gravity load from the tributary area of the beam onto the neighbouring columns on line B. This action an axial force increase of up to 40% the columns along line B.

Stage 3
As the building attempts to return to an upright condition by moving west, the unseated beams are inhibited from fully returning due to the presence of the west wall.

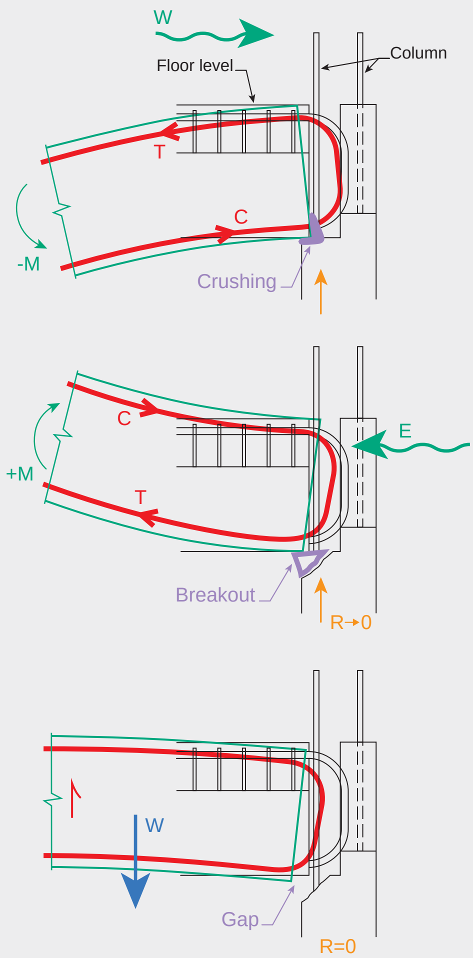


Figure 96: Professor Mander's diagram of the trigger for the east-west collapse failure mode

Stage 4
Permanent differential deformations remain that inhibit the columns along line B from remaining straight. This sets the columns up for a classic Euler buckling type failure, especially under further axial load derived from vertical accelerations and their consequent vibrations.

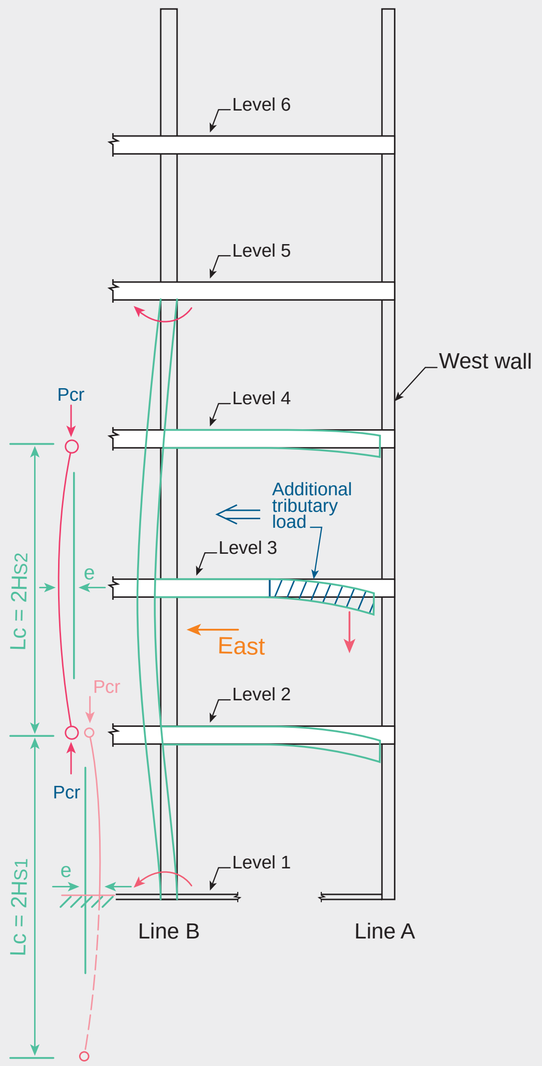


Figure 97: Professor Mander's diagram of four-storey double bending buckling failure starting on column line B leading to the east-west collapse failure mode

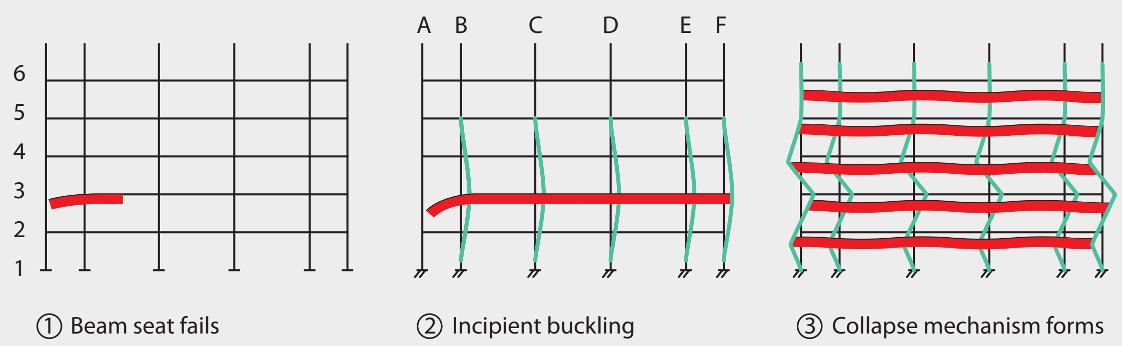


Figure 98: Professor Mander's collapse scenario diagram for east-west collapse mode

7.2.4.2 Collapse mechanism under north-south shaking, northward mechanism

Professor Mander postulated that the metal tray in the composite floor debonded from the concrete due to the high vertical acceleration. He suggested that this debonding could have resulted in the deflection of the floors associated with the hump on level 4 which was noted by Ms Margaret Aydon, Ms Marie-Claire Brehaut and Mr Ronald Godkin from King's Education after the September earthquake and some subsequent aftershocks. This is an alternative explanation to that given by Mr David Coatsworth who believed deflection was due to differential shrinkage of the concrete, which was known to be a common cause of such slab deflections increasing with time.

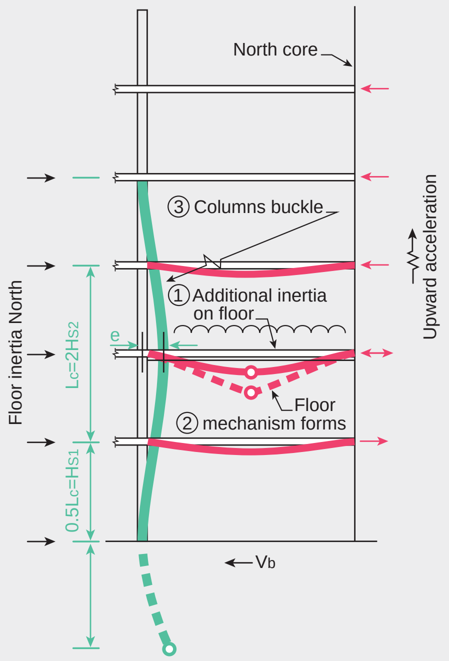
This collapse scenario is based on the assumption that, in a strong northward earthquake pulse, the sagging floor in the bay between grid lines 3 and 4 would have sagged sufficiently due to gravity loading and vertical ground motion to cause it to fail due to P-delta actions (see Volume 1, section 3.2.5) when the floor was subjected to compression as it transferred inertial force to the north wall complex. Collapse of the slab would drag the columns on lines 2 and 3 towards the north resulting in a classic P-delta collapse of the columns, as shown in Figure 99. This figure shows the collapse being initiated by the sagging of a single slab, inducing a P-delta collapse mode in two or more storeys of a column or columns on line 3.

Sequence

Step 1
Due to the lack of beams in the N-S direction and very high vertical motions, the in-plane stiffness is low. The slab buckles downward due to a combination of upward vertical acceleration, and N-S sideway of the frame.

Step 2
Because the slab buckles, and the columns lack lateral support in the N-S direction, a 4-storey, double bending column buckling mechanism forms.

Step 3
Column buckles and collapses.



Northward collapse mechanism

Figure 99: Professor Mander's possible collapse mode diagram for north-south shaking, northward mechanism

7.2.4.3 Collapse mechanism under north-south shaking, southward mechanism

In this scenario southward inertial forces drag the slabs towards the south leading to disconnection of the slabs from the north wall complex. It is suggested that the lack of drag bars on the lower floor levels (2 and 3) would result in the reinforcement that connects the floors to the north wall complex fracturing in low cycle fatigue, after one or two cycles of displacement. The disconnected floors would enable the columns on lines 2 and 3 to form a classic buckling deflected shape (see Figure 100). The buckling failure of the most heavily loaded columns on lines 2 and 3 would result in their axial loads being transferred to other columns, leading to complete collapse occurring.

Sequence

Step 1
Due to the absence of drag bars in the lower stories, there is a large strain demand placed on the slab steel connecting with the North core. After one or two cycles the bars fracture due to low cycle fatigue.

Step 2
The columns on lines 2 and 3 lack lateral restraint from moving independently southward, therefore they move away from the north core. A double-bending column buckling mechanism forms in the lower four stories.

Step 3
Several columns buckle and the structure collapses downward.

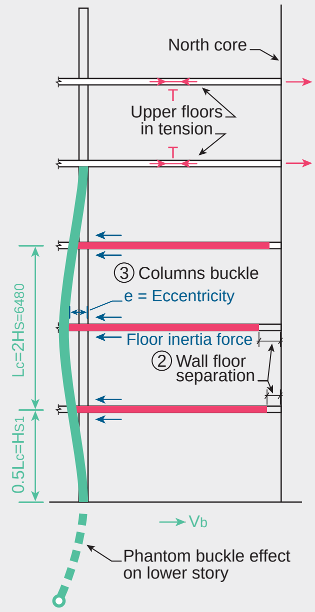


Figure 100: Professor Mander's possible collapse mode diagram for north-south shaking, southward mechanism

7.2.4.4 Discussion

The Royal Commission accepts that two of Professor Mander's proposed failure mechanisms are possible, but unlikely.

We consider the first mechanism to be an unlikely initiator of the collapse. However, we do accept that the failure of the beam anchorage into the column is very likely to have contributed to the collapse of the building.

The differential movement of the column on line B relative to the column on line A, as shown in Figures 97 and 98, implies that the floors provide very little resistance to lateral displacement. This could only have been the case if the floors were disconnected from at least one of the structural walls on lines 1 and 5. The elongation associated with the formation of the crack behind the hooked beam bars in the column, or alternatively the wedging action of the beam when it settles due to the spalling of concrete, would have induced axial forces in the beam that spans between the columns on lines A and B. However, it is not clear why this should push all the columns and floor slabs to the east of line A towards the east but not push the single column on line A to the west. It may be noted that there is little holding the columns on line A into the building. As the building sways in the east-west direction it is likely that cracks that separate the beams from the columns, which are illustrated in stages 1 and 2 in Figure 96, would develop over the height of the column. Once these cracks have formed, all that is holding the column to the floors are the 12mm starters at 600mm centres, anchored into the 140mm-wide precast beams on levels 2 and 3, but not above level 3. Due to the narrowness of these precast beams and the ineffective way they are anchored into the columns (see Figure 89 in section 6.3.5), the starter bars would provide little lateral restraint to the columns. Above level 3 there is no reinforcement holding the columns into the floors except in the beams, which will have been broken out of the columns, as illustrated in Figure 96 and described above. We do not accept this as a possible failure mode.

The second collapse scenario depends on the floor slab being debonded from the metal tray, which it is postulated occurred in the September earthquake or in subsequent aftershocks. However, we find it difficult to envisage such complete debonding occurring unless the vertical accelerations were so high that load reversal occurred. The vertical accelerations in the September earthquake were considerably smaller than those in the February earthquake. However, even in February the response spectra for the four ground motion records in the Central Business District (CBD)[6] show that vertical accelerations for elastic response only exceed gravity for periods of less than 0.2 seconds. We consider that the combination of a high initial stiffness for the floor slab to sustain forces that were sufficient to debond the concrete from the metal tray, followed immediately by a very flexible slab necessary to enable a P-delta failure, is unlikely. Consequently we do not consider that this failure mechanism is likely to have occurred.

The third scenario is based on the assumption that the floors at levels 2 and 3 disconnected from the north wall complex and sway of these floors occurred until the columns collapsed in a P-delta mode. We accept this is a possible failure mode but consider it unlikely.

7.2.5 Closing submission from Alan Reay Consultants Ltd and Dr Reay

In closing submissions, counsel for Alan Reay Consultants Ltd (ARCL) and Dr Reay submitted that "the collapse probably initiated from the southern shear wall (leading directly to column and slab failure in the immediate vicinity followed by all elements save the north shear core)". It was also submitted that collapse may not have occurred without the initial trigger of disconnection of the south shear wall. On this analysis the CTV building failed severely in part because of the damage caused by the September design level earthquake, which meant that the building, without repair, could not withstand the exceptional vertical and other forces experienced in the February earthquake.

A schedule titled "Probable collapse initiation mechanism" was attached to Mr Rennie[7] QC's written submissions. This set out the argument in support of this failure mechanism, including references to evidence and a series of diagrams. The schedule described reinforcing failure in the connections of the floor slabs to the south shear wall causing the slabs at that point to disconnect from the wall and collapse. A hinging effect on the beams at line 2 commencing at level 3 was illustrated in a diagram (see Figure 101).

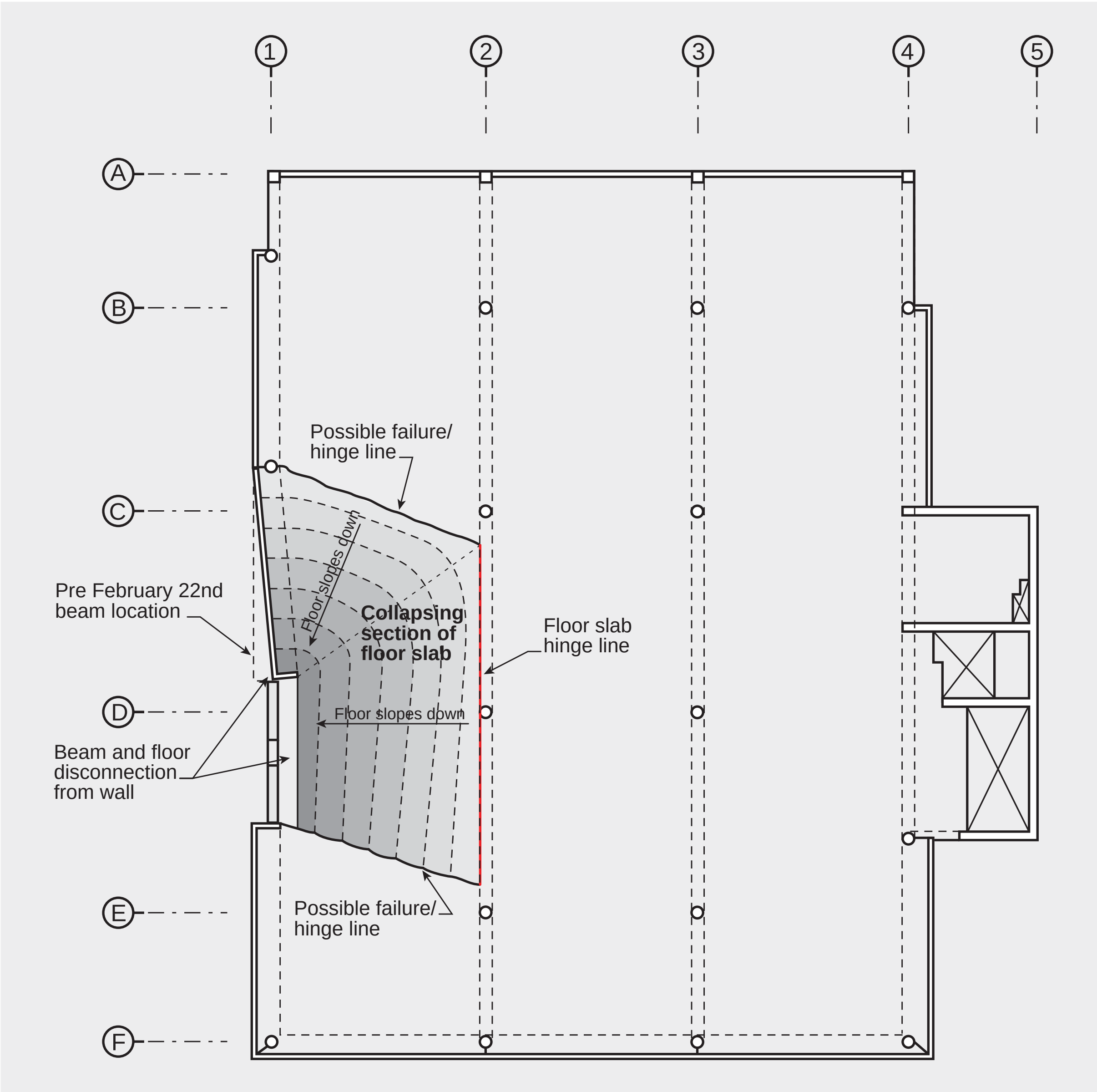


Figure 101: ARCL and Dr Reay's "Probable collapse mechanism stage 1 – level 3"

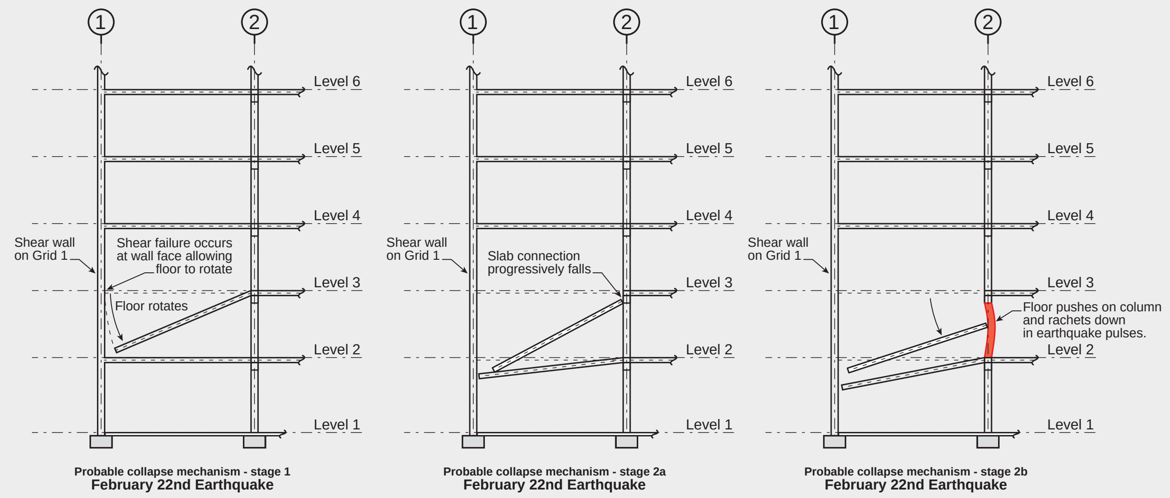


Figure 102: ARCL and Dr Reay's "Probable collapse mechanism"

The scenario is based on the premise that inter-storey drift of the order of 1.5 per cent would cause the reinforcement connecting the slab to the wall to fail, resulting in the slab on the third level collapsing onto the lower level as shown in Figure 102. It was postulated that this collapse would occur by hinging of the floor slab around the beams and columns on line 2, with the slab then acting as a prop that would bear against a column to cause it to bend and fail as indicated in the Figure 102. The schedule went on to say:

> Once the collapse of the floors in that area occurred, it set in motion the collapse of the columns on line 2 (by the bending/buckling failure of the column/s). This occurred at the same time as the columns were subjected to additional high axial loads resulting from the very high vertical accelerations. The chain reaction that followed brought the building down.

The text includes a number of photographs of cracks in the coupling beams (see Figure 103) and it was submitted that some of the cracks were 2mm in width. In addition it was noted that Mr Coatsworth identified a few cracks in the south shear wall. It was suggested that the damage represented by these cracks would have contributed to the collapse of the building.

Figure 103: Cracking in south shear wall coupling beams (source: Leonard Pagan)

We do not accept that the cracks shown in the photographs or those identified by Mr Coatsworth were of significance in the collapse. We consider that if the cracks had been as wide as ARCL indicates Mr Coatsworth would have recorded their widths. We note that reinforced concrete has to crack to enable the reinforcement to work and crack widths apparent in the photographs do not indicate a significant loss of strength or seismic performance. The seismic motion in the February earthquake was considerably more violent than that in the September earthquake or in any of the aftershocks that occurred before 22 February. In the first few seconds of the February event more damage would have been sustained in the south shear wall and local floor areas than would have been induced in the previous events. We conclude that any damage sustained in the south shear wall and its connection to the floors would have had no significant influence on its structural performance in the February earthquake.

An inter-storey drift of 1.5 per cent in the north-south direction could be expected to generate a crack width just over 2mm at the level of the reinforcement. Tests have shown crack width of this order can cause non-ductile mesh such as the 664 mesh used in the CTV floors to fail[8]. However, this would not cause failure of the slab wall connection. Eight high-strength ductile 12mm bars projected from the wall into each floor slab and these, together with the Hi-Bond metal tray (which the drawings show was supported on a 25mm ledge into the wall), would have maintained the shear strength connection between the floors and the wall. Tests[9] on high-strength 12mm bars extracted from the collapsed floor slabs have shown that this reinforcement was ductile. Tests[10] at the University of Canterbury have shown that such reinforcement can sustain crack widths of the order of 20mm before failure occurs. An inter-storey drift in excess of 10 per cent of the storey height would be required for this metal tray to be pulled out so as to cause collapse.

We believe that this proposed failure mechanism does not provide a valid explanation of the collapse mechanism of the building.

7.2.6 Mr Harding's scenario

Mr David Harding noted in his evidence that the Hyland/Smith report acknowledged that the vertical ground accelerations increased the axial loads on the columns and thereby reduced the column drift capacity. In Mr Harding's opinion, this was "the key to why the building failed as it did". Mr Harding said that, in hindsight, the structural design of the CTV building was vulnerable to the effects of severe vertical acceleration. He provided calculations showing levels of load on columns that indicated they were susceptible to failure when exposed to the level of loading produced by the February earthquake. He said it was possible that failure could have been initiated at any level in the building and that, once one floor failed, all of the floors would have pancaked.

Mr Harding was one of several witnesses that referred to the potential adverse effects of vertical accelerations on structural performance. That issue is addressed in section 7.3.1, and Mr Harding's evidence is considered in more detail in section 7.3.1.3.

7.3 Contributors to collapse

Dr Reay gave evidence that "there are at least five scenarios" that in his opinion had not been adequately considered in relation to potential reasons for the collapse of the building. One of these, which related to building modifications, has been considered in other sections of this Report. The other four scenarios are addressed in this section.

7.3.1 Vertical accelerations

7.3.1.1 Introduction

Vertical accelerations alone have been considered as a primary cause of collapse, most prominently by Mr Harding. It has been recognised by many expert witnesses that the contribution of high vertical accelerations would have had a detrimental effect, exacerbating weaknesses in the structure.

The seismicity and ground motion aspects of the 22 February 2011 earthquake have been examined in Volume 1 of this Report and details of the ground motions and associated response spectra are given in the report that we commissioned from Professor Carr[6], "Inelastic response spectra for the Christchurch earthquake records". In Volume 1 we note that the shallowness of the rupture and its proximity to the city contributed to the high vertical accelerations. Basin and topographical effects and the high water table are likely to have added to the shaking in the earthquake. Vertical accelerations reached 2.2g, with horizontal accelerations of 1.7g in the Heathcote Valley (which was near the epicentre) and up to 0.8g in both the vertical and horizontal directions in the CBD. Three recommendations relating to vertical ground motion have been made previously by the Royal Commission. These are reported in Volumes 1 and 2 of this report.

All buildings in the Christchurch CBD were subjected to vertical accelerations, however only the CTV building collapsed completely, in such a way that all slabs ended up on top of each other, leaving little chance of survival. Dr Reay accepted that it was the only building that collapsed in that manner.

Counsel acting for ARCL and Dr Reay submitted that it will never be possible to say with certainty how the CTV building site reacted in the February earthquake, despite the investigations that they commissioned. ARCL and Dr Reay arranged for a strong ground motion instrument to be deployed at the CTV site in March 2012. Dr Brendon Bradley noted that the ground motions recorded from this instrument could be compared with those concurrently observed at nearby Geonet strong motion stations in order to ascertain whether there were any peculiarities in the ground motions at the CTV site. The ground motion amplitudes of earthquakes measured after March 2012 were small relative to the September and February earthquakes. Therefore the effect of non-linear soil response was not as evident in the recordings for these smaller earthquakes.

Dr Bradley co-authored a paper[11] published in the *Bulletin of the New Zealand Society for Earthquake Engineering* on the strong ground motions observed in the February earthquake. In that paper it was noted that the large vertical ground motions observed were the result of the significant number of near-source recordings rather than any event-specific features. In a technical report[12] prepared for the Royal Commission, Dr Bradley assessed the ground motion aspects of the February earthquake related to the CTV building. Figure 104 illustrates the response spectra of vertical ground motions at the four CBD measuring stations in proximity to the CTV site. High vertical accelerations were recorded in the short period range, below 0.5 seconds. Dr Bradley's report established that the results of the four Geonet stations (that is, CCCC, CHHC, CBGS and REHS) were appropriate to model the ground motion at the CTV site in the September and February earthquakes. The REHS site had not been included in the first Compusoft analysis[13], which formed the basis of the Hyland/Smith report. The Bradley report did not indicate any unique peculiarities at the CTV site, which would have substantially increased site ground motions.

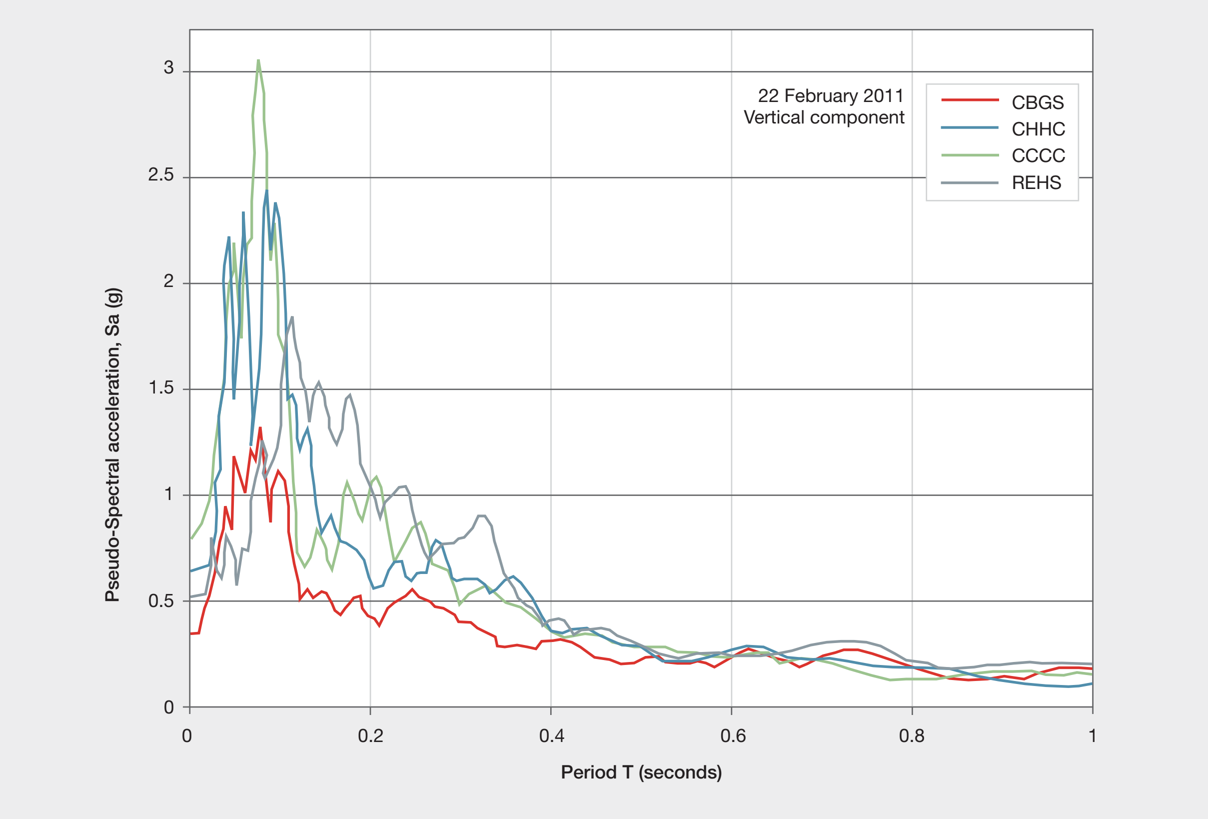


Figure 104: Response spectra of vertical ground motions observed during the 22 February 2011 earthquake (source: Brendon Bradley[9])

7.3.1.2 Non-linear time history analyses

The effects of vertical accelerations were modelled in the Compusoft[13] non-linear time history analyses. In the second report[3], prepared for the Royal Commission, the maximum variation in column axial load is shown in Figure 105. This Figure gives the predicted values for the internal column at the intersection of line C and line 2 when the analyses were run using the ground motion records obtained at the CBGS and CCCC Geonet stations.

Compusoft found that the CCCC ground motion record produced a much larger variation in the predicted axial loads in the columns than the corresponding values obtained with use of the CBGS record. This can be attributed to the difference in frequency components between the two records and the magnitude of the vertical accelerations present in each. Compusoft noted that the peak axial demands may not be concurrent with the peak bending actions that occur as a result of building drift. Consequently, when assessing vertical earthquake demands, consideration should be given to concurrency of actions.

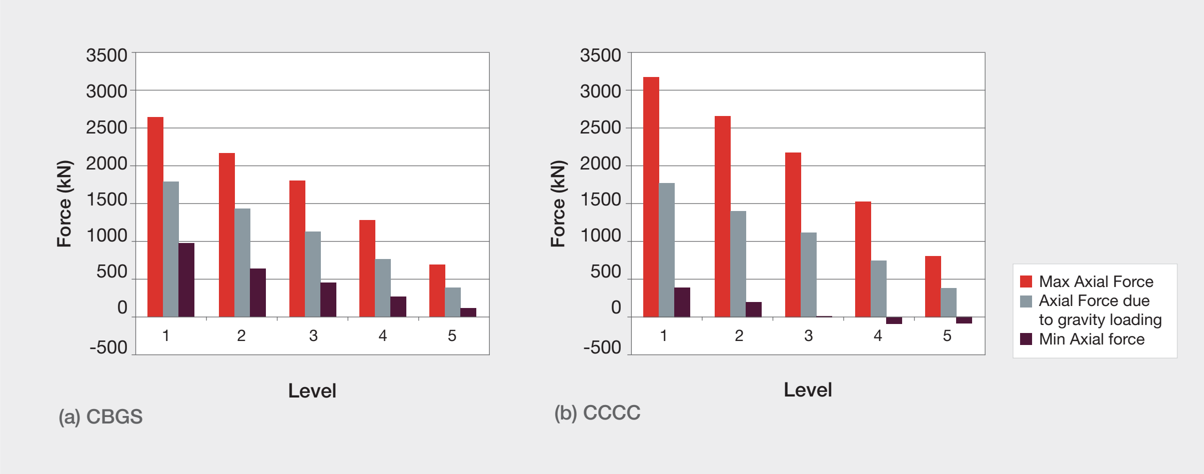


Figure 105: Column C2 axial load variation, 22 February earthquake (source: Compusoft Engineering)

Mr Smith (co-author of the Hyland/Smith report) presented in evidence a plot of the concurrent actions of shear, axial load and bending moments in a column at level 1 at the intersection of lines D and 2 during the February earthquake. The axial load (blue line), shear force (red line) and bending moment (green line) plotted against time are shown in Figure 106.

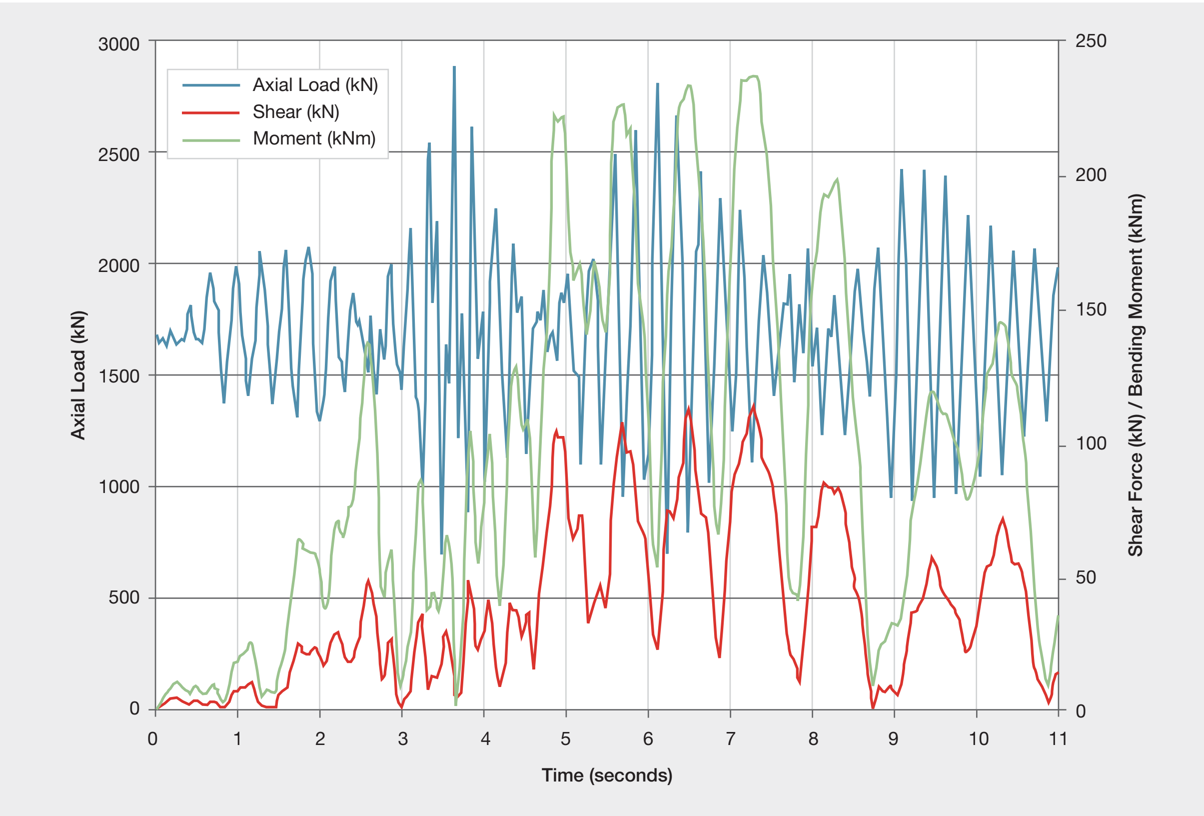


Figure 106: Predicted concurrent actions in column D-2 at level 1

The design column gravity load assumed to apply with seismic load combinations is around 1700kN, which is the starting point for the blue line. The vertical oscillations are caused by vertical accelerations. Mr Smith mentioned that the damping value adopted was low and higher damping was used in subsequent analyses. He noted that vertical vibrations tend to decay earlier with higher damping values and there is therefore probably a slight overestimate of the effect of vertical accelerations. Mr Smith explained that this analysis is not exact and requires consideration of trends and general performance indicators. In particular he was looking for concurrent peaks of actions which may give critical conditions.

Dr Bradley stated that the effects of the sub-surface soils were modelled by Compusoft simply as linear springs with tension gapping. This soil-structure modelling was crude in comparison to the non-linearities modelled in the structural elements. He said at the least, a sensitivity study should have been considered. Soil non-linearity occurs at infinitesimal strains, and therefore plastic deformation of soils is essentially always occurring. Dr Bradley indicated that the elastic springs representing soil deformation did not match actual behaviour, which would have been inelastic from low strain levels. However, more refined models were not readily available.

We consider that the soil-structure interaction can have a significant influence on the vertical accelerations transmitted into the structure. Inelastic deformation of the soil should increase effective damping and this might significantly reduce vertical excitation actions.

Furthermore Dr Bradley explained that the beam-column joint model used by Compusoft did not consider the time varying effect of axial load, which is known to be significant as a result of significant vertical acceleration.

Mr William T. Holmes agreed that vertical accelerations probably had some detrimental effect on the overall response of the CTV building in the February earthquake. However, he had not seen a method of definitively combining the vertical effects with lateral effects. He said it is very hard to do and had doubts as to how the non-linear time history modelling would try to combine all these things.

7.3.1.3 Mr Harding

Mr Harding stated that vertical acceleration increased axial loads on the columns, thereby reducing the column drift capacity and that this was an explanation to why the CTV building failed as it did. He went on to note that a number of eyewitnesses had referred to an upwards jolt at the outset, which was a manifestation of vertical acceleration. This was consistent, in his view, with vertical acceleration effects overloading all of the columns on one storey at a similar time causing sudden and rapid collapse rather than a progressive collapse.

Mr Harding referred to a photo taken by Mr Becker (see Figure 107), which shows books stacked up on steel trestle tables in the IRD building, at 224 Cashel Street directly opposite the site (we discuss the IRD building in section 6.5.3 of Volume 2). He observed that the lateral forces have not been sufficient to dislodge the books, however it appears the tables have a major sag in the middle “as if the books had tripled in weight”. He said this showed the site was subject to very high levels of vertical acceleration.

Figure 107: Bookshop in the IRD buildings after the February earthquake (source: Ross Becker)

Mr Harding said there was no provision for vertical acceleration in buildings in NZS 4203:1984[14] and, in hindsight, the structural design of the CTV building was vulnerable to the effects of severe vertical acceleration because it had heavy floors. In evidence he gave his prediction of the actual axial loads on the internal columns due to the recorded vertical accelerations. He also reproduced his 1986 calculation of the design axial load. At level 1 (ground floor) he had the following loads:

- dead load, D = 1522kN;
- reduced live load, L_R = 352kN;
- total unfactored load, D + L_R = 1874kN; and
- design load, 1.4D + 1.7L_R = 2729 kN.

Mr Harding then stated that the column capacity as designed was 3100kN. In Mr Harding's 1986 calculations he used axial-moment interaction column design charts to check that the column capacity was greater than demands. However his calculations did not satisfy Clause 6.4.1.5 of NZS 3101:1982[15] for the maximum design axial load in compression. This clause required the design axial load to be less than $0.8\phi P_0$, which equates to 2460kN where P_0 is the crush load for the column. This value is less than the design load of 2729kN. Therefore Mr Harding's column design in 1986 did not comply with NZS 3101:1982.

Mr Harding claimed that the axial loads induced by vertical accelerations recorded at two adjacent ground motion stations had been shown to be well above the column capacity (which he stated as 3100kN), and were high enough to initiate failure. He noted that the CCCC and PRPC stations recorded 0.79g and 1.88g vertical acceleration, which gave actual loads on the column of 3354kN and 5397kN, respectively. However, the PRPC strong motion site is located well outside the CBD and close to the epicentre of the February earthquake. It was not considered appropriate for comparison to the CTV site in a report by Dr Bradley[12]. We agree with Dr Bradley.

Professor Priestley explained the complexities of estimating the effect of vertical movements. He said it is unlikely that the different floor levels and the different bays of the floors at a given level would respond synchronously. Thus determining the effect of vertical acceleration by simply factoring up the axial loads on the columns by an assumed vertical acceleration response factor would be an extremely crude approach. The second set of NLTHA analysis[3] used four ground motion site records, with the REHS being included. The Compusoft analysis (see Figure 105 above) gave a maximum axial load of around 3200kN for an internal column at level 1. Mr Harding calculated an actual column load of 3354kN from vertical acceleration, which was close to the predicted NLTHA value. However the axial load at which concrete crushing occurs is above this value of 3354kN. With no strength-reduction factors the column capacity is of the order of 4400kN, assuming f'_c of 35MPa with a column diameter of 400mm and six H20 longitudinal bars. Further concrete strength testing has found the concrete to be at or above the specified strength (see section 2.3.4).

The Royal Commission does not consider that vertical acceleration by itself was a primary cause of collapse, principally because the maximum axial column demand (in the order of 3200–3400kN from the Compusoft analysis and Mr Harding's calculation) is less than the column concrete crushing strength of 4400kN at level 1. The demand calculated by Mr Harding is based on the peak ground motion. The Compusoft NLTHA analysis is a more refined estimate. It should be noted that the Compusoft estimate (~3200kN) is the peak, or maximum, over the whole earthquake record and that vertical movements are high-frequency (rapid) motions. Furthermore the speed of loading and lower characteristic concrete strength specified may give a column capacity higher than theoretically calculated.

Mr Harding contended that if the building had been designed for the degree of vertical acceleration that it was subjected to, it would have made a significant difference to the size of the columns. However an increase in column size may not have had the positive effect Mr Harding was implying, because of the increased seismic actions that the stiffer member would have attracted.

7.3.1.4 Dr Reay

Dr Reay expressed the view that the lateral load resistance, or stability, of the southern wall was dependent on the gravity restoring force provided by the floor, and vertical accelerations would potentially increase or decrease this force. He said that if this force was substantially diminished at the same time as there was a significant lateral load on the wall, the wall would tend to commence overturning and allow a significant rotation in the south side of the building. He said collapse initiated by this scenario is highly feasible.

We note that the south wall tributary area for gravity loads is about a fifteenth of the total floor area, while close to half of the floor contributes lateral seismic inertial forces on the wall. Consequently the gravity loading is relatively small compared to the lateral seismic inertial forces.

Dr Reay anticipated a fundamental building period of 1.2 seconds in the east-west direction. The fundamental period for the vertical stiffness of the wall is much lower, about 0.2 seconds. The response spectra for vertical accelerations (see Figure 104) give a peak vertical acceleration of between 1.0g and 0.5g.

We note that the acceleration in the upward direction would last for 0.1 seconds before reversing in the opposite direction for a further 0.1 seconds. At the same time the wall would be vibrating backwards and forwards with a period of about 1–1.2 seconds. The vertical upward acceleration would increase the axial load on the wall and increase its lateral strength for the 0.1 seconds followed by a decrease in lateral strength when the downward acceleration occurs in the following 0.1 seconds. The overall effect on the lateral performance of the wall would be negligible.

We do not consider that the high-frequency accelerations can have had any significant effect on the overall stability of the south wall.

7.3.1.5 Professor Mander

Professor Mander described the February earthquake as having exceptionally high vertical accelerations. In response to comments from Professor Priestley he conceded that the September vertical accelerations were very high, but not exceptionally high. Professor Mander considered that while the exceptionally high vertical motions were not the sole cause of failure, they would tend to vibrate vertical load bearing elements, such as the columns and floor slabs, and add considerably to the resulting damage.

Under questioning from Mr Mills QC, Professor Mander accepted that no one had established exactly what the effects of the vertical accelerations were. He agreed that the way a building reacts to vertical forces is affected by factors such as previous cracking in the floor slabs, how those floors respond, distribution of live loading and the axial flexibility of the columns. This makes it a complex task to estimate the vertical period. Professor Mander agreed that the CTV building had a significant number of structural weaknesses and that vertical accelerations would have exacerbated these existing weaknesses. Dr Bradley observed that the vertical accelerations increase the demand imposed already by horizontal ground motions and that if a structure is particularly vulnerable, for example because of a lack of ductile detailing, then the additional effect of vertical accelerations will probably be more important than for structures that are well-detailed.

Many of Professor Mander's collapse scenarios included concurrent vertical vibrations; however, these were ancillary to the primary mechanism. Professor Priestley did not find any of Professor Mander's scenarios very convincing, considering that they rested on a number of assumptions that were asserted as facts. Professor Mander claimed it was "inevitable that the two displacement and force maxima would coincide momentarily producing extremely high loading and stress demands on the materials". Professor Priestley disagreed with this statement, citing the non-linear time history analysis results to show this was not strictly correct. Professor Mander later said that a vertical load peak will coincide with large drifts, but acknowledged that the peak values will not coincide exactly.

Mr Brian Kehoe did not consider that vertical accelerations in the September earthquake would have had any effect on the building because the amount of vertical acceleration was relatively small compared to the design level gravity forces that the structural elements were designed to resist. While the February vertical accelerations were significantly higher, he did not believe they were the primary cause of failure because they were rapid movements and they had to overcome the design gravity load of the elements. He referred to studies he had been part of in the past, including analyses of several buildings for the effects of vertical acceleration. These were found to be minor compared to the vertical design loads (that is, the maximum gravity and live load combination) of structural elements.

Mr Holmes commented on Professor Mander's suggestion that the floor slab connection to the precast beams was broken at the negative moment areas over the beam during the September earthquake. While he was unsure what Professor Mander meant by "broken", he thought it unlikely that this could occur given the ductile reinforcing in this location. He assumed it to mean that the moment fixity to the beam was broken, resulting in an increase in the displacements. Professor Priestley said if Professor Mander was suggesting that the reinforcement yielded and reduced the moment fixity, then the displacement would be much less than the 500 per cent claimed by Professor Mander. Hysteretic behaviour would have further reduced response by increasing the effective damping associated with inelastic deformation. Professor Mander conceded that it would only have been an increase of 400 per cent if the reinforcement had fractured reducing the support conditions to that of a simple support.

The Royal Commission's calculations show that the vertical accelerations would not "break" the moment connection of the slabs to the beam as the accelerations do not appear to be sufficient to reverse the direction of the bending moments at the supports. Furthermore we do not accept that the ductile 12mm bars above the beams would fracture under the action of gravity loading and vertical seismic forces. It seems unlikely that the floor slab would fail in P-delta actions, which would be required to trigger this failure mechanism.

7.3.1.6 Other opinions on vertical ground movements

Mr Graham Frost, whose evidence we discussed in section 5.1, postulated that beam-column joint failure was a very likely scenario if the building was subjected to very high vertical accelerations. He also raised the possibility that vertical accelerations may have been high enough to raise the gravity load moments in the slabs to the level where the slabs failed in simple bending, either through the loss of bond between the metal decking and slab concrete or through tension failure of the metal decking.

We consider that the reinforcement in the slab would have been sufficient to prevent complete collapse even if the metal tray had fractured and therefore we do not accept the postulate that the slabs could fail. However, we do accept the possibility that the high vertical accelerations may have influenced failure of the beam-column joints (see section 6.3.5).

Professor Priestley saw photos of column failures that he thought tended to indicate failure occurred at the top of the splices, probably due to high vertical compression force. However, in his opinion a more likely collapse scenario is that which is reported in section 7.2.2.

Professor Carr stated that based on the revised non-linear time history analysis, the building collapsed due to the inability of the heavily loaded interior columns to carry their imposed vertical loads when subjected to the effects of inter-storey drifts. He said that a large variation of actual forces due to the very large vertical accelerations may also have had a part to play in the column failure.

Counsel assisting the Royal Commission submitted that while vertical forces may well have had a contributing role, the exact vertical forces to which the building was subjected are not known. Counsel for Dr Reay and ARCL contended that focusing on “critical structural weaknesses” minimised to an unreasonable degree the important effects of extremely high vertical accelerations.

We accept that vertical seismic forces may have had an influence on the collapse mechanism and this is recognised in section 6.3.5.

7.3.2 Cumulative damage, low-cycle fatigue, strain hardening and strain ageing

The phenomena of cumulative damage, low-cycle fatigue and strain hardening have been referred to on numerous occasions during the course of the hearings. They are discussed in section 3.5.7 in the context of Mr Coatsworth’s inspection of the building after the September earthquake. They are examined as a probable collapse scenario or contributor to collapse. There was some lack of clarity in the discussion of these concepts in the evidence of the expert witnesses.

7.3.2.1 Cumulative damage

Dr Reay said that during the Canterbury earthquake sequence he had observed cracks in several buildings that were originally limited in extent and crack width. Over time these cracks had gradually increased in number, length and width. He gave the shear walls in the IRD building as an example. He said this change had occurred progressively due to aftershocks. In his opinion, the ongoing sequence of aftershocks continued to cause cumulative damage to reinforced concrete buildings, each significant aftershock reducing the capacity of the building to some extent.

In the United States, guidance on evaluating the damage and analysing the future performance of concrete wall buildings is given in FEMA 306[16]. This report was prepared by the Applied Technology Council (ATC-43 Project) in 1998. Mr Kehoe and Professor Mander were both ATC-43 Project participants. Mr Kehoe explained that studies made as part of the FEMA 306, 307 and 308 process examined whether previous earthquake damage would affect a building’s ability to resist further earthquakes. The prologue section of this report concluded as follows:

> **Damage may not significantly affect displacement demand in future larger earthquakes.**
>
> One of the findings of the ATC-43 project is that prior earthquake damage does not affect maximum displacement response in future, larger earthquakes in many instances. At first, this may seem illogical. Observing a building with cracks in its walls after an earthquake and visualizing its future performance in an even larger event, it is natural to assume that it is worse off than if the damage had not occurred. It seems likely that the maximum displacement in the future, larger earthquake would be greater than if it had not been damaged. Extensive nonlinear time history analyses performed for the project indicated otherwise for many structures. This was particularly true in cases in which significant strength degradation did not occur during the prior, smaller earthquake. Careful examination of the results revealed that maximum displacements in time histories of relatively large earthquakes tended to occur after the loss of stiffness and strength would have taken place even in an undamaged structure. In other words, the damage that occurs in a prior, smaller event would have occurred early in the subsequent, larger event anyway.

Mr Kehoe’s opinion, after reviewing the damage assessment made by Mr Coatsworth, was that the 4 September and subsequent aftershocks up to the time of the inspection on 29 September 2011 had no effect on the capacity of the building. Mr Kehoe explained that a finding of the ATC-43 investigation was that, in some cases, damaged buildings performed better in a subsequent earthquake because they were "softer". In some cases the damaged buildings performed slightly worse. However, there was no real evidence that damage in one event would cause a building to perform worse in a future earthquake of similar magnitude.

Professor Mander agreed that in order for the CTV building to have undergone relevant low-cycle fatigue it must have sustained physical damage, although that damage might have been hidden. He considered damage was not "hidden" simply because it was not readily examinable. Under questioning Professor Mander said that more sophisticated techniques, such as ultrasonic tomography, may be necessary to confirm there is no damage. He claimed that hidden damage may include reinforcing steel that appears from visual inspection to be in good condition after many cycles of loading but on a further cycle a fatigue crack may appear in the steel. He cited the beam-column joint as a region where hidden damage may occur but conceded that tension induced into the joint zones by reinforcement that was anchored by hooks in the mid-region of the joint could cause cracks to form. He noted that the presence of the precast elements may have hidden this cracking from view. Mr Coatsworth inspected the beam-column joints on the lower levels and the external beam-column joints and did not see any visible signs of distress.

The NLTHA panel produced two sets of comparative analyses to assess the likely effect of the damage sustained in the September earthquake on the building's performance in the February earthquake. It was found that there was very little difference in projected performance assuming the building was undamaged at the start of the February earthquake and analysing the building subjected first to the September and then to the February earthquake. In the September earthquake some inelastic deformation was predicted to occur and potentially some drag bar disconnection may have occurred.

Dr Bradley noted that the NLTHA modelling limitations were such that cumulative damage could not be assessed in the beam-column joint models. Professor Carr said such beam-column joint models were not available in SAP2000, the programme used in the analyses. He also noted that there are no recognised techniques for modelling two-way interacting beam-column joints and there is little research on the structural configuration of the joints used in the CTV building. Therefore, there will always be considerable uncertainty as to the adequacy of any joint model. Professor Carr stated that if accumulative degradation is considered then more damage will concentrate in the joint and less in the columns.

We note that comparative non-linear time history analyses were made using a simplified analytical model of the Hotel Grand Chancellor. These analyses are discussed in section 3.5.3 of Volume 2. The analyses indicated that the predicted damage sustained in the building in the September and Boxing Day earthquakes made very little difference to the predicted performance of the building in the February earthquake.

Professor Mander[17] produced a table of significant aftershocks felt in Christchurch earthquakes between 4 September 2010 and 22 February 2011. Significant earthquakes for the purposes of this list were those of magnitude 5 or higher. This table gave the peak ground acceleration but it gave no indication of the associated response spectra, information that is required to gauge the likely structural effects of the earthquakes. A letter[18] dated 15 June 2012 from counsel representing ARCL and Dr Reay suggested that the NLTHA should include modelling of significant earthquakes between September and February so that cumulative fatigue effects could be considered. The only comments from the NLTHA panel came from Professor Carr and Dr Barry Davidson. Professor Carr noted that magnitude 5 earthquakes are two magnitudes smaller than the September earthquake and hence a thousand times smaller in terms of energy released. Most earthquakes on Professor Mander's list were at similar distance from the CBD as the September events, with the exception of the Boxing Day 2010 earthquake. A study by Professor Carr using the Boxing Day earthquake on a similar shear wall building in the CBD showed that the building's response to that earthquake, when compared with the September and February earthquakes, was insignificant at the "noise level". Dr Davidson commented on the significant time it would take to perform an analysis that put the building through many more earthquakes.

We consider it probable that any damage that may have occurred in prior earthquakes would have been induced in any event in the early stages of the subsequent larger February earthquake. In our opinion any pre-existing damage would have made very little difference to the performance of the building in the February event, and any damage that had been sustained in previous earthquakes was not a primary cause of collapse.

We note from our study of the representative sample of buildings, which is described in Volumes 2 and 4, that there are two types of structure.

- There are brittle structures, which give little warning in terms of apparent damage before collapse occurs. These buildings include the PGC building, unreinforced masonry buildings and concrete buildings, which have low reinforcement contents such that yielding of reinforcement in a wall is confined to a single primary crack.
- The second type of building is ductile. In these buildings cracks may form and extend in aftershocks but because the reinforcement is ductile there is no loss of strength. There can be some stiffness degradation. These ductile structures are relatively insensitive to accumulated damage, as this can only occur by low-cycle fatigue. We note that while low-cycle fatigue has been seen in tests it only occurs under extreme conditions (see section 7.3.2.2).

7.3.2.2 Low-cycle fatigue and strain hardening

Dr Reay stated that "reinforcing strain hardening" was a "collapse consideration", which required further attention. He said ARCL had found reinforcing steel that had been subject to strain hardening in several shear wall buildings with bar yielding being limited to a very short length, about 1–2 bar diameters, along the reinforcing steel. He indicated that this short length of yielding prevented the extent of elongation necessary to reach the level of ductility assumed in the code. He referred to a loss of capacity of about 40–50 per cent, depending on the degree of strain hardening.

In Dr Reay's view strain hardening in the CTV building could have potentially caused a materially different response than that predicted by analysis. He stated that the floor diaphragms and shear walls in the building may have been subject to reinforcing fracture.

We note that the issue of strain hardening and the premature failure of reinforcing bars due to strain concentration in some shear walls was considered in some detail in section 8 of Volume 2 of our Report. This is not an issue that could affect the performance of the structural walls in the CTV building as it can only arise where reinforcement content is low. This is not the case in the structural walls of the CTV building.

We note that strain hardening is an essential feature in the design of reinforced concrete as it causes yielding to propagate along the length of the reinforcing bar, allowing plastic hinges to form. The higher the strain hardening characteristic of reinforcement, the more the yielding will spread. Questioned by Commissioner Fenwick, Dr Reay agreed that strain hardening actually had beneficial effects and it is not strain hardening "per se" that is modifying the propagation of yielding through reinforcing.

The phenomenon of strain hardening alone has no adverse effects on the structure and therefore it did not contribute to collapse. Professor Mander proposed low-cycle fatigue as a surrogate for strain hardening. He said that any steel that is strained under reverse cyclic loading is suffering low-cycle fatigue.

In previous hearings the low-cycle fatigue phenomenon was described by a simplified paper clip analogy. If the paper clip is bent back and forth it will eventually break after a few cycles. Professor Mander explained that structures that undergo extreme loadings such as occur in earthquakes can sustain low-cycle fatigue.

The material behaviour of steel for low-cycle fatigue is in the inelastic (post-yield) range. Professor Mander agreed that the number of cycles that the reinforcing steel will go through before it starts to suffer strain damage depends on the amplitude of the strain to which it is exposed. Steel typically starts to yield at about 0.002 per cent strain and strain hardens at 1.0–2.5 per cent strain. In a paper written by Mander et al.[19] It was noted that if the steel is loaded to a strain of three per cent then it can undergo four full tension compression cycles before suffering fracture. This level of strain is well into the strain hardening range. Professor Mander said that if the September shaking was large enough to cause yielding but not strain hardening, the steel might be expected to undergo about 50–150 cycles before fracturing.

Mr Kehoe stated that in order for strain hardening to occur there has to be a large amount of strain in the reinforcing steel. For that strain to occur there necessarily has to be cracking in the concrete, which would be visible. Mr Kehoe explained that, even if hidden by non-structural elements, the effect of large cracking is such that it would be visible elsewhere in the finishes, and in other portions of the building. Mr Kehoe considered that the fact that Mr Coatsworth did not see cracking in some areas was an indication that any cracking that may have occurred was going to be small.

Dr Hyland said he observed low-cycle fatigue fractures in the 2010 Chilean earthquake. He said the steel tended to bend and buckle and he believed that, if low-cyclic failures had occurred in the September earthquake, there would have been visible evidence of major spalling of the concrete. However, there was no evidence of this.

We note that many tests of reinforced concrete beams and beam-column joints were carried out at The University of Auckland between 1980 and 2000. In many of these tests high strain levels (typically 20–50 yield strains) were induced in ductile longitudinal reinforcement under cyclic loading conditions. Low-cycle fatigue failure was observed to occur in only a few cases and only in situations where the bars had started to buckle and straighten out as they were subjected first to compression and then to tension during a load cycle. Buckling of the bars is always associated with extensive spalling as the buckling bars push the cover concrete away from the member.

While low-cyclic fatigue is a known phenomenon, we have seen no evidence that this occurred in the CTV building in the September or Boxing Day earthquakes. Evidence of high strain levels in the reinforcement would be associated with the formation of wide cracks in regions containing reinforcement and buckling, which would be associated with extensive spalling. Given the absence of such indications of damage and the short duration of strong motion in the earthquakes, we are satisfied that low-cycle fatigue was unlikely to be an issue in the performance of the CTV building.

7.3.2.3 Strain ageing

Strain ageing was not an issue that was specifically raised in the hearing on the CTV building. Potential problems with strain ageing in reinforcement were described in section 8 of Volume 2 of our Report. We understand that the sensitivity of reinforcement that is available in New Zealand has varied significantly between differing reinforcement sources over the years. A paper on strain ageing of some recently manufactured reinforcement was briefly reviewed in Volume 2 of our Report. The reported test results indicated that strain ageing could have some effect on the performance of the reinforcement. In the circumstances, we considered it advisable to see whether the reinforcement used in the construction of the CTV building could have been adversely affected by strain ageing.

Holmes Solutions Limited[9] was commissioned to investigate the strain-ageing characteristics of reinforcing steel from the CTV building. They reported that an appreciable number of research projects have been completed, which indicate that reinforcing bars can suffer the effects of strain ageing.

Holmes Solutions salvaged samples of 12mm diameter deformed reinforcing bars from the CTV building debris at the Burwood landfill. Two samples were cut from each reinforcing bar to give a matching pair. One sample from each pair was subjected to an initial level of pre-strain and then immediately tested to destruction. The other sample was pre-strained and then left to age for the desired period prior to undergoing a test to destruction.

It was found that two grades of 12mm diameter bars were used in construction. Thirty of the 36 samples had a lower characteristic yield strength of 517MPa with the other six samples having an average yield strength of 380MPa. The drawings specified grade, 380MPa for all the 12mm reinforcement in the building.

Holmes Solutions stated that results reported by previous researchers are often contradictory with regards to the extent of influence of strain ageing. Some of the reported research indicated changes of up to 25 per cent increase in peak stress and 30 per cent reduction in uniform elongation capacity.

The strain ageing testing results obtained from the CTV building reinforcing indicated the following trends:

- the ultimate strength of the strain aged steel samples was on average 5 per cent lower than non-aged samples;
- the average uniform strain capacity of the strain-aged steel was found to decrease when compared to the non-aged samples by nine per cent;
- there were no specific trends for both the ultimate strength and the uniform strain capacity of the samples with regard to the level of pre-strain applied to the steel, or the length of ageing period; and
- all the reinforcing bars that were tested were ductile, whether strain aged or not, and in all cases the strain at maximum stress exceeded 10 per cent. In many cases it was considerably greater than this level.

Our conclusion from these tests is that strain ageing of the 12mm bars in the CTV building would not have had a noticeable effect on the seismic performance of the building in the February earthquake.

7.3.3 Shake table testing

Dr Reay also gave evidence of his opinion that “there should be shake testing undertaken on a six degree-of-freedom shake table to investigate the overall behaviour and to recreate the structural failure of the CTV building”.

As testified by several witnesses, shake table testing would be an extremely difficult and expensive undertaking. If such a test was to be carried out it would be necessary to arrange for the use of a shake table in Japan or the USA. Then it would be necessary to build a scale model of the building. That would introduce numerous problems in terms of scale effects and therefore considerable doubt as to the validity of the test in representing the behaviour of the actual building. In such a model there would be a major issue in obtaining and installing strain and displacement equipment to effectively monitor the behaviour during the test.

We note that the weaknesses that led to the collapse of the building have been clearly identified. Some quasi-static model testing of the elements in the building might be justified. However, the urgent need for resources is not so much for research into structural weakness apparent in the CTV building but for finding other buildings that may contain similar faults.

7.4 Most likely collapse scenario

Our conclusions on the most likely cause of the collapse of the CTV building are based on a review of the proposed collapse mechanisms presented by the expert witnesses (as described in this section), our assessment of the design calculations and the different structural elements in the structure (described in section 6).

There were two major weaknesses in the structure, which are identified below:

1. Mr Holmes, Professor Priestley and Professor Mander all identified the beam-column joints as being potentially weaker than the columns. This is consistent with our own assessment. All of the beam-column joints in the building contained a basic flaw. In every case the bottom beam bars were anchored into the columns by 90° hooks, located in the mid-regions of the beam-column joints. For the internal columns on lines 2 and 3, there was a small overlap between the hooked bars. This overlap was not sufficient to enable the tension force to be transferred into the compression zone on the other side of the joint zone, which was essential for the load carrying capacity of the joint zone to be maintained under cyclic loading (see Figure 87 in section 6.3.5). In the external beam-column joints on lines A and F the hooked bars were anchored short of the mid-section of the beam-column joint. In both the internal and external joint zones flexural tension forces in the beam bars would create tensile stresses in the joint zone. When the concrete cracked in tension there would have been little left to stop the hooked bar from being pulled out of the joint zone (see Figures 89 and 90 in section 6.3.5). This would have led to rapid strength degradation and a near vertical collapse of the floor slabs and beams. We note that all of the joint zones were susceptible to this failure mechanism and it is not possible to identify which beam-column joint zone would fail first.

   The beam-column joint zones on lines 1, 2, 3 and 4 at the west wall (line A) were particularly vulnerable as both the top and bottom beam bars were anchored by hooks into the mid-regions of the beam-column joint zones. This vulnerability was pointed out by Mr Holmes and Professor Mander. There was little to hold the beams into the joints except one or possibly two 10mm ties once cracks had formed behind the hooked bars (see Figure 88 in section 6.3.5).

   The most critical joint zone is likely to have been located where the inter-storey drift was greatest. Due to the high lateral stiffness of the north wall complex the highest inter-storey drifts would be expected to occur on lines 1 and F. The beam-column joints on line A, for drifts in the north-south direction, were particularly vulnerable but there may initially have been lateral stiffness from the masonry infill walls, which would reduce the north-south drift on the beam-column joint zones on levels 2 and 3.

   There is a further possible failure mechanism for the internal beam-column joint zones on lines 2 and 3. As shown in Figure 87(e) in section 6.3.5, support to the beams may have been lost due to spalling of the concrete immediately below the beam support. Mr Frost suggested a mechanism similar to this due to the interface between the end of the beam and the in situ column concrete not being roughened, which prevented it from resisting shear by aggregate interlock action. We note that premature spalling of this supporting concrete ledge could have occurred by prising action of the beam when tension in the bottom beam bars opened up a crack at the interface between the in situ and precast concretes. This is discussed in section 6.3.5.

2. The second basic weakness was in the connection of the floors to the north wall complex. As discussed in section 6.3.3, there were errors in the design of the connections between the floors and the north wall complex. The design forces that were used to resist seismic forces in the east-west direction were approximately half of the values that would have been consistent with NZS 4203:1984. While the time history analyses predicted peak forces considerably in excess of the NZS 4203:1984 values, we consider it likely that, if connections had been provided through the use of ductile reinforcement, the ductility would have prevented complete failure of the connections. We note that the correct design values for seismic actions in the north-south direction were met after the addition of the drag bars. However, the drag bars lacked ductility and, as indicated by the non-linear time history analyses, the drag bar connections could be expected to have failed either in the September earthquake or near the start of the intense ground motion in the February earthquake.

We consider it likely that most of the drag bars failed in the early stages of the February earthquake with some tearing of the floors near line 4 in the region between walls C and D, as illustrated in Figure 108. Partial disconnection of the floors by the loss of the drag bars and the tearing of the floors, in this way, would have greatly increased the flexibility of the load path between the floor and the north wall complex. This would account for the lack of damage to the north wall complex. The tearing of the floor from the north wall complex would have been completed by the collapse of the floors. This process would account for the remnant portions of the floors remaining piled up against the north wall complex after the earthquake.

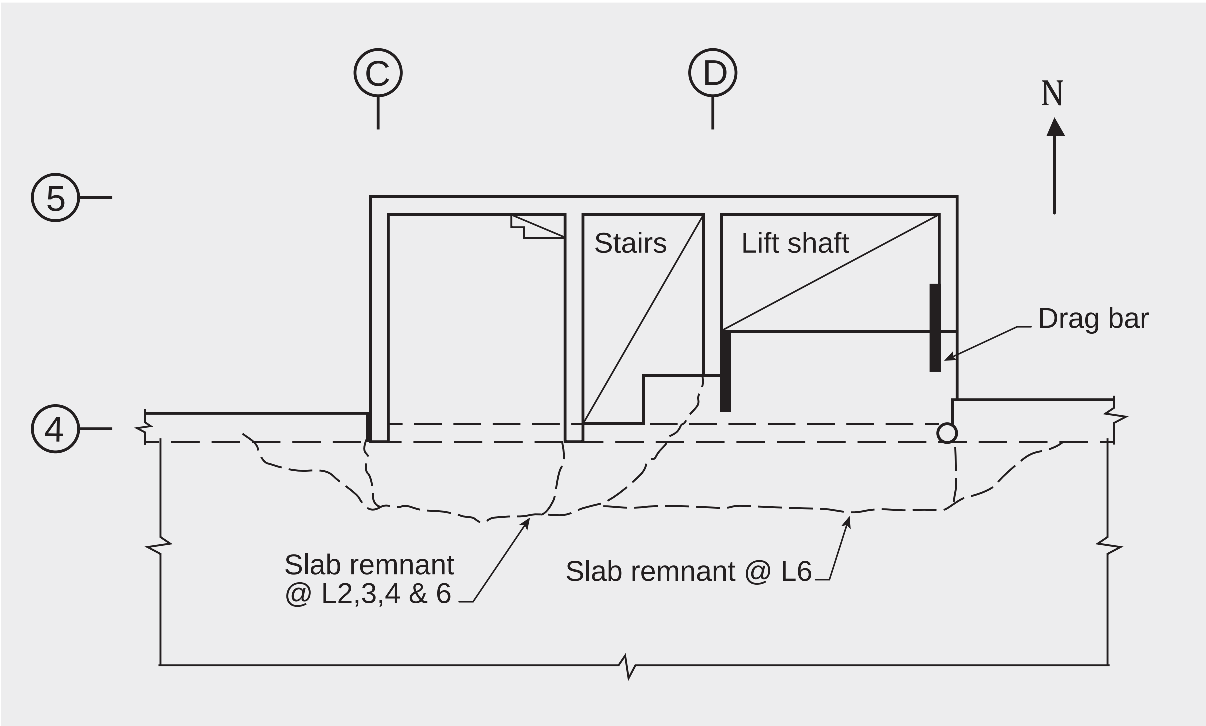


Figure 108: Remnant slab profiles at north wall complex after February earthquake (adapted from Figure 43 of the Hyland/Smith report)

The failure of the drag bars and the partial disconnection of the floors from the north wall complex would have led to an increase in the inter-storey drifts in the structure. It is probable that the inter-storey drifts would have been sufficient to cause the collapse of the joint zones without the partial disconnection of the floors from the north wall complex. However, it is likely that this partial disconnection accelerated the failure of the building.

It is not possible for us to identify the critical beam-column joint that started the failure. However we note that the joint zones where lines 1 and 2 intersect with line A are particularly susceptible to failure, under east-west seismic ground motion, due to the hooks from both the top and bottom beam reinforcement being bent into the mid-region of the joint zones. The east-west shaking in the February earthquake was particularly strong. The failure of these joint zones and the resultant collapse of the beams may have been the reason why the collapse occurred from the south to the north.

We note that if the joint zones did not fail first, collapse would still have occurred due to the brittle characteristics of the columns, arising from their lack of confinement reinforcement. In our opinion collapse of the building can be accounted for without allowing for the additional forces induced by vertical ground motion. While this motion may have contributed to the collapse of the building we do not consider it to be the reason for the collapse.

The basic mechanics of beam-column joints were studied in detail starting from the late 1960s. By the mid-1970s there was a good understanding of the basic load paths through beam-column joints [20].

In summary, 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 through 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;
  - ensure the columns were adequately confined so that they could sustain the required deformation without failure;
  - determine the correct tie forces between the floors and the north wall complex and to track the load path between the wall and the floors;
  - ensure the adequate connection of the walls on lines D and D–E to the floors to resist seismic actions in the north-south direction. While the addition of drag bars 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 builder failed to roughen the interface between the ends of the precast beams and the in situ concrete in the columns, so that shear could be transmitted across the interface by aggregate interlock action. This was not picked up by those responsible for monitoring the construction works.

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 eyewitness 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 come 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.

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2. Catenary action can be envisaged by a chain or cable. The member is suspended from its end points and resists vertical loads by hanging in tension.

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

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 105[1]. 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 95[2], was published in 1936. NZSS 1900[3] 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 4203:1976[4] dealt with general structural design and design loadings. This was superseded by NZS 4203:1984[5]. NZS 3101P:1970[6], which related to concrete, was subsequently replaced by NZS 3101:1982[7]. 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 report[8] 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 Williams[9], 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 1170.5:2004[10]), 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 report[11] 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.

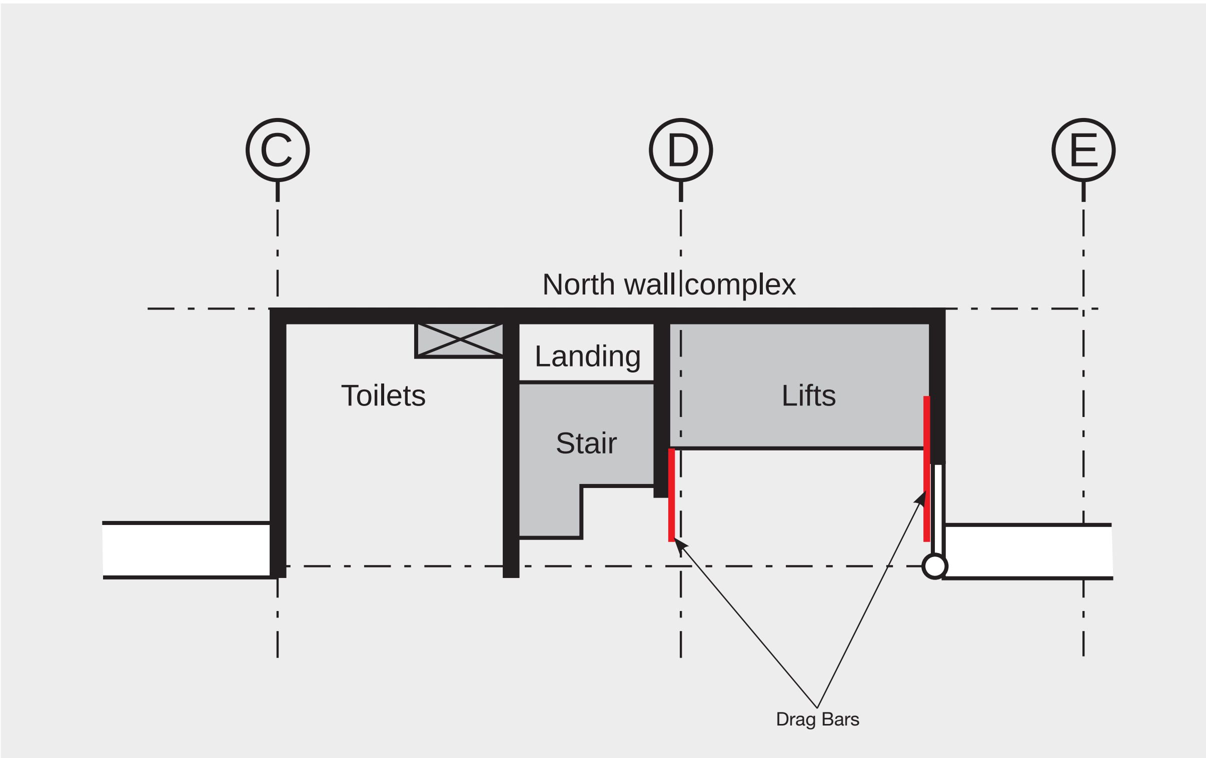

Figure 109: The north wall complex. The approximate locations of the drag bars, attached in 1991, are marked in red

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/Smith[12] 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 140m^2 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 $v\Delta$[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 3101:1995[14], 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":

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

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:

> 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 3101:2006[15], 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 $v\Delta$, specified in NZS 4203, and the assumptions of elastic behaviour.
>
> (b) Additional seismic requirements of this Code shall be met when plastic behaviour is assumed at levels of deformation below $v\Delta$.
>
> (c) Inertia loadings E_p shall be that specified by NZS 4203.
>
> (d) Loadings induced by the deformation of the primary elements shall be those arising from the level of deformation $v\Delta$, specified in NZS 4203 having due regard to the pattern and likely simultaneity of deformation.
>
> (e) Analysis may be by any rational method, in accordance with the principles of elastic or plastic theory, or both. Elastic theory shall be used to at least the level of deformation corresponding to and compatible with one-quarter of the amplified deformation, $v\Delta$, 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 $v\Delta$ 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 ($v\Delta$) 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 $v\Delta$.

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 $v\Delta$, the additional seismic requirements need not be applied. The calculation of $v\Delta$ 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. Δ 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.

"Δ" 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 (Δ). 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 vΔ

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 vΔ.

Counsel assisting referred to Clause 3.5.3.2 of NZS 3101 which 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*Δ.

Mr William T. Holmes said in his report[16] 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 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:

> 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.

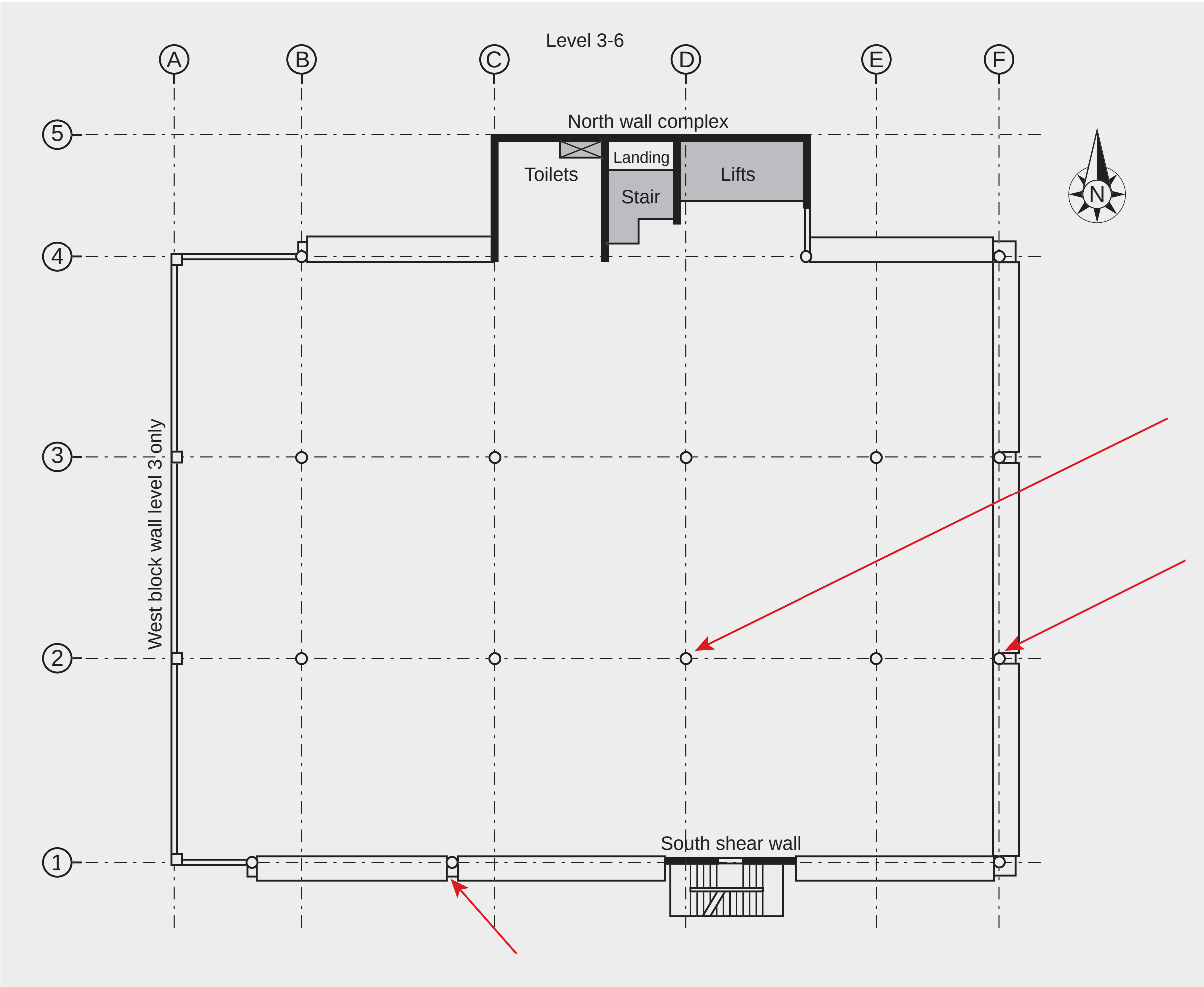

Figure 110: The indicator columns identified by Dr Hyland and Mr Smith were located at the intersection of gridlines D and 2, gridlines F and 2 and to the left of the intersection of gridlines C and 1

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, $v\Delta$) 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, $v\Delta$

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 $v\Delta$ 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 $v\Delta$.

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 2012[17] 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 $A_g f'_c$, 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/m^2 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 $v\Delta$ and whether the columns would be elastic at $v\Delta$; 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 $v\Delta$.

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 3109:1980[19], 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 tolerance 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, "$v\Delta$", 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 Paulay[20] 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 Standards Institute of New Zealand.

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.

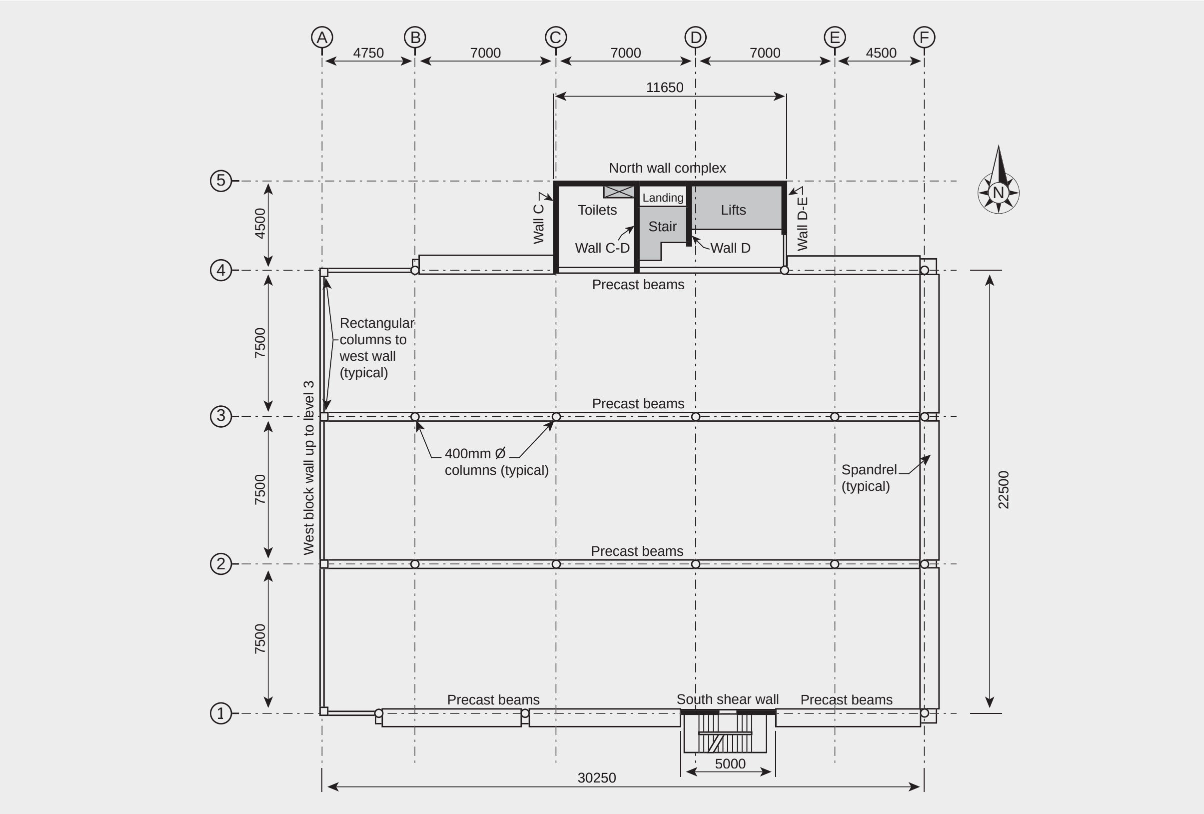

Figure 111: Typical upper floor structure

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 Code[2] 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 4203:1984[3]). 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 3101:1982[4]). 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.

Appendix 1:
List of people mentioned in this Volume

Acronyms used in this appendix

ARCE	Alan M Reay Consulting Engineer
ARCL	Alan Reay Consultants Limited
BE	Bachelor of Engineering
BSc	Bachelor of Science
CPEng	Chartered Professional Engineer
CCC	Christchurch City Council
DBH	Department of Building and Housing
ERSA	Elastic response spectra analysis
IPENZ	The Institution of Professional Engineers New Zealand
Hons	Honours
ME	Master of Engineering
MSc or MS	Master of Science
NLTHA	Non-linear time history analysis
NZFS	New Zealand Fire Service
NZSEE	New Zealand Society for Earthquake Engineering
PhD	Doctor of Philosophy
SESOC	Structural Engineering Society New Zealand
USAR	Urban Search and Rescue

Margaret Aydon worked for King's Education and was on level 4 of the CTV building when it collapsed.

Andrew Ayers is a member of USAR and carried out a visual inspection of the CTV building after the Boxing Day earthquake.

David Bainbridge visited level 6 of the CTV building after the September earthquake and gave evidence about damage he observed.

Geoffrey (Geoff) Banks carried out the engineering design that resulted in the installation of drag bars between the floor and the north wall complex on levels 4, 5 and 6. He has a BE (Hons), is a CPEng, a member of IPENZ, and is a former director of ARCL. He now runs his own engineering consultancy.

Neil Blair was a director and shareholder of Prime West Corporation, a property development company incorporated in 1983. Mr Blair, on behalf of Prime West Corporation, engaged Williams Construction to build the CTV building in 1986.

Bryan Bluck was the CCC's Chief Buildings Engineer at the time the CTV building was permitted and constructed. He had a BE, and was a registered engineer and a member of IPENZ. He is now deceased.

Dr Brendon Bradley was instructed by counsel for ARCL to provide expert evidence on concrete and ground motions. He holds a PhD in civil engineering and is a member of NZSEE and other professional bodies. He operates his own consultancy firm, Bradley Seismic Limited, which provides research and consultancy services in seismic hazard and risk analysis and structural and geotechnical seismic response analysis. He also lectures at the University of Canterbury in a number of areas including structural dynamics, engineering mechanics and mathematics, geotechnical earthquake engineering and seismic hazard and risk analysis.

Derek Bradley is a senior engineer for Compusoft Engineering Limited and was involved in carrying out a NLTHA of the CTV building for the Royal Commission. He was also a member of the NLTHA Panel. He has a BE Civil (Hons), is a CPEng, and is qualified as an international professional engineer.

Marie-Claire Brehaut worked for King's Education and provided evidence about damage to the CTV building after the September earthquake.

Michael Brooks was the Managing Director of Williams Construction when construction of the CTV building commenced in 1986.

Peter Brown was working for CTV at the time of the September and February earthquakes and gave evidence about damage to the building after the September earthquake.

Graeme Calvert is a former CCC building officer who was part of the team that carried out the Level 2 Rapid Assessment of the CTV building on 7 September 2010.

Elizabeth (Liz) Cammock works for Relationships Aotearoa and was on level 6 of the CTV building when it collapsed.

Bruce Campbell is the Director of Bruce Campbell Roofing, the company that was undertaking the weatherproofing of the west block wall on 21 and 22 February 2011. He provided evidence of the condition of the wall prior to the collapse.

Professor Athol Carr was engaged by the Royal Commission to act as a facilitator for the NLTHA and ERSA expert panels. He has an extensive background in the development and teaching of structural analyses. He is Professor Emeritus in the Department of Civil and Natural Resources Engineering at the University of Canterbury and has a PhD in engineering from the University of California, Berkeley. He is a fellow of IPENZ, a member of SESOC, a life member of NZSEE, and a member of the American Society of Civil Engineers. He is a member of the Faculty of the ROSE School (Centre of Postgraduate Training and Research in Earthquake Engineering and Engineering Seismology) at the University of Pavia, Italy, where he teaches a course in structural dynamics and earthquake engineering analysis, focusing on NLTHA.

David Coatsworth carried out an engineering assessment of the CTV building after the September earthquake. He is a senior associate of CPG New Zealand and has practised as an engineer specialising in civil and structural engineering for approximately 40 years. He has a BE (Hons), is a CPEng, an international professional engineer and a member of IPENZ.

Dr Barry Davidson is the Director of Compusoft Engineering Limited which carried out a NLTHA of the CTV building for the Royal Commission. He was also a member of the NLTHA Panel. He has a PhD in engineering, is a fellow of IPENZ and the NZSEE, and a life member and a past president of SESOC. He has 28 years of experience teaching and researching in structural engineering, specialising in structural dynamics, finite element theory and earthquake engineering.

Dr Andrew Dickson is a technical director with the civil structures section of Beca Infrastructure Limited. At the request of counsel assisting, he carried out an assessment of the tension capacities of the steel drag bars installed in the CTV building in 1991 between the floor slabs and north wall complex on levels 4, 5 and 6. He has a PhD in civil engineering and is a member of IPENZ, NZSEE, SESOC and the New Zealand Concrete Society.

William (Bill) Dray is a civil engineer working in the engineering services team of the building operations unit of the CCC. He has a BE and is a member of SESOC and the Building Officials Institute of New Zealand.

John Drew was the building manager of the CTV building at the time of its collapse and moved his medical centre (The Clinic) into the building in January 2011.

Alan Edge is the Director and Chief Executive Officer of Southern Demolition & Salvage Limited. He was asked to assist with provision of equipment to the rescue effort at the CTV building site on the afternoon of 22 February 2011.

Shane Fairmaid worked at ARCE as a draughtsman between 1981 and 1986.

David Falloon is the Principal of Falloon & Wilson Limited, who were engaged to do the structural engineering aspects of a fit-out of the building in 2000. He gave evidence in relation to the installation of an internal staircase between levels 1 and 2. He has a BE and is a member of IPENZ.

David Flewellen is a former CCC building officer who was part of the team that carried out the Level 2 Rapid Assessment of the CTV building on 7 September 2010.

Leonard Fortune was on a scissor lift working for Bruce Campbell Roofing on the weather proofing of the west block wall of the CTV building when it collapsed.

Graham Frost was on the CTV site from the evening of 23 February 2011 in his role as a USAR support engineer for the NZFS. His primary role in the days after the collapse was to assist in recognising and minimising the risks to USAR and police teams while performing their work. He has a BE (Civil), and has worked as a construction and structural engineer since 1978. He is the Chief Engineer for the Fletcher Construction Company, a fellow of IPENZ, and a member of the American Society of Civil Engineers. He is now a contracted USAR engineer.

Robert Gaimster is the Chief Executive Officer of the Cement and Concrete Association of New Zealand (CCANZ). He presented the CCANZ submission regarding the concrete testing section of the Hyland/Smith report. He has a BE (Civil) from Portsmouth Polytechnic/University and is a chartered civil engineer with the Institution of Civil Engineers (UK). He also has an Advanced Diploma in Concrete Technology and is a member of the Institute of Concrete Technology (UK).

Stephen (Steve) Gill witnessed the collapse of the CTV building from the roof of the Les Mills building (to the west of the CTV building).

Ronald (Ron) Godkin worked for King's Education and was on level 4 of the CTV building when it collapsed.

Stephen Grenfell witnessed the collapse of the CTV building from Madras Street (to the east of the CTV building).

Euan Gutteridge witnessed the collapse of the CTV building from the south-east corner of the Cashel and Madras Street intersection.

Douglas Haavik is a consultant engineer specialising in concrete and concrete materials, based in California. He has an MSc in civil engineering from the University of California, Berkeley. He is a registered civil engineer in California, a member of the American Concrete Institute, the American Society of Civil Engineers, and the International Concrete Repair Institute. He was instructed by Buddle Findlay on behalf of ARCL to provide independent expert advice on concrete-related issues relevant to the collapse of the CTV building.

David Harding worked for ARCE as a structural engineer between 1978 and 1980 and between 1985 and 1988. His qualifications and experience are discussed in detail in section 2.1 of this Volume of the Report.

John Hare identified the non-complying connections between the floors and the north wall complex of the CTV building in January 1990. At the time of his involvement with the building he was a senior engineer for Holmes Consulting Group. He is now a director of that company. He has a BE (Civil) (Hons), is a member of IPENZ and NZSEE, and is the President of SESOC. He is also a licensed professional engineer in California. He is currently acting Principal Engineering Adviser to the Canterbury Earthquake Recovery Authority.

Malcolm Harris works for CTV and provided evidence about damage to the building after the September earthquake.

Thomas (Tom) Hawker worked for CTV and witnessed the collapse of the CTV building from the middle of Cashel Street (to the south of the building).

John Henry was employed by ARCE as a structural engineer from 1984 to 1985. During this period he designed Landsborough House, which is discussed in sections 2.1 and 6.2 of this Volume. Between 1992 and 1995 he was employed by the CCC as an engineer, later becoming Building Control Engineer and Building Consents Manager. He has a BE (Hons) from the University of Canterbury and is a CPEng.

Dr Robert (**Rob) Heywood** is a forensic structural engineer from Brisbane, Australia who was at the CTV building site from the morning of 24 February 2011, acting as a USAR support engineer for the NZFS. He has a PhD in engineering, and was a principal researcher and senior lecturer at the School of Civil Engineering at the Queensland University of Technology from 1985 to 1998. He is a registered professional engineer in Queensland, a chartered professional engineer and is registered with the National Professional Engineers Register in Australia. He is a fellow of the Institution of Engineers Australia, and a member of the American Society of Civil Engineers and the International Association of Bridge and Structural Engineers.

Peter Higgins is the Southern Regional Manager for Construction Techniques Limited. He gave evidence of his observations of the CTV building in February 2011 while inspecting it in order to provide an estimate for crack repairs.

Marie Holland is a former CCC building officer who carried out a Level 1 Rapid Assessment after the Boxing Day earthquake.

William T. Holmes was engaged by the Royal Commission to peer review the Hyland/Smith and DBH Expert Panel reports. He is a Principal with Rutherford and Chekene, Consulting Engineers in San Francisco and has 45 years of practical experience in all aspects of designing structures, particularly designing for protection from earthquakes. Mr Holmes' experience includes post-earthquake reconnaissance and analysis, fragility and retrofit standards for unreinforced masonry and concrete buildings, development of seismic standards for both new and existing buildings, research and development of seismic technology, and performance-based seismic engineering. He has a MS in structural engineering from Stanford University, is a registered civil and structural engineer in California, and a registered professional engineer in Tennessee.

Terence (Terry) Horn worked at ARCE as a draughtsman between 1985 and 1995.

Lionel Hunter is the sole Director and a shareholder of Madras Equities Limited, which owned the CTV building.

David Hutt is a CCC officer who was seconded to the Royal Commission for the duration of the Inquiry and retained full access to the CCC electronic systems. At the request of counsel assisting, he searched CCC records for any other buildings in Madras Street built between 1985–1986. He is a team leader of building consents at the CCC, and has been a building officer since 1993.

Dr Clark Hyland is a Director of Hyland Consultants Limited and co-author of the Hyland/Smith report. Dr Hyland also prepared the site investigation and materials test report for the CTV building. He has a PhD in civil engineering from The University of Auckland, has been a registered engineer since 1989, and a CPEng since 2004. He has been involved in the formulation of New Zealand Standards and joint Australia/New Zealand Standards for steel structures.

Russell Ibbotson is a former Director and shareholder of Madras Equities Limited and acted as the manager of the CTV building from 1990 until March 2010.

Maryanne Jackson works for CTV and escaped from the CTV building during the February earthquake.

Dr Murray Jacobs is a consulting engineer with over 35 years of experience in the design of structures in Auckland. He was called by counsel assisting to give expert evidence on whether the CTV building complied with the Christchurch City Bylaw No 105 (1985) and the relevant Standards when the building permit was issued on 30 September 1986. He has a PhD in engineering and is a member of IPENZ, a CPEng, and an international professional engineer.

William (Bill) Jones was the original construction site foreman for the CTV building and was employed by Williams Construction and then Union Construction for the project.

Robert (Rob) Jury is the Technical Director of structural engineering with Beca Carter Hollings and Ferner and was a member of the DBH Expert Panel. He has an ME, and is a CPEng, and a fellow of IPENZ and NZSEE. He was a member of the committee that developed the Loadings Standard (including earthquakes) (NZS 4203:1992) and the current Earthquake Loadings Standard (NZS 1170.5).

Brian Kehoe and his colleague **Terrence Paret** are Associate Principal and Senior Principal, respectively, of Wiss, Janney, Elstner Associates Inc in California. Mr Kehoe and Mr Paret prepared a joint statement of evidence and Mr Kehoe appeared at the hearing. At the request of CPG New Zealand Limited, they reviewed the damage assessment performed by Mr Coatsworth on the CTV building after the September earthquake and were asked to provide an expert opinion as to whether the assessment undertaken by Mr Coatsworth was appropriate and whether his conclusions and recommendations were properly made. Mr Kehoe has an MSc in civil engineering, and is registered as a civil engineer in California, a professional (civil) engineer in Oregon, and a structural engineer in California, Hawaii, Oregon, and Utah. His areas of practice are earthquake engineering, structural evaluation, repair and rehabilitation design. Mr Kehoe is a steering committee member of ASCE-41 (Seismic Evaluation and Rehabilitation of Existing Buildings) and sits on the American Concrete Institute's committee ACI-374 for Performance-Based Seismic Design of Concrete Buildings (among other committees). He was a member of the American Society of Civil Engineers reconnaissance team which visited Christchurch after the September and February earthquakes. Mr Paret has an MSc in structural engineering, and has professional affiliations with a number of American organisations including the American Society of Civil Engineers, the Earthquake Engineering Research Institute, the Seismological Society of America, and the Structural Stability Research Council. His practice areas include structural performance evaluation, repair and rehabilitation design, seismic risk assessment and seismic repair and retrofit design.

Geoffrey Jones is the Manager of Opus International Consultants' (Opus) Materials Testing Laboratory in Christchurch, which carried out concrete testing for the Hyland/Smith report. Mr Jones has 42 years of experience as a laboratory technician undertaking mechanical testing of aggregates, soils, bitumen and concrete. He holds a Technicians Certificate (Civil) and is a Registered Engineering Associate.

Stephen Kissell is a service technician for Otis Elevator Company Limited. He provided access to a lift shaft in the CTV building on 18 February 2011 so that it could be inspected by Mr Graeme Smith. He provided evidence of his observations during that visit.

Nilgun Kulpe works for Relationships Aotearoa and was on level 6 of the CTV building when it collapsed.

Douglas Latham is a Structural Engineer for ARCL. He has a BE (Hons) and is a graduate member of IPENZ. He was involved in the collection of concrete samples from the CTV building debris at the Burwood landfill and prepared evidence as to the methodology of the sampling and chain of custody. He also carried out an ERSA analysis of the CTV building.

Phillippa Lee worked for The Clinic as a receptionist and was on level 5 of the CTV building when it collapsed.

Stephen McCarthy was the Environmental Policy and Approvals Manager for the CCC at the time of the earthquakes. He is now the Resource Consents and Building Policy Manager. During the state of emergency following the September earthquake he was one of the Building Evaluation Managers in the Christchurch City Emergency Operations Centre. He has worked for the CCC since 1 May 2006 and has 36 years of experience working for local government, including 16 years in building control. He has a degree in applied science and a postgraduate diploma in management from Massey University.

Dr Graeme McVerry is an engineering seismologist with GNS Science. He was a member of the ERSA and NLTHA expert panels convened by the Royal Commission. He has a PhD in Applied Mechanics and has been involved in research and consulting on seismic hazard assessment for over 20 years.

Dr James Mackechnie peer reviewed the concrete testing methodology used for the Site Examination and Materials Tests report prepared by Dr Hyland. He has a PhD and is a CPEng. He is an adjunct senior fellow in the Department of Civil and Natural Resources Engineering at the University of Canterbury and a council member of the New Zealand Concrete Society. He is also employed by Allied Concrete as its South Island Plant Engineer.

Professor John Mander is the Inaugural Zachry Professor in Design and Construction Integration I, at the Zachry Department of Civil Engineering, Texas A&M University. He was instructed by Buddle Findlay, on behalf of ARCL, to provide expert evidence to the Royal Commission on issues relevant to the CTV building, including: a review of the key findings of the DBH investigation; the results of new investigations commissioned by ARCL into ground motions, concrete tests and columns tests; an analysis of column performance; and alternative collapse hypotheses. He has a PhD in civil engineering, is a member of a number of organisations, including the American Society of Civil Engineers, the American Concrete Institute, the Earthquake Engineering Research Institute, NZSEE, and is a fellow of IPENZ.

Kendyll Mitchell and her two children were in the reception area of Relationship Services on level 6 of the CTV building when it collapsed.

Murray Mitchell is a senior structural engineer with Opus. He reviewed structural drawings for the CTV building in 1998/1999 when Opus was considering leasing the building and identified an issue with the connections between the floors and the north wall complex. He has a BE (Hons) and has 42 years of experience as a civil and structural engineer. He is a member of IPENZ and a past member of SESOC.

Daniel Morris owned Knock Out Concrete Cutters Limited between 1991 and 2000.

Peter Nichols worked as a structural checking engineer for the CCC from 1978 until 1984 and subsequently became the Riccarton Borough Engineer. He has a BE (Civil). He also observed the CTV building during construction and had a conversation with Mr Bluck about it.

Dr Arthur O'Leary is a retired structural engineer with 40 years of experience in the design and design management of commercial buildings with emphasis on earthquake engineering. He was called by the CCC to provide expert evidence on the Standards that applied when the building permit for the original construction of the CTV building was issued, the Hyland/Smith and Expert Panel reports and the peer review by Mr William T. Holmes. He has a PhD in civil engineering, and was a CPEng, and on the New Zealand register of the International Professional Engineers Register up until his retirement. He is a fellow of IPENZ and NZSEE, and a member of SESOC.

John O'Loughlin is a consulting engineer based in Christchurch with 42 years of experience in the engineering profession. He was asked by the CCC to provide expert evidence from the perspective of a practising structural engineer about the nature of the structural review undertaken by the CCC during the permit stage and whether the CCC engineers were in a position to identify the areas of non-compliance identified in the Hyland/Smith report. He has a BE (Hons), was a CPEng and an international professional engineer until December 2010, and is a member of IPENZ.

Leo O'Loughlin has worked for the CCC as a building officer since 1983. He received the original permit application for the CTV building, and gave evidence of the processes that applied to permit applications at that time.

Leonard Pagan is a quantity surveyor employed by Rawlinsons Limited. He accompanied Mr Coatsworth and Mr Drew on the engineering assessment of the CTV building in October 2010 and prepared a cost estimate for repairs.

Professor Nigel Priestley was Deputy Chair of DBH Expert Panel and provided expert evidence to the Royal Commission at the request of counsel assisting. He has been involved in the seismic performance of structures for more than 45 years. He has a PhD and has been a faculty member at the University of Canterbury, a Professor of Structural Engineering at the University of California, San Diego, Co-director of the European School for Graduate Studies in Reduction of Seismic Risk (the "ROSE" School). He has emeritus status at the University of California, San Diego and the ROSE School. He has also worked as a structural consultant. He is an honorary fellow of the Royal Society of New Zealand, fellow of the American Concrete Institute, fellow and past president of the NZSEE, and a fellow of IPENZ.

Dr Alan Reay is a structural engineer, was the Principal of ARCE and is a Director of ARCL. His qualifications and experience are discussed in section 2.1 of this Volume.

Phillip Reynish is the Managing Director of Reynish Decorators Limited, the firm carrying out painting on level 5 of the CTV building in January and February 2011. He provided evidence of his observations of the building during this time.

Trevor Robertson is the Senior Principal of Sinclair Knight Merz working in the role of Principal Structural Engineer (NZ). He was instructed by DLA Phillips Fox, counsel for Holmes Consulting Group Ltd, to provide expert evidence on issues relevant to the ethical, conduct and reporting obligations owed by engineers. He is a member of the IPENZ Ethical Complaints Investigating Committee. He has BE (Civil) (Hons), is a CPEng, a fellow of IPENZ, a member of SESOC and NZSEE and is registered as an international professional engineer.

Matthew Ross witnessed the collapse of the CTV building from the south-east corner of the Cashel and Madras Street intersections.

Anthony (Tony) Scott was the Quantity Surveyor and Project Development Manager for Williams Construction and then Union Construction during the design and construction of the CTV building.

Professor Robin Shepherd is a consulting engineer in New Zealand and California, and is Emeritus Professor of Civil Engineering at the University of California. He was instructed by Buddle Findlay on behalf of ARCL to provide expert evidence on forensic engineering practice, the evolution of seismic design standards, cumulative earthquake damage, seismic excitation of the building site, and dynamic analyses including NLTHA. He has a PhD in engineering and holds a number of professional memberships including the American Society of Civil Engineers, of which he is a fellow and life member, the Institution of Civil Engineers (London) of which he is a fellow, and is a fellow and life member of NZSEE.

Gerald Shirtcliff was the Construction Manager for Williams Construction and Union Construction during the construction of the CTV building.

Russell Simson is one of three CCC building officers who was part of the team that carried out the Level 2 Rapid Assessment of the CTV building on 7 September 2010.

Timothy (Tim) Sinclair is a geotechnical engineer with Tonkin & Taylor Limited, where he is a principal. Tonkin & Taylor Limited produced a report on the geotechnical conditions of the CTV building site for the DBH for use in the production of the Hyland/Smith report. His qualifications include a Master of Arts in engineering science and a Masters in soil mechanics. He has over 40 years in practice and is the firm's principal expert in geotechnical earthquake engineering and seismic hazard assessment and is also a specialist in soil dynamics and machine foundations. He is a CPEng and a fellow of IPENZ.

Ashley Smith is the Principal of StructureSmith Limited and co-authored the Hyland/Smith report with Dr Hyland. He has an ME from The University of Auckland, has been a practising structural engineer since 1981, and a CPEng since 2010. He has been the project engineer on significant buildings located in Auckland (up to 41 storeys), and his areas of specialist expertise include structural engineering analysis and design and earthquake engineering.

Paul Smith is a draughtsman who has worked for ARCL since November 1987. He is now a Director of that company.

Graeme Smith of Concrete Repair and Protection Limited carried out inspections and prepared an estimate for repair of the damage caused to the CTV building following the September earthquake. He has a BE (Civil) and has worked in the concrete repair industry since 1994.

Penelope Spencer worked for CTV and witnessed the collapse of the CTV building from the middle of Cashel Street (to the south of the CTV building).

Wayne Strachan worked at ARCE as a draughtsman between 1979 and 1988.

Tony Stuart is a structural engineer with Compusoft Engineering and was involved in the NLTHA of the CTV building carried out for the Royal Commission. He was also a member of the NLTHA Panel. He has a BE (Hons) and is a member of NZSEE and SESOC.

Richard Sullivan is a structural engineer who assisted the CCC in carrying out rapid assessments of buildings in the Christchurch Central Business District after the September earthquake. He was involved in the Level 1 Rapid Assessment of the CTV building on 5 September 2010. He has a BE (Civil) and is a member of IPENZ and a CPEng.

Graeme Tapper was the Assistant Buildings Engineer at the CCC at the time the CTV building was permitted. He assessed the CTV building for compliance with the structural aspects of Bylaw 105. He was a registered engineer and a member of IPENZ. He is now deceased.

Patricia Tapper is the widow of Graeme Tapper.

Simon Thomas worked for CTV and prepared evidence about the damage to the CTV building after the September earthquake.

John Trowsdale worked as a USAR support engineer after the February earthquake. He was at the CTV building site from the evening of 22 February 2011 and was asked to give evidence about his observations of the state of the building and its elements after the collapse. He has a BE (Civil) (Hons), is registered as an international professional engineer, and is a CPEng and a member of IPENZ.

Arthur Tyndall is a structural engineer who provided evidence about a post-earthquake inspection that he carried out on the CTV building after the June 1994 “Arthur’s Pass” earthquake. He has a BE, is registered as an international professional engineer, and is a CPEng and a fellow of IPENZ.

Christopher (Chris) Urmson is a structural engineer employed by ARCL. He has an MSc and is a graduate member of IPENZ. He was involved in the collection of column samples from the CTV building remains at the Burwood landfill and observed testing of samples.

Pieter Van den Berg operates a building company, Standstill & Seymour Builders Limited/Ltd, and provided a quotation for cladding of the west wall and gave evidence about his observations of the building at that time.

Peter Van der Zee is a CCC building officer who carried out the Level 1 Rapid Assessment after the September earthquake.

Jo-Ann Vivian is the National Practice Manager for Relationships Aotearoa, based in Wellington. She visited the CTV building after the Boxing Day earthquake.

Peter Wilding is the National Manager of Fire Investigation and Arson Reduction for NZFS, and gave evidence about the processes followed for investigation of the fire after the collapse.

Alun Wilkie was the architect for the CTV building.

Grant Wilkinson was the Project Director for the assessment of the CTV building undertaken by Holmes Consulting Group in 1990, and reviewed the assessment and report of Mr Hare. He has a BE (Hons), is a fellow of IPENZ, a CPEng and a member of NZSEE. He has worked as a consulting structural engineer since 1984 and is the Managing Director of Ruamoko Solutions, a consulting structural engineering firm.

Michael Williams witnessed the collapse of the CTV building from the Inland Revenue building (to the south of the CTV building).

Appendix 2: Chronology

Early 1986	Mr Neil Blair of Prime West Corporation Limited (Prime West) engages Williams Construction Limited (Williams Construction) to submit a design-build proposal for an office building at 249 Madras Street
February 1986	Monthly time records of Alan M Reay Consulting Engineer (ARCE) show first time recorded for Dr Alan Reay for the CTV building – 2 hours
March 1986	Monthly time records show first time recorded for Mr David Harding on the CTV building design (22 hours)
3 April 1986	Mr Michael Brooks and Mr Anthony Scott of Williams Construction submit architectural drawings and structural sketches for the building to Mr Blair, with a preliminary cost estimate of $2,450,000 plus GST
May 1986	Prime West approves Williams Construction instructing Alun Wilkie Associates and ARCE to prepare drawings for permit and construction
23 May 1986	Date on gravity calculations for CTV building
10 June 1986	Date on seismic calculations for CTV building
18 June 1986	Date of Soils and Foundations Report for “Proposed Development – 249 Madras Street”
26 June 1986	Date on foundation calculations for CTV building
7 July 1986	Date of invoice for computer charges of $163.09 for June 1986 from University of Canterbury to ARCE ‘File 2503’ (CTV)
17 July 1986	Application for building permit for CTV building filed with Christchurch City Council (CCC)
26 August 1986	CCC file indicates CTV structural drawings received
27 August 1986	Letter from Mr Graeme Tapper of the CCC to ARCE regarding CTV building permit application
1 September 1986	Mr David Harding writes “rec’d 1/9/86” on the letter from Mr Tapper
5 September 1986	Document Transfer Form sent from ARCE (signed by Mr Harding) to CCC (Mr Tapper)
10 September 1986	Mr Tapper signs off on CTV building permit application
30 September 1986	CCC issues building permit for the CTV building
October 1986	Formal building contract signed by Prime West
16 October 1986	CCC building inspection record reads (first recorded CCC inspection): *Founds (sic)…Set out by Surveyor OK Steel to Finish Engineer Due*
11 December 1986	CCC inspection: *Founds (sic)…Last of Found (sic) beams*

18 February 1987	CCC inspection: *1st Floor…OK*
8 March 1987	CCC inspection: *Shear Wall… Ok Gantry Up*
31 March 1987	CCC inspection: *Shear Wall…Preparing/Proparing (sic) 2nd Frame*
1 May 1987	Union Construction Limited (Union Construction) begins trading
17 August 1987	Letter from Mr Bryan Bluck of CCC to Williams Construction regarding *“recent inspection”* and refers to the building nearing completion.
20 August 1987	CCC Inspection: *Fixing Gib wrong Card Left New foreman*
September 1987	Assignment of CTV contract from Williams Construction to Union Construction
9 October 1987	CCC Inspection: *Foreman to prove Front Cols on site and fill Block Work 1st/2nd West End*
16 October 1987	CCC Inspection: *No Contact from site so visited found peg cols to be removed. Foreman advised* *Col 200 on Street. Survey peg set at 3m from Kerb*
11 January 1988	CCC Inspection: *Cols removed. Boxing ready to pour*
22 February 1988	CCC Inspection (last recorded CCC inspection): *Finishing Handrails & Hardware*
24 January 1990	Holmes Consulting Group (HCG) engaged by Buddle Findlay and Schulz Knight Consultants Limited to prepare a structural report on the CTV building for the Canterbury Regional Council
26 January 1990	Mr John Hare of HCG views ARCL’s file on the CTV building
29 January 1990	Mr Hare visits Mr Bluck at CCC to discuss the CTV building
30 January 1990	Mr Hare visits CTV building with Mr Banks for inspection
31 January 1990	HCG provides draft report to Schulz Knight Consultants, that identifies “a vital area of non-compliance” in the connection of the floors to the north wall complex
1 February 1990	Dr Reay and Mr Geoffrey Banks meet with Mr P W Young of KPMG Peat Marwick (receivers of Prime West) Mr Banks, on behalf of ARCL, advises insurer that their own review of the drawings confirmed the apparent lack of ties to two walls
2 February 1990	Letter from Mr Banks to Mr Grant Wilkinson of HCG regarding proposed remedial work
14 February 1990	Further discussion between Mr Banks and Mr Hare

9 April 1990	Mr Banks completes Annual Report on status of claim for the Consulting Engineers Advancement Society (CEAS) and records "[w]e are still investigating whether there is a deficiency, and if so, details of remedial work"
21 December 1990	Settlement of purchase of CTV building by Madras Equities Limited (Madras Equities)
4 February 1991	Article appears in Christchurch newspaper *The Press* reporting that the CTV building has been sold
9 April 1991	Following the receipt of legal advice, Mr Peter Smith of CEAS authorises ARCL to inform the new owner of the CTV building about the floor connection issue
30 September 1991	Letter from Mr Russell Ibbotson of Madras Equities to Mr Banks recording their discussions on or about 11 September 1991 that "there may be a engineering design fault omission" in the CTV building
1 October 1991	CCC issues building permit for fit-out work for ANZ Bank tenancy of the CTV building, that includes converting part of level 1 from car parking to office space and installation of concrete block walls on level 1
15 October 1991	Letter from Mr Banks to Mr Ibbotson, recording that "a limited amount of additional reinforcing work is required in order to provide the seismic strength to meet the current New Zealand Standards"
16 October 1991	Mr Ibbotson advises Mr Banks that the quotation for the remedial work is accepted and instructs ARCL to proceed
17 October 1991	Letter from Mr Banks to Mr Ibbotson confirming that "the proposed remedial work will give the floor to wall connection the seismic strength required by the current New Zealand loadings code, NZS 4203:1984"
24 October 1991	Letter from Mr Banks to CEAS advising that the remedial work was underway
4 March 1992	Mr Banks fills out an Annual Report on status of claim recording that the remedial work had been completed, at a cost of $4,483.50 plus GST and that the building was occupied
10 May 2000	CCC issues building consent for fit-out work for Christchurch Television (CHTV) that includes cutting a penetration in the floor of level 2 so that an internal stair could be installed
16 May 2001	Design Edge Limited filed building consent application (on behalf of Madras Equities) with CCC for a fitout on level 3 (Going Places), indicating building will undergo a change of use
24 August 2010	Application lodged with CCC for consent to demolish building at 213 Cashel Street to the west of the CTV building
4 September 2010	September earthquake
5 September 2010	Level 1 Rapid Assessment of CTV building
7 September 2010	Level 2 Rapid Assessment of CTV building carried out by Mr Calvert, Mr Flewellen and Mr Simson
29 September 2010	Inspection of CTV Building by Mr Coatsworth. Mr Leonard Pagan (Quantity Surveyor with Rawlinsons Ltd) and Mr John Drew also present
6 October 2010	David Coatsworth reports to Mr Drew
12 October 2010	Building consent for demolition of 213 Cashel Street granted by CCC
12 October 2010	Mr Drew requests CCC property file for CTV building

19 October 2010	Mr Coatsworth returns to inspect the CTV Building at Mr Drew's request
22 October 2010	CCC advises Mr Drew CTV property file is available to view
21 December 2010	CTV property file returned to storage
26 December 2010	Boxing Day earthquake
27 December 2010	Level 1 Rapid Assessment and Urban Search and Rescue rapid visual survey of the CTV building
January 2011	The Clinic moves into the CTV building
5 January 2011	Jo-Ann Vivian requests CCC inspection of CTV building
7 January 2011	Jo-Ann Vivian cancels CCC inspection of CTV building after speaking to Mr Drew
31 January 2011	Mr Drew requests property file for CTV building from CCC
1 February 2011	CCC advises Mr Drew that the property file for the CTV building is available to view
2 February 2011	CCC records show the property file for the CTV building was returned to storage
22 February 2011	The CTV building collapses as a result of the February earthquake

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