Section 5: Post-collapse investigations

5.1 Post-collapse examinations of building debris

5.1.1 Introduction

Despite the state of the CTV building after it collapsed, the arrangement and condition of the debris provided important insights about how the building failed. Fortunately, a great deal of evidence was available to the Royal Commission, largely due to the initiative and efforts of three engineers who were part of the Urban Search and Rescue (USAR) effort.

Mr Graham Frost is Chief Engineer at Fletcher Construction Company with over 30 years of design and construction experience. Mr John Trowsdale is a chartered professional engineer and Project Director for Holmes Consulting Group with seven years' experience as a consulting engineer. Mr Frost and Mr Trowsdale were members of the New Zealand USAR team. Dr Robert Heywood is a forensic engineer living in Australia. He has 38 years of structural engineering experience in design, field testing, failure investigation, research and education. He is part of the Queensland USAR team that came to Christchurch to assist after the earthquake. All three provided valuable evidence to the Royal Commission.

The primary objective of USAR officers immediately after the collapse was the rescue and recovery of people trapped in the building. Mr Frost, Mr Trowsdale and Dr Heywood assisted with this effort by identifying risks to New Zealand Police and USAR team members at the site. However, they realised that evidence about the state of the building would be important in identifying why it collapsed and decided to gather information. They made sketches and notes, took photographs and marked and retained around 30 samples of structural elements. Dr Heywood did not know of any standard or best-practice for the retention of samples and explained that they did the best they could in the circumstances.

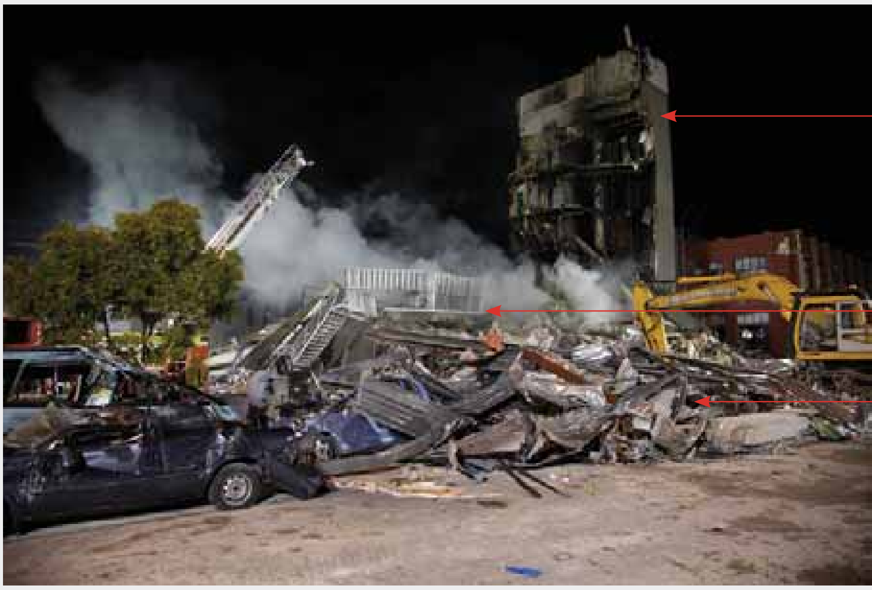
The 30 samples were left with New Zealand Police and Dr Heywood had no further involvement with them. Debris from the CTV building was taken to a secure site at the Burwood landfill. Mr Frost and Dr Heywood visited this site on 28 June 2012 after they had given their initial evidence to the Royal Commission and subsequently they provided further evidence.

5.1.2 Location of debris after the collapse

Mr Trowsdale arrived at the site at around 7.30pm on 22 February. Mr Frost arrived on the evening of 23 February, about 30 hours after the building collapsed. The original structural form was not clearly discernible to him (see Figure 58(a)), however he identified the main seismic resisting elements as the wall complex to the north which was still standing and the south shear wall at the southern end of the site.

Dr Heywood started working on the CTV building site at 4:00am on 24 February 2011. Figure 58(b) is a photograph of the building taken by him at this time. Substantial quantities of material, particularly on the eastern side, had been shifted or removed before Dr Heywood's arrival. This consisted of the spandrels and edge beams that spilled out on to the cars parked on Madras Street, as shown in Figure 58(a). Dr Heywood explained that the building had fallen mainly within its own footprint and only the north wall complex remained standing. Remnants of floor slabs leant against the north wall complex and the south shear wall had fallen inwards on top of the debris. The external steel staircase attached to the side of the south shear wall can be used to identify its location in the debris (see Figure 58(b)). The south shear wall lay horizontally with the top section sloping down relative to the rest of the wall.

(a) Immediately following collapse (source: Graham Frost)



(b) North wall complex and collapsed south shear wall with relocated material in the foreground at 4am on 24 February 2011 (source: Robert Heywood)

Figure 58: View of debris from the corner of Cashel and Madras streets looking north-west

There was little debris south of the base of the south shear wall. This indicated that the southern half of the building had collapsed vertically with little horizontal displacement. All six levels had compressed into a pile of rubble approximately 3.7 metres high, as shown in Figure 59(a), which was equivalent to the height of level 1 prior to the earthquake. Figure 59(b) identifies some edge beams that detached from the south shear wall. Dr Heywood explained that not all edge beams detached, with some edge beams from level 5 and/or 6 transported northwards with the south shear wall when it fell. He also described the steel roof material being concertinaed up against the base of the north wall complex. Mr Frost said the roof had been folded in two back over the northern half of the building and was trapped under the top section of the south shear wall (see Figure 59). This observation suggested to him that the central section of the roof fell before the south wall was pulled over on top of it.

Figure 59(a): South shear wall bent over at the height of level 1 with all floors slabs compacted beneath (source: Graham Frost)

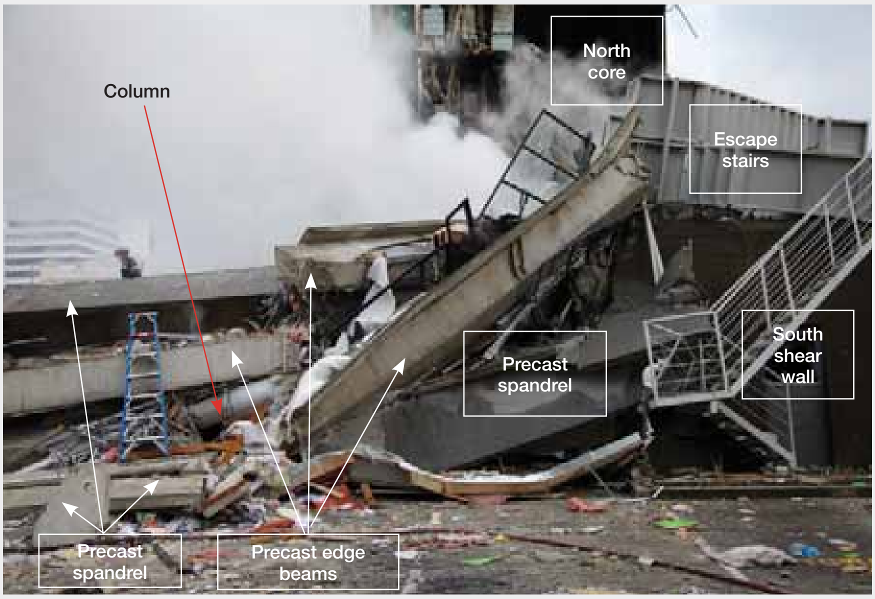


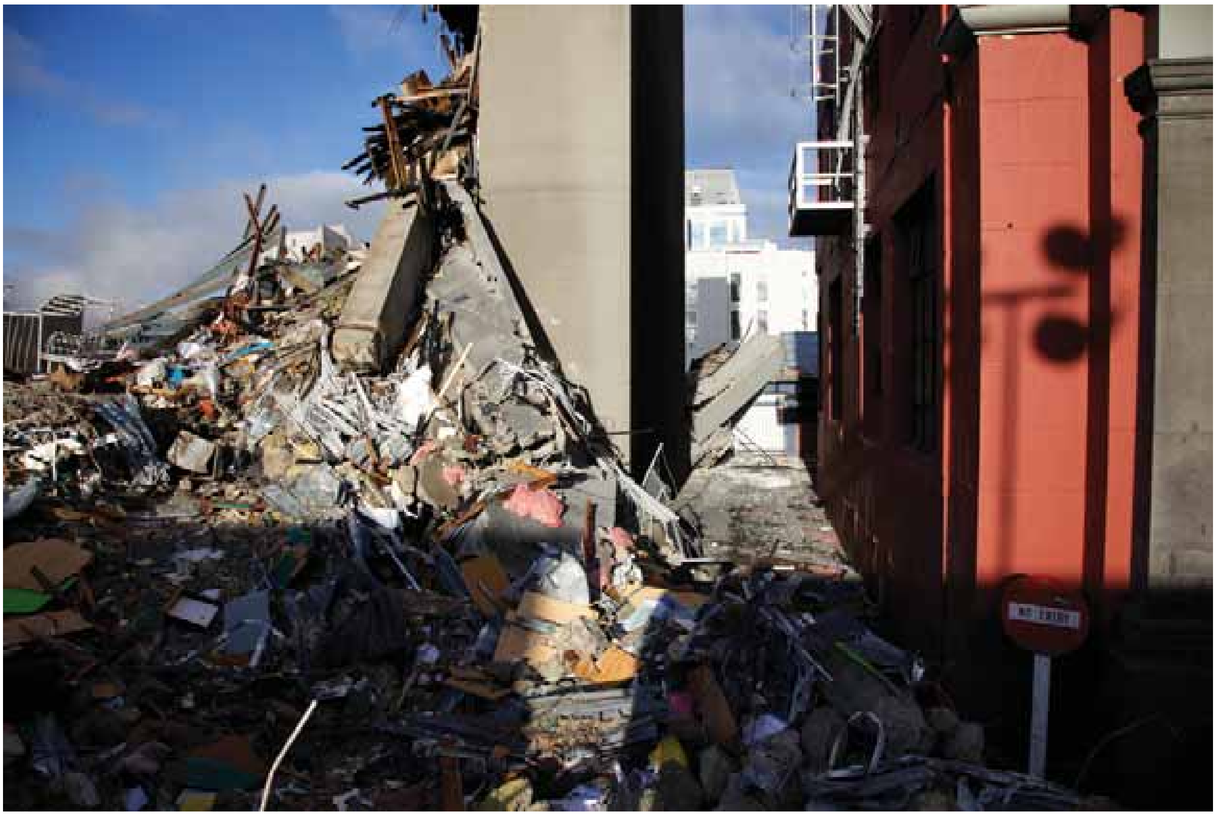
Figure 59(b): Spandrel panels and edge beams to the west of the south shear wall (source: Robert Heywood)

Figure 59(c): View of debris from a New Zealand Fire Service snorkel basket (source: John Trowsdale)

Figure 59: Southern elevations of the collapse debris

Mr Frost said that some slabs and beams in the northern half of the building had moved several metres to the north relative to their original position. The debris to the north-east and north-west of the north wall complex had spilled out northwards, as shown in Figure 60(a) and (b). He stated that the slabs, beams and spandrel panels in the northern half of the building had been thrown northwards due to a “pendulum effect”.
He considered this was caused by a combination of support being lost near the centre of the building and the building elements remaining attached to the north wall complex. As the building elements swung down they detached and were thrown northwards.

(a) North-west corner of the building looking east (source: Robert Heywood)

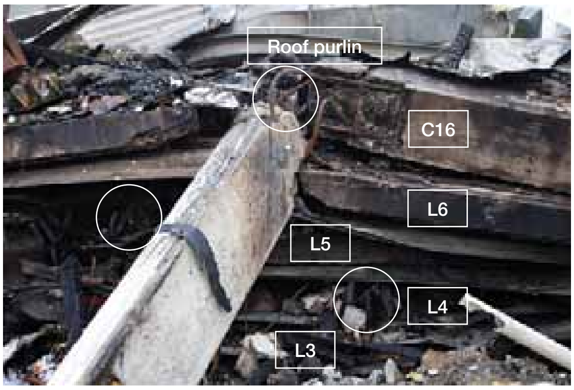


(b) North-east corner of the building looking west (source: Robert Heywood)

Figure 60: Debris located north of north wall complex

Dr Heywood described debris on the western side of the building being more defined than other sides (see Figure 61(b)). Most floor slabs lay with their ends vertically above each other, although not all slabs fell directly on top of each other. Most of the block wall had been removed before Dr Heywood arrived at the site with only a single section partially upright, as can be seen in Figure 61(a). The ends of the internal floor beams are circled in Figure 61(b). These are approximately in a vertical plane but have some relative displacement in the north-south direction. The 90º bent bars protruding from the beam ends have pulled out of the rectangular columns.

(a) Columns and block wall on the western side (source: Robert Heywood)



(b) Collapsed slab and beam ends at the end of line 3 (source: Robert Heywood)

Figure 61: Western elevation of building

5.1.3 Observations of structural elements

5.1.3.1 Columns

Mr Frost gave evidence that the only real damage that he saw to columns was at their ends. The central sections of the columns he saw were largely intact. Figure 62(b) shows where a precast spandrel panel adjoined a column. The unpainted area on the left of the column was covered by the spandrel panel. There was no damage to this column at the level adjacent to the top of the spandrel and Mr Frost saw no evidence of column hinging caused by spandrel panel interaction. Figure 62(c) shows the vertical and spiral reinforcement within a typical column.

Mr Trowsdale said that the columns were a mess and “lying all over the place”, and it was hard to determine whether the columns were intact at the middle or end. Beams had disconnected from the columns but he could not determine whether they had disconnected from the top or the bottom of the column.

Dr Heywood said that the west, south and east perimeter, including the exterior columns of the building, had been substantially altered before he arrived. The circular columns on the eastern side had been removed before he arrived, although he saw and photographed segments of the circular column C2 (see Figure 59(b)).

Dr Heywood referred to photographs from the Hyland/Smith report (see Figures 62(d) and (e)) which show the column on the south-west corner visible. A part of column C4 (about one storey in length) (see the column location diagram in Figure 62(a)) remained vertical after the collapse although concrete was missing at its base. The rectangular columns on the western elevation were detached from the building as continuous lengths but the concrete in the vicinity of the beam-column joints had been lost. The segments of columns between the floors appeared to be sound with the segments connected by the column reinforcement.

Dr Heywood said that column C19 (see Figure 62(a)) collapsed against the brick wall of the adjacent undercover car park (see Figure 62(f)). He said that this column appeared to have moved with the collapsing edge beams. The ends of the edge beams attached to C19 came to rest near the base of the north wall complex at one end and on top of the roof of an adjacent building at the other end. He also described two storey-long segments of column C18 which remained intact and joined by column reinforcement (see Figure 62(g)). Column C18 was unique since it connected to the top of the north wall complex. This meant it could continue to support some loads even if the columns below it had gone.

Dr Heywood said that a number of interior columns above level 6 remained intact, with some maintaining their connection with the level 6 floor slab and interior beams. However, below level 6 his overall impression was that interior columns were largely reduced to reinforcement and rubble with the occasional short length of column (see Figure 62(h)).

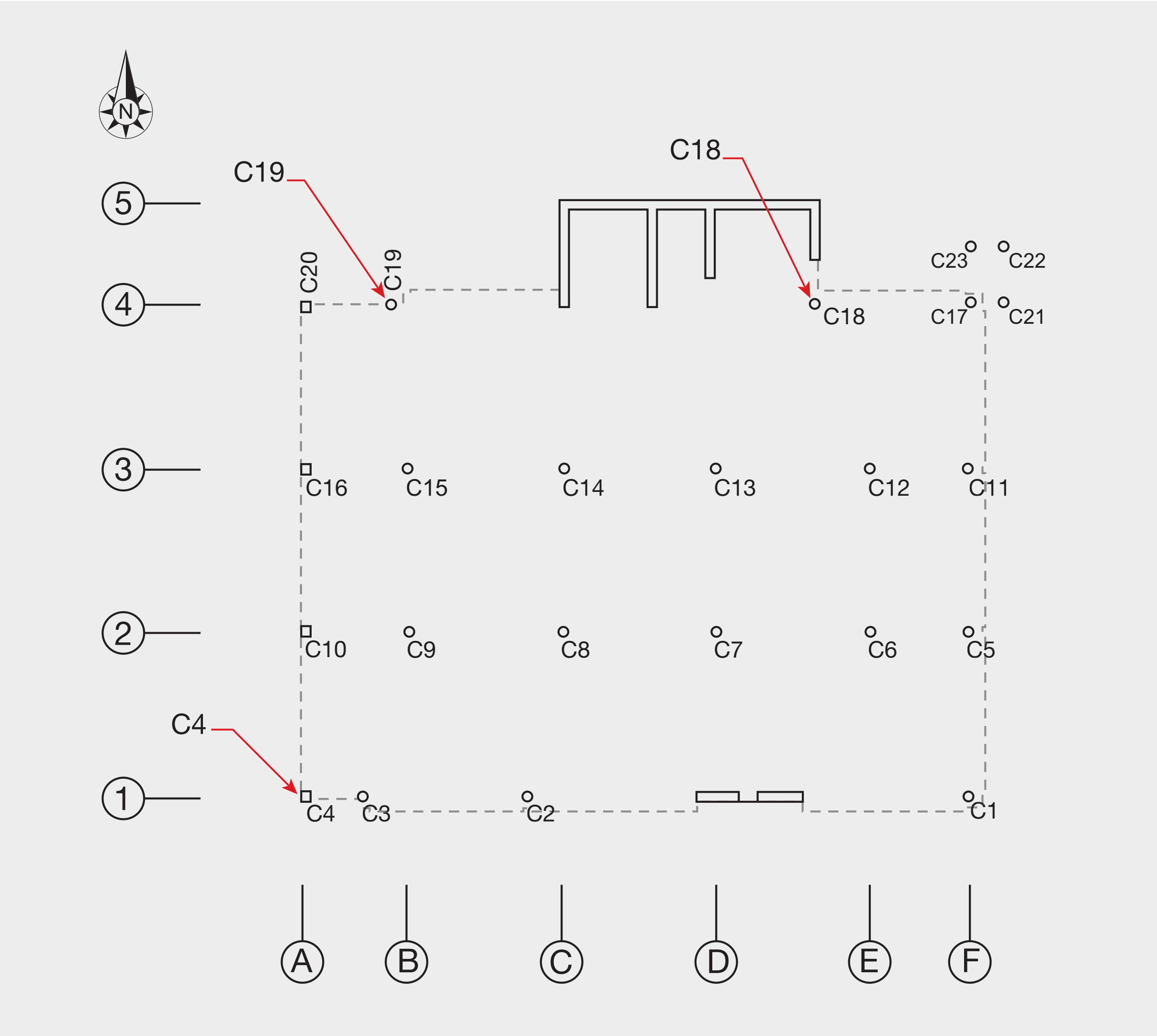


Figure 62(a): Plan of the building showing column locations

Figure 62(b): Exterior column (source: Graham Frost)

Figure 62(c): Column of unknown location showing vertical reinforcing bars and spiral transverse reinforcement. Concrete has spalled off from outside the reinforcement and fractured inside (source: Graham Frost)

Figures 62(d) and (e): The western elevation with the column on the south-west corner visible (source: Hyland/Smith[1] report)

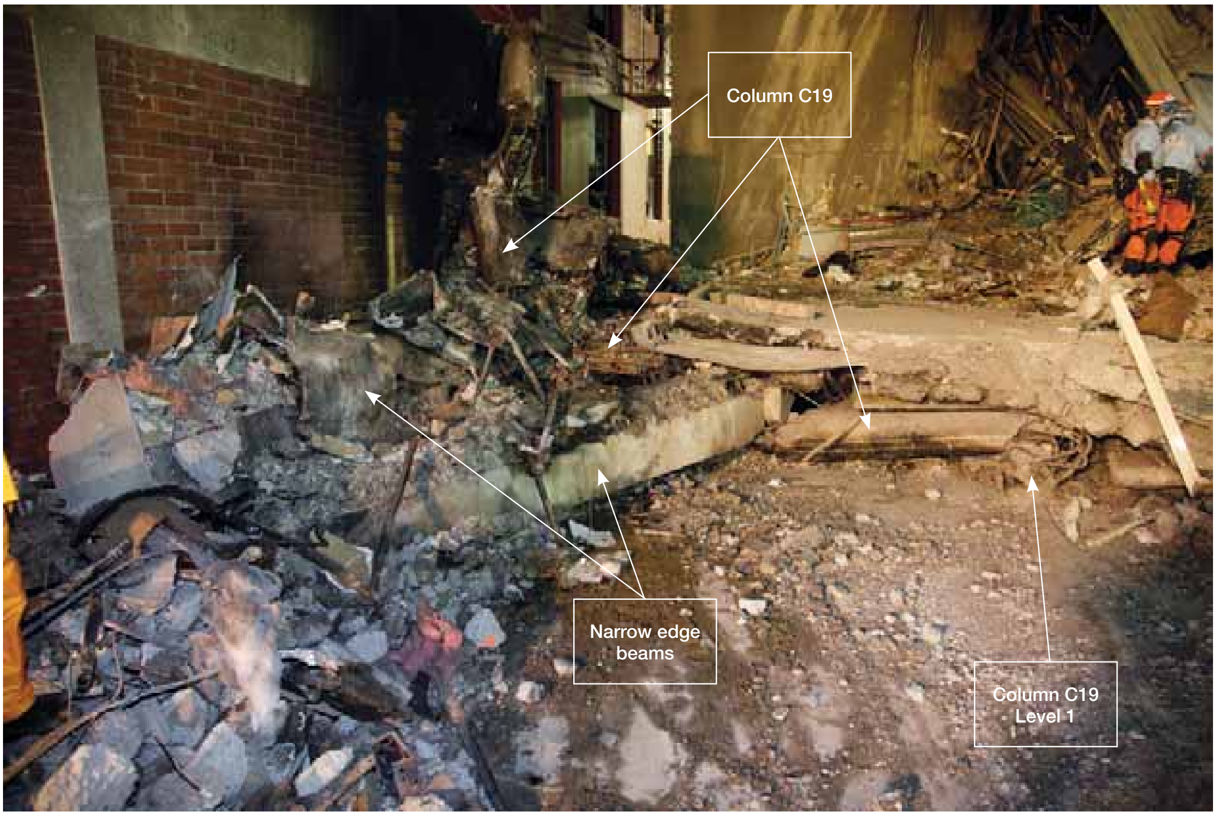


Figure 62(f): View of the north-west corner looking towards the north wall complex (source: Robert Heywood)

Figure 62(g): View towards the north wall complex from the south-east showing two storey-long segments of column C18 (source: Robert Heywood)

Figure 62(h): The junction between line 2 and column C7 looking north towards the north wall complex. The remains of column C7 are in the centre (source: Robert Heywood)

5.1.3.2 Beams

The beams were largely intact with the damage typically at the ends and on the top where the floor slab had detached. Most of the beams Mr Frost saw had the end corners broken off with little damage along the beam length, as shown in Figure 63(a).

Mr Frost said that no interior beams were found with the concrete slabs still attached, as illustrated by Figure 63(b). The internal beam to floor connection is illustrated by the structural drawing section in Figure 63(c). The steel Hi-Bond flooring is discontinuous at the beams with a 60mm seating and the slab mesh passes over the top of the beam stirrups and longitudinal reinforcement. Dr Heywood indicated in red where the failure surface was typically observed. He also observed that all connections between the edge beams and floor slabs were severed. In every case the mesh and supplementary H12 reinforcement was torn from the top surface of the beam. The structural drawings specified that slab reinforcing should be placed above the beam steel. Mr Frost's opinion was that this detailing provided poor connectivity. He would have placed some slab reinforcing underneath the top longitudinal steel to increase the connectivity between the beam and the slab.

Figure 63(a): Edge beam from line 4 between lines A and B (source: Robert Heywood)

Figure 63(b): Interior beam from the western side of the building (source: Graham Frost)

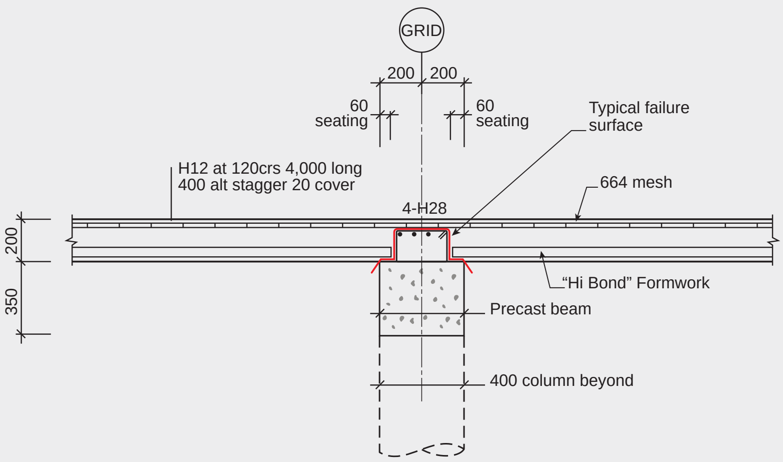


Figure 63(c): Section through precast internal beam to slab with typical observed failure surface indicated in red

Figure 63: Beam exhibits and details

5.1.3.3 Beam-column joints

Neither Mr Trowsdale nor Mr Frost saw any beam-column joints intact. Mr Frost stated that the joint regions had fallen apart as there was no steel to confine the concrete, as illustrated in Figure 64(a). Dr Heywood did not observe any interior beam-column joint where the concrete in the joint had not been lost or rendered ineffective, other than at level 6 where the columns supported the roof. The precast beam in Figure 64(b) has pulled away from the beam-column joint and is lying on its side. The photograph illustrates the smooth formed circular surface and broken off corners (or "wings") that were a common observation with these beams.

Dr Heywood expressed the opinion that it was likely that the connections between the rectangular columns and the adjoining beams on the western frame on line A (see Figure 62(a)) disintegrated during the early stages of collapse. As a result the rectangular columns detached from the building. Dr Heywood considered that if the columns had remained attached they would have suffered considerable damage and the floors would not have come to rest one above another as he observed. The end of a beam which was connected to a corner column on line A is shown in Figure 64(c). Dr Heywood described the concrete from within the beam-column joint remaining within the 90° bent bars in the beams. The beam pulled out from the column between the column reinforcement. An imprint of the vertical column reinforcement is visible in this narrow edge beam end as well as the smooth end from the precast portion. This was a poorly detailed connection, as we discuss in section 6.3.5.

Figure 64(a): Disintegrated beam-column joint (source: Graham Frost)

(b) Beam end with a smoothed circular end and bottom longitudinal reinforcement that connected to columns (source: Graham Frost)

(c) End of a beam (source: Graham Frost)

Figure 64: Beam-column joint details

When examining the remnants of the CTV building at the Burwood landfill on 28 June 2012, Dr Heywood observed a number of internal and edge beams in the rubble. All of the "wings" at the end of these beams had broken away. These wings acted as a mould for the column concrete which was cast in situ (see Figures 64(b) and 65). The semi-circular surface at the beam ends had no surface roughening and there was no evidence of any substantial bond between the beam and column concrete.

Mr Daniel Morris, who operated a concrete cutting business in the 1990s, was called as a witness by Alan Reay Consultants Limited (ARCL) and Dr Reay. Mr Morris initially gave evidence that about 200 holes had been drilled by employees of his firm in the CTV building at some point in the 1990s, about 50 of which were in beams. However, in cross-examination he conceded that his evidence was a guess which could be "wildly out". We add that, while at the Burwood landfill, Dr Heywood looked for evidence of holes cored through the beams.

He did not find any internal or edge beams with holes cored through them. In view of his answers in cross-examination we did not consider the evidence of Mr Morris to be of any probative value.

Mr Frost, who is experienced in concrete construction, gave evidence about the weakness of the internal beam-column joints. He explained that when using precast concrete, designers should detail the construction joint to be perpendicular to the beam axis so that the compression forces can be transferred directly across the joint to limit the risk of slip along the interface. He pointed out that sloping construction joints can result in a greater spacing between stirrups, which reduces the confinement of the concrete.

Figure 65 is a diagram of the interior beam-column joint prepared by Mr Frost. This diagram illustrates the forces that could lead to the beam wings breaking off. The red arrows indicate the direction of the compression forces from gravity loading at the bottom of the beam. The very smooth interface between the beam and column means that forces cannot be effectively transferred straight across the joint. This, together with the curved surface, generates radial forces from the column which tend to split the beam wing sections off. Since there is no reinforcing acting to confine these wings, a crack may develop as illustrated in Figure 65. Mr Frost thought it was a very strong possibility that a pulse from vertical acceleration created a force sufficient to break the wings off. He stated that, after these wings were lost, the joint would have little capacity left. The gravity and seismic shear loads on the beams would then have to be transferred through the slab, dowel action and the much reduced bearing area at the bottom of the beam. Mr Frost considered that this bearing area was a vulnerable region at the top of the column. The beam sat on the cover concrete of the column which would spall and fall away when the column was exposed to high drifts.

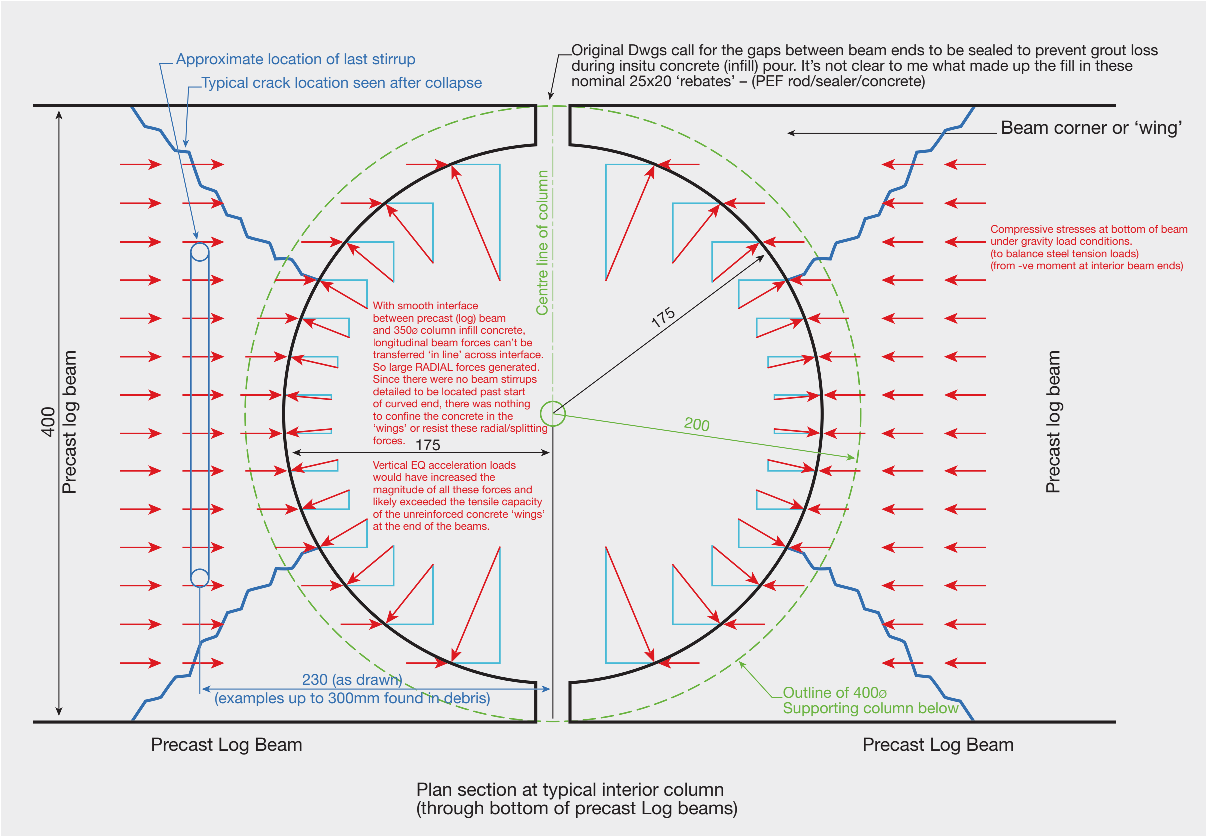


Figure 65: Plan view of beam-column joint prepared by Graham Frost

These observations led Mr Frost to conclude that a possible building failure mechanism was the failure of one or more beam-column joints due to a lack of confining steel in the beam ends and beam-column joints, exacerbated by the transfer of beam compression stresses across smooth formed surfaces that were not perpendicular to the line of action of those forces.

The beam-column joints are also discussed in section 6.3.5 and sections 7 and 8 of this Volume.