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

BRIEF OF EVIDENCE OF ROBERT JAMES HEYWOOD
IN RELATION TO THE CTV BUILDING

DATE OF HEARING: COMMENCING 25 JUNE 2012

BRIEF OF EVIDENCE OF ROBERT JAMES HEYWOOD
IN RELATION TO THE CTV BUILDING

1. My full name is Robert James Heywood. I am a Forensic Structural Engineer living in Brisbane, Australia.

QUALIFICATIONS AND EXPERIENCE

2. I have 38 years' experience as a structural engineer. I graduated from the University of Queensland, Brisbane with a Bachelor of Engineering (Civil) Honours in 1974, completing a Masters of Engineering Science degree in 1982 and a PhD in 1993. I am the Principal of Heywood Engineering Solutions P/L and an Urban Search and Rescue Engineer.

3. I have extensive experience in the design, field testing, failure investigation, research and education of most facets of structural engineering, including the design of infrastructure to resist earthquakes in the Solomon Islands and Papua New Guinea and the design of medium rise buildings. I was a Principal Researcher and Senior Lecturer at the School of Civil Engineering at the Queensland University of Technology from 1985 to 1998. I have published in refereed journals and conferences in the areas of design of concrete structures, the static and dynamic loads applied to bridges, the field performance of structures (including tests to destruction), fatigue of structures, engineering education and forensic engineering.

4. I am the Deputy Chairman of the Structural College Board of Engineers Australia and a committee member of the Queensland Division Structures Panel, Engineers Australia. I am a Registered Professional Engineer in Queensland, a Chartered Professional Engineer and registered with the National Professional Engineers Register in Australia.

5. I hold professional memberships with:
   a. Institution of Engineers Australia, of which I am also a fellow;
   b. American Society of Civil Engineers;
   c. International Association of Bridge and Structural Engineers.

6. In 1988 I was awarded a Centenary Scholarship to study at the University of Michigan, and was the recipient of the 1995 Warren Medal for the best paper in civil engineering.

Both awards were made by the Civil College Board of the Institution of Engineers Australia.

7. Part of my role as a Forensic Engineer is to identify the cause/s of failure and to disseminate the lessons of failure.

8. I am professionally aware of the dynamics of structures, the damage induced by earthquakes and the response of structures when tested to destruction in laboratories and in the field. However, the 22 February 2011 earthquake in Christchurch was my first exposure to the immediate aftermath of a destructive earthquake.

EVIDENCE

9. My evidence will address the following topics:
   a. My role at the CTV Building (Building) site in the days following the February 2011 earthquake;
   b. My initial observations of the CTV Building in its collapsed state;
   c. Some observations of the state of certain parts of the Building which I considered to be of particular interest.

10. In referring to, and commenting on, the state of particular parts of the Building following the collapse, I have drawn upon my expertise as a forensic engineer.

11. I have read and agree to comply with the Code of Conduct for Expert Witnesses. I confirm that all of the matters set out in this brief are within my areas of expertise.

12. I am aware that Dr Clark Hyland and Ashley Smith carried out an investigation and prepared a report addressing the possible failure mechanisms of the Building. I have not carried out any such investigation myself and am not in a position to express an opinion about the likely failure mechanism.

13. However, I believe that my observations may assist the Royal Commission in assessing the performance of certain parts of the Building and in identifying and verifying possible failure mechanisms.

ROLE AT THE CTV BUILDING SITE

14. I flew to Christchurch on 23 February 2011 and started work at the Building site at 4.00am on 24 February 2011 with the Queensland Taskforce 1 USAR team. My role, as one of two structural engineers on the USAR team, was to provide engineering support to the search and rescue operations of Queensland Taskforce 1, which is certified by the United Nations for global heavy lift search and rescue operations.

15. I was involved in the urban search and rescue operation for 11 days in total, spending the first 3 shifts at the site of the CTV Building. However, because of my interest in learning about the cause of the collapse of the Building and to gather information that may help facilitate the dissemination of the lessons learnt, I returned to the site as often as my responsibilities permitted.

16. During my time at the site, I made a number of observations about the condition of the debris from the Building collapse which I will discuss in detail shortly.

17. No architectural or structural engineering drawings of the Building were made available to me at the Building site. A hand sketch of the general layout of the Building prepared by NZ USAR representatives, and a pre-earthquake picture of the Building retrieved from the internet provided background information.

18. I took approximately 500 photographs throughout my time at the Building site. These photos form my diary of events. The initial purpose of taking photographs at the site was to document the search and rescue operation and as a resource for future training of engineers and USAR taskforce members. As time passed, the focus of the photographs shifted towards documenting the collapsed Building. The first photos I took of the Building were before sunrise on 24 February 2011. The last photographs I took of the Building were on 4 March 2011.

Floor Plans – Explanation of Reference Terms

19. A schematic typical floor plan of the Building is presented in Figure 1 [**WIT.HEYWOOD.0001.30**]. The layout has been derived from Drawing S 15 from Hyland and Smith "CTV Building Collapse Investigation Appendix L Structural Drawings – CTV Building, 249 Madras Street, Christchurch" (Appendix L to the Hyland Report).

Drawings from Appendix L are referred to herein as *Drawing S* and the relevant drawing number.

20. The schematic typical floor plan presented in Figure 1 defines the following terms:
    a. North – as indicated;
    b. South Wall – the shear wall on Line 1;
    c. North Core - the concrete walls that surround the lift well, the stair well and the amenities at the northern end of the Building between Lines 4 and 5;
    d. Internal beams – the beams on Lines 2, 3 and 4 in front of the North Core;
    e. Edge beams – the edge beams on the southern, eastern and northern perimeter of the Building to the east of Line B. These beams are 960 mm wide and support precast spandrel beams;
    f. Narrow edge beams – the two narrower edge beams on the southern and northern perimeter of the Building to the west of Line B. These beams are 400 mm wide and do not support precast spandrel beams.

21. The structural drawings adopt a numbering scheme for the beams and columns. These beam and column numbers are referred to throughout this Brief of Evidence as per the location plans presented in Figure 2 [**WIT.HEYWOOD.0001.31**] and Figure 3 [**WIT.HEYWOOD.0001.32**]. In the case of beams, a "B" is placed in front of the position number and when the floor number is known it is placed in brackets after the position number. For example B14(4) refers to the beam in position 14 on Level 4, which can also be described as the edge beam on Line F between Lines 1 and 2 on Level 4.

INITIAL OBSERVATIONS AT THE CTV BUILDING SITE

22. When I arrived at the Building site, only the lift core (North Core) of the Building remained standing (refer Figure 4 at **WIT.HEYWOOD.0001.33**). Other than the remnants of some floors leaning against the North Core, the collapsed Building had largely collapsed vertically and the South Wall had fallen on top of the collapsed Building. Some small areas of the floor slabs remained attached to the North Core. The remaining collapsed portions of the Building appeared to be higher towards the centre of the Building (refer Figure 4 and Figure 5 [**WIT.HEYWOOD.0001.34**]).

23. Smoke was rising from the collapsed Building and an excavator was working to remove material from the Madras Street side of the site. The Building site was lit by temporary floodlights. Lights were also still burning in some adjacent buildings.

24. Comparison of Figure 4 and with Figures 1, 6, 31, 32, 33, 71 and 72 in the Hyland and Smith "CTV Building Collapse Investigation" (25 Jan 2012) (the Hyland Report) taken before the removal of any material confirms that substantial quantities of materials had been shifted and/or removed before my arrival. For example:

    a. The roof sheeting, concrete beams and other debris had been piled in Madras Street (refer Figure 4 [**WIT.HEYWOOD.0001.33**] and Figure 5 [**WIT.HEYWOOD.0001.34**]);

    b. The spandrels, edge beams and columns visible in the Hyland Report Figure 1 (page 2) had been removed and the cars rearranged; and

    c. Most of the block wall on the western side of the Building evident in the Drawing S 9 and Figure 71 and Figure 72 of the Hyland Report (page 156) had already been removed (refer Figure 6 [**WIT.HEYWOOD.0001.35**]).

25. During the preparation of this Brief of Evidence I have ensured that the effects of the search and rescue operations and the removal of material have been taken into account.

26. As time passed, it became necessary to remove the stockpiles of sifted rubble to facilitate search and rescue operations. The removal process involved demolition equipment breaking the structural elements into smaller pieces and loading the rubble onto trucks for transport to a location unknown to me. I became concerned that potentially critical evidence was being lost and that this loss might hinder future investigation into the cause of the Building collapse. Consequently, with the support of the police, exhibits were put aside at the Building site. These exhibits were selected by Graham Frost (a NZ USAR engineer) and me. Where possible, arrangements were made with the demolition contractors to place the exhibits in an area on the site near Cashel Street. The objective was to collect a sample of the various elements evident in the collapsed Building with minimal disruption to the search and rescue operations.

27. Initially the samples were small material samples but the size of the exhibits grew to include complete beams and the South Wall. Graham Frost and I allocated a number to each exhibit ("E" plus a unique number) and labelled them with yellow spray paint. Graham Frost prepared a list of the exhibits and I photographed them. This was our method of preserving evidence as best we could without hindering the search and rescue process.

OBSERVATIONS OF PARTICULAR INTEREST

28. During my time at the Building site, I saw a number of things which I considered to be of particular interest in assessing the performance of the Building during the earthquake. I set these out below:

The way the Building fell

29. On arrival, I observed that the Building had fallen largely within its footprint. The floors had become detached from the North Core and the South Wall had collapsed onto the Building.

The way the eastern side (Madras St or Line F) of the Building fell

30. The Madras Street side of the Building (Line F) was indistinct because the search and rescue operations had moved most of the elements that defined the edge of the Building.

The way the southern side (Cashel St or Line 1) of the Building fell

31. The southern side of the Building (Line 1) had collapsed within the South Wall with some elements rolling away from the Building. On the eastern side of the South Wall, most of the edge beams and spandrel panels had been removed prior to my arrival (refer Figure 8 [**WIT.HEYWOOD.0001.37**] Figure 9 [**WIT.HEYWOOD.0001.38**]). On the western side of the South Wall, some of the edge beams, spandrels and columns remained in the vicinity of Line 1 (refer Figure 7 [**WIT.HEYWOOD.0001.36**]). The edge beams evident in Figure 7 appear to have become detached from the South Wall.

32. Not all edge beams, however, detached from the South Wall. It seems that some edge beams from Level 5/6 remained attached to the wall and were transported northwards with the South Wall when it collapsed. The aerial view presented as Figure 81 in the Hyland Report (page 161) indicates that some of the edge beams remained attached to the western side of the South Wall.

33. The photograph in Figure 8 [**WIT.HEYWOOD.0001.37**] shows the eastern side of the South Wall taken from the southeast corner of the Building looking towards the North Core. An edge beam can be seen lying almost parallel to the eastern side of the

collapsed South Wall. Its southern end is adjacent to the Level 3 landing and the northern end of the edge beam appears to be adjacent to the Level 6 landing. This is consistent with the 7.8 m length of the edge beam that spanned between the South Wall and the column on the southeast corner of the Building. The position of this edge beam indicates it was transported with the collapse of the South Wall.

34. The photograph in Figure 9 [**WIT.HEYWOOD.0001.38**] is taken from near the entrance of the Building looking across the collapsed South Wall towards the North Core. The South Wall had fallen across the top of the Building's floors. The South Wall had "hinged" through an angle of about 75 degrees about a horizontal line parallel to Line 1 at about Level 2, as indicated by the position of the stair landing. The section of the South Wall between Levels 1 and 2 had rotated through an angle of about 15 degrees so that the South Wall was lying nominally horizontally across the collapsed Building. The floors of the Building had compressed into a pile of rubble of the height of a single storey of the Building (approximately 3.7 metres).

35. Figure 10 [**WIT.HEYWOOD.0001.39**] and Figure 11 [**WIT.HEYWOOD.0001.40**] is the view inside the Building through the broken window in the South Wall that can be seen in Figure 9 [**WIT.HEYWOOD.0001.38**]. The severed edges of two floor slabs are visible through the broken window. Their position suggests that the floor slabs are from Level 2 and Level 3. This is consistent with the observations that follow. The carpet varied from floor to floor and this may assist in identifying the levels of the floor slabs. The "664 mesh" used as reinforcement in the top of the slab consists of 6 millimetres (1/4 inch) diameter wires welded together at 150 millimetres (6 inch) centres to form a square grid. The Hi Bond metal decking used as formwork and reinforcement for the concrete floor slabs is visible on the outside of the floor slabs.

36. Figure 12 [**WIT.HEYWOOD.0001.41**] is a closer view of the edges of the floor slabs visible to the right of the South Wall in Figure 9 [**WIT.HEYWOOD.0001.38**], but after the upper portion of the South Wall had been removed and rubble removed from the face. The floor levels of the slabs have been labelled. The location of the Level 2 slab can be identified by the end of a steel trimmer beam installed on the underside of the Level 2 floor slab to support an opening cut in the Level 2 slab for an internal staircase.

37. Another view (Figure 13 [**WIT.HEYWOOD.0001.42**]) taken at the same time as Figure 9 appears to show another edge of a floor slab above the Level 5 floor slab, indicating the Level 6 floor slab possibly came to rest near the base of the South Wall. Thus five and

possibly six of the connections between each of the suspended floor slabs and the South Wall were severed and the edges of the floor slabs came to rest relatively close to the base of the South Wall.

38. The observation that the floor slabs had become detached from the South Wall and five, possibly six, of them came to rest with their points of detachment near Line 1, suggests that the floors most likely became detached from the South Wall before it collapsed. If the floor slabs had remained attached to the South Wall during/after its collapse, these elements would have been transported north with the collapsing South Wall rather than remaining at Line 1.

The way the western side (Line A) of the Building fell

39. On the western side (Line A), the edge of the collapse was more defined with some of the floor slabs lying with their edges vertically above each other at some locations. However, all slabs did not fall directly on top of each other.

40. Figure 6 [**WIT.HEYWOOD.0001.35**] and Figure 14 [**WIT.HEYWOOD.0001.43**] are views along Line A (western face) towards the north from Line 1. Most of the block wall has been removed with only a single segment "standing" and another segment on the ground.

41. Segments of the Line A rectangular columns (C4, C10, C16 and C20 on Drawing S 14) are indicated by the arrows in Figure 14. These rectangular columns were the only multi-storey non-circular columns shown on the drawings. The Line A columns were separate from the Building with the concrete in the columns absent at the point where the columns connected to the suspended floors. The columns had become a series of storey-length segments of reinforced concrete connected by the exposed steel column reinforcement at the points where the columns connected to the floor slabs, internal beams and sill beams. In general, the concrete was missing from the columns at the junctions with the suspended floors and the associated floor and sill beams. The remaining concrete column segments were in sound condition without any reinforcement protruding from the faces of the column segments.

42. Figure 15 [**WIT.HEYWOOD.0001.44**] shows a portion of the western edge (Line A) of the collapsed Building with the floor slabs sitting one above the other. The edges of the floor slabs are unbroken and quite distinct. The edges of the floor slab have been

labelled with the associated level number. The roofing material is still in place immediately above the Level 6 floor slab (L6). The lower floor slabs are less defined although the layers of Hi Bond metal decking delineate the floors. A segment of rectangular column lies on top of the L6 slab and is connected by its exposed reinforcement to a further segment of rectangular column in the foreground of the photograph.

43. The ends of the internal floor beams that were once connected to the rectangular columns are circled in Figure 15. The concrete at the ends of the internal floor beams was missing, revealing the bent ends of the beam reinforcement. These bars were specified on the drawings to be bent up and down by 90 degrees (refer Details 1 and 2 of Drawing S19, and B05 and B10 on Drawing S20). The 90 degree bends specified on the drawings are still obvious, but there has been some additional deformation, likely attributable to the collapse.

44. The ends of the floor beams circled in Figure 15 remain approximately in a vertical plane but there has been some relative displacement between the floors along Line A (i.e., in the north south direction).

45. Figure 16 [**WIT.HEYWOOD.0001.45**] is the bottom right hand corner of Figure 6 [**WIT.HEYWOOD.0001.35**] and shows the edges of the two floor slabs. Reinforcement starter bars project from the edge of the lower floor slab (L3) but there are no reinforcement bars projecting from the L4 floor slab above. The drawings show these bars were used to tie the precast sill beams (B19, B20 and B21 of Drawing S18) into the concrete floors on Levels 2 and 3 only. These precast sill beams supported the block walls on Levels 2 and 3. Thus the floor slab edges evident in the foreground are likely to be the Level 3 (L3) and Level 4 (L4) floor slabs as indicated. The sill beams were not positively identified on site or in the photographs.

46. The Level 3 and 4 floor slabs shown in Figure 16 [**WIT.HEYWOOD.0001.45**] appear in the bottom right hand corner of Figure 17 [**WIT.HEYWOOD.0001.46**], as labelled. The edges of the slabs corresponding to Line A are visible in the foreground, with the North Core and some steel framing in the background. The intact L4 floor slab at the right of the photograph transitions to twisted reinforcement and concrete rubble in the centre and left of the photograph. The "H12 @ 120" and the "664 mesh" reinforcement exposed in the rubble corresponds to the reinforcement shown in Drawing S 16 immediately above the internal floor beams in the top of the slab on Lines 2 and 3. The

proximity to the southern elevation of the Building suggests this area of slab disintegration corresponds to the Line 2 internal floor beam on Level 4 near the junction with the Line A rectangular column. Close inspection of the reinforcement in the rubble zone shows that the "H12 @ 120" reinforcement was laid on top of the "664 mesh" and that the edge of the L4 slab has two bars in the bottom of the slab – a detail consistent with this being the L4 floor slab, as Drawing S16 Section 5 shows that these bottom bars occur only on Level 4.

47. It is not possible to observe where the edges to the Level 5 and Level 6 floor slabs fell in relation to Line A as their edges are broken, possibly from search and rescue operations (Figure 17 [**WIT.HEYWOOD.0001.46**]). The position end of the Level 5 floor beam circled in Figure 17 suggests that Level 5 came to rest some distance (1.5 metres say) to the east of the Level 4 slab edge.

48. The North Core of the Building appears to have been constructed close to the northern boundary of the property on a similar line to the southern boundary of the covered car park on a property adjacent to the north-west corner of the Building. The red brick southern boundary of this undercover car park is on the left of Figure 18 and the North Core is in the background. The three segments of a rectangular column (most likely the north-west corner column C20) in the foreground of the photograph are detached from the floors of the Building.

49. Roof sheeting and wall cladding have come to rest against the southern wall of the undercover car park (refer Figure 18 [**WIT.HEYWOOD.0001.47**]). The structural drawings indicate the northern walls of the Building, on both sides of the North Core, are offset approximately 4.0 to 4.5 metres to the south from the northern face of the North Core, which is in approximate alignment with the southern wall of the covered car park. This suggests that portions of the northwest corner of the Building have fallen to the north.

50. The roof sheeting has come to rest with approximately 3 metres of its length resting vertically against the southern wall of the car park (refer Figure 18). This indicates this portion of the roof moved northwards during the collapse by a distance of approximately 3 + 4.5 = 7.5 metres.

51. The edges of two floor slabs are visible immediately below the roofing material and rubble and between the segments of column in Figure 18. These correspond to the

Level 6 and Level 5 floor slabs. Search and rescue operations subsequently revealed these floor slabs. It is estimated that the Level 5 floor slab moved north by an estimated 3.5 to 4 metres, slightly further than the estimated 3 to 3.5 metre movement north experienced by the Level 6 floor slab (refer Figure 19 [**WIT.HEYWOOD.0001.48**]).

52. A northern (Line 4) narrow edge beam had become detached from the floor slab and lodged at the base of the red brick wall of the adjacent car park. The end of this beam, circled in Figure 19, moved approximately 4.5 metres north during the collapse.

53. The end of the narrow edge beam circled in Figure 19 is shown in more detail in Figure 20 [**WIT.HEYWOOD.0001.49**]. In contrast to the ends of the internal beams, concrete remains within the bent up bottom bars and the bent down top bars of the edge beam, indicating that the beam has been "pulled out" of the column between the vertical column reinforcement. The imprint of the vertical column reinforcement visible in the end of the beam is highlighted by the arrows in Figure 20. The smooth off-from end of the precast portion of this narrow edge beam can be seen to the right of the reinforcement imprint.

The way the northern side (Line 4) of the Building fell

54. As stated above, the upper levels of the northwest corner of the Building moved northwards a number of metres.

55. The connections between the upper edge beams and the western face of the North Core were severed during the collapse (refer Figure 21 [**WIT.HEYWOOD.0001.50**]). The precast spandrels and their supporting edge beams collapsed together. Two of the five precast spandrels and their edge beams (hidden) are seen leaning against the North Core in Figure 21. The connection between the edge beams and the floor slabs was severed during the collapse.

56. The length of the edge and spandrel beams is such that the two edge and spandrel beams leaning against the North Core in Figure 21 are most likely from Levels 4, 5 or 6. Another edge beam is circled in Figure 21. The western end of this edge beam came to rest on the roof of the adjacent undercover car park. The western ends of the edge/spandrel beams had been supported on the circular column C19 (B-4). A segment of column C19, blackened from the fire, is visible against the car park wall at

the left of the circle drawn in Figure 21. The edge and spandrel beam and C19 are shown from a different angle in Figure 22 [**WIT.HEYWOOD.0001.51**].

57. Figure 22 shows the edge and spandrel beam B23 and column C19, when viewed from near the lift core looking northwest. The narrow edge beam (B22) to the west of C19 lodged on the car park roof an estimated 5 to 6 metres north of its "as built" position. The concrete in the junction between C19, B22 and B23 has disappeared – only the reinforcement remains. The connections between the edge beams and the slabs they supported have been severed – the concrete has broken, the reinforcement has failed in tension and the Hi Bond metal decking is no longer attached.

58. The blackened metal sheeting against the brick wall of the car park appears to be light weight wall cladding. It is also visible in Figure 18 [**WIT.HEYWOOD.0001.47**] immediately against the brick wall and below the roof sheeting – before it was blackened by the effects of fire. The cladding appears to be supported by timber framing, with wall lining also evident.

59. The northern wall to the east of the North Core has formed into a pile of rubble radiating out from the southern edge of the North Core (refer Figure 23 [**WIT.HEYWOOD.0001.52**] and Figure 24 [**WIT.HEYWOOD.0001.53**]).

60. In Figure 24, a typical Line 4 edge beam is lying on top of the rubble with a circular column lying beside it. These edge beams spanned between two circular columns on Line 4 – one on the northeast corner of the Building (C17) and the other immediately in front of the eastern edge of the North Core (C18).

61. The B25 internal beam and the B11 edge beam are considered to be from Level 5 and the two column segments are considered to be from column C18 because:

    a. The internal beam is attached to Level 5 by its bottom reinforcement (refer Figure 25 [**WIT.HEYWOOD.0001.54]**) and therefore corresponds to B25(5);

    b. The eastern end of B25(5) and the top of the column require support to remain at such a precarious position;

    c. This support can be provided by the bottom reinforcement and edge beam B11 seated on the rubble as long as the internal beam, the edge beam and the column remain connected. This is consistent with the positioning of these elements as shown in Figure 24.

62. The dashed arrow-lines in Figure 24 indicate the approximate movement of the elements from their "as built" positions to their "collapsed" positions.

63. A similar pattern was revealed one level below (Level 4) following the partial removal of the rubble (refer Figure 26 [**WIT.HEYWOOD.0001.55**]). The western end of the internal beam B25(4), which remained connected to the stairwell wall of the North Core, sloped down to the east to the junction with column C18 and the edge beam B11(4). The edge beam B11(4) became exhibit E10.

64. Both edge beam B11(5) and B11(4) had been detached from the floor slab. The slab reinforcement had torn out of the concrete on top of the beam B11(4), as indicated by the spall and the reinforcement imprints in Figure 26. The discolouration of the concrete spall suggests that the floor slab had separated from B11(4) before the fire and before the partial removal of the rubble under which it was buried.

North Core Slab Detachment

65. Figure 1 of the Hyland Report (page 2) showed the upper floors had detached from the North Core prior to the removal of rubble and prior to the fire. Some changes had occurred following the collapse and before my arrival on site, including the fire damage to the North Core and the removal of the piece of floor slab hanging precariously from Level 5.

66. The severed connection between the floors and the North Core is also evident in many of the figures in this Brief of Evidence, including Figure 24 [**WIT.HEYWOOD.0001.53**]. The detachment was complete on Levels 4, 5 and 6. Although some internal and edge beams were still attached to the North Core on Levels 4 and 5, the floor slabs were detached from these beams. The detachment of floor slabs on Levels 2 and 3 is discussed in paragraph 71.

67. The following observations are made with respect to Levels 4, 5 and 6:

    a. In the vicinity of the amenities (western portion of the North Core), the floor slab was severed a metre or two south of Line 4 leaving the floor slab cantilevering from the North Core (refer Figure 25 [**WIT.HEYWOOD.0001.54**]). The 664 mesh was observed to have failed in tension in the Level 6 failure surface (refer Figure 27 [**WIT.HEYWOOD.0001.56**]). The downward angle of the exposed mesh and the spalling of the underside of the slab on the left are consistent with some vertical relative movement during failure. The top cover to the 664 mesh appears to be greater than the 30 mm specified on Drawing S15;

b. On Level 6, the failure line exhibited in the vicinity of the amenities extended across in front of the stair and lift wells before turning north at the eastern edge of the North Core. This section of the slab was subsequently removed because of safety concerns;

c. On Level 5, the failure line exhibited in the vicinity of the amenities extended across in front of the stair well before turning north into the lift well;

d. On Level 4, the slab failure appeared similar to that on Level 5 but it was not completely visible;

e. The Hi Bond metal decking reinforcement had either detached from the underside of the floor slab or torn near the position where the Level 4 floor slab was severed.

68. The concrete floors slabs came to rest leaning against North Core – particularly the western portion of the core (refer Figure 29 [**WIT.HEYWOOD.0001.58**] and Figure 30 [**WIT.HEYWOOD.0001.59**]). The uppermost floor is most likely Level 6 because:

a. Level 6 would be expected to be uppermost as it was the top floor;

b. The colour of the floor treatment on the upper portions of the floor slab in Figure 29 matches both the colour of the adjoining Level 6 floor slab shown being cut away from the North Core in Figure 28 [**WIT.HEYWOOD.0001.57**] and the floor colour scheme;

c. The top of the column C14 in Figure 29 [**WIT.HEYWOOD.0001.58**] is flat and contains holding down bolts indicating that this column supported the roof from Level 6. This column remained fixed to the slab.

d. Close inspection of the collapsed floors in Figure 30 [**WIT.HEYWOOD.0001.59**] and Figure 31 [**WIT.HEYWOOD.0001.60**] reveals the likely edges of the 5 suspended floors.

69. There have been metres of slip between the concrete floor slab and the Hi Bond Metal decking, as indicated by the Hi Bond metal decking projecting above the Level 6 floor slab in Figure 29 [**WIT.HEYWOOD.0001.58**].

70. The Level 6 floor slab has come to rest at an angle of approximately 60 degrees below the horizontal with a "hinge" forming in the slab just to the south of Line 3 (refer Figure 29 [**WIT.HEYWOOD.0001.58**], Figure 30 [**WIT.HEYWOOD.0001.59**] and Figure 31 [**WIT.HEYWOOD.0001.60**]).

71. The likely edges of the 5 suspended floors are labelled in Figure 31. It appears likely that the Level 2 floor slab has remained attached to the North Core and hinged through an angle of about 45 degrees. It appears that the Level 3 floor slab has been severed from the North Core at a position similar to where the Levels 4, 5 and 6 floor slabs were also severed from the North Core.

72. A segment of column C14 is sandwiched between the L5 and L6 floor slabs (refer Figure 31).

73. Line 3 internal beams on Level 6, which were located 7.5 metres to the south of the southern face of the North Core, came to rest approximately 4 metres from the North Core (refer Figure 30 and Figure 31). Thus the Level 6 Line 3 internal beam moved north by approximately 7.5 minus 4 = 3.5 m during the collapse. The Level 5 internal beam has moved slightly further north than its Level 6 and Level 4 counterparts.

Drag Bars

74. Drag bars were installed on Levels 4, 5 and 6 on both sides of the lift well walls of the North Core to provide an earthquake resistant connection between the North Core and the floor slabs. As illustrated in Figure 25 [**WIT.HEYWOOD.0001.54**], the drag bars did not prevent the slab from detaching from the North Core as either the slab failed around the drag bars (e.g. Level 6) or the slab pulled away from the drag bars (e.g. Level 5).

The drag bars remain attached to the lift well. In some places, a piece of Hi Bond metal decking remained attached to the drag bars (refer Figure 25).

75. The drag bars on the eastern side of the lift well on Level 6 were cut using oxy-acetylene cutting equipment during the removal of the segment of the slab in front of the lift well. The beam reinforcement at the western end of on this floor segment was also cut (refer Figure 28 [**WIT.HEYWOOD.0001.57**]).

76. The portion of the drag bars that cantilevered from the lift well were all bent downwards, consistent with supporting the weight of the floor slabs during the collapse rather than the forces required to resist earthquake forces, which would have applied principally horizontal forces to the drag bars.

The way the internal beams (Lines 2 and 3) of the Building fell

77. As discussed above, the upper level internal beams on Line 3 moved north with the floor slabs by a distance of 3 to 4 metres (refer also Figure 30 [**WIT.HEYWOOD.0001.59**] and Figure 31 [**WIT.HEYWOOD.0001.60**]). The internal beams have rotated with the floor slabs. The floor slabs have "hinged" at their junctions with these internal beams. Examples of these "hinges" are also presented in Figure 32 [**WIT.HEYWOOD.0001.61**], Figure 33 [**WIT.HEYWOOD.0001.62**] and Figure 48 [**WIT.HEYWOOD.0001.77**].

78. Figure 33 [**WIT.HEYWOOD.0001.62**] is photographed from a similar point to the photograph in Figure 15 [**WIT.HEYWOOD.0001.44**], but after the roofing material and the column had been removed. The end reinforcement that once tied the Line 3 Level 6 beam (B10(6)) into column C16 and the formed edges of the slab on Line A are visible. Some of the slab reinforcement has been cut in preparation for removing the floor.

79. Two "hinges" have formed in the Level 6 floor slab at the internal beam on Line 3 – one "hinge" on the southern side of the internal beam, and the second "hinge" on the northern side of the internal beam (refer Figure 33). These hinges have rotated in opposite directions and the level of the floor slab changes level across the internal beam B10(6), which has rotated such that the top of the beam has moved towards the south compared to the bottom of the beam – or clockwise when looking east. This is the same direction of rotation evident in the Line 3 internal beam presented in Figure 31

and Figure 32. A double "hinge" can also be observed in Figure 48 [**WIT.HEYWOOD.0001.77**].

80. The Level 6 floor slab appears to have been ruptured by column C15, as indicated in Figure 33 [**WIT.HEYWOOD.0001.62**]. The two reinforcing bars circled are from the column, which has separated from the internal beam by 1 to 2 metres.

81. Two internal beams of Line 2 on the eastern side of the Building came to rest almost directly one above the other as shown in Figure 34 [**WIT.HEYWOOD.0001.63**] and Figure 35 [**WIT.HEYWOOD.0001.64**]. The South Wall and the floors to the south of Line 2 and to the east of Line D had been completely removed, exposing the beams on Line 2. The floor levels of the two beams that are visible are not known – probably upper floors.

82. The concrete within the junction between the internal beams (B02 and B03) and column C7 has largely turned to rubble (refer Figure 35). The column and beam reinforcement is visible within the rubble. The upper column beam junction has moved to the east by about twice the column diameter or about 0.8 metre compared to the lower column beam junction.

83. The precast portions of the internal beams appear in sound condition (refer Figure 35). They appear to have rotated in an anticlockwise direction when looking east with possibly a larger rotation evident in the eastern beams (B02) compared to the western beams (B03).

The way the floor slabs of the Building fell

84. The upper floor slabs detached from the North Core and came to rest leaning on the North Core such that the Line 3 beams moved northward by up to 4 m. This movement was probably less on the lower levels.

85. The floor slabs – particularly the Level 5 and 6 floor slabs – have moved northwards in the area to the west of the North Core in a manner that is broadly consistent with the movements induced by the floor slabs leaning against the North Core.

86. As discussed, the southern ends of the floor slabs fell close to the base of the South Wall – mostly within 1 metre of the base of the South Wall.

87. The resulting increase in length of the floor slab between Lines 1 and 3 of up to approximately 3 metres indicates that the floor slab would have most likely torn in one or more locations between Lines 1 and 3. It is considered likely that the bulk of this "stretching" occurred between Lines 2 and 3 as there was a "valley" in the rubble at this location. This requires confirmation.

88. The observation that that the floor slabs were leaning up against the North Core indicates that Line 3 had collapsed before the floor slabs were detached from the North Core.

89. The movements in the east-west direction appear to have been less significant than those in the north-south direction. It is not possible to comment on any rotation of the floor slabs about a vertical axis.

Exterior Columns

90. The west, south and east perimeter, including the exterior columns, of the Building had been substantially altered before my arrival.

91. The circular columns on the eastern elevation of the Building had been removed before my arrival. Some segments of the circular columns (C2 and C3) were on site. These had broken into pieces that were a storey length or less (refer Figure 7 [**WIT.HEYWOOD.0001.36**]).

92. Figures 71 and 72 of the Hyland Report (page 156) show the western elevation immediately after the collapse but only column C4 on the southwest corner of the Building is visible. One level of column C4 remained vertical after the collapse although all the concrete is missing at its base. The block walls and light weight cladding on the western elevation that obscured the columns from view had been removed, along with column C4, before my arrival. The rectangular columns observed on the western elevation were detached from the Building as continuous lengths, but the concrete in the vicinity of the column beam junctions had been lost. The segments of columns between the floors appeared sound with the segments connected by the column reinforcement.

93. Column C19 on the northern elevation collapsed against the brick wall of the adjacent undercover car park (refer Figure 36 [**WIT.HEYWOOD.0001.65**]). The path of the column is complex. C19 appears to have been moved with the collapsing edge beams. The ends of the edge beam attached to C19 came to rest near the base of the North Core, on top of the roof of the adjacent car park, near the base of the adjacent car park, near the base of the North Core, and between those last two points.

94. Like the columns on the western wall, two storey-long segments of column C18 remained intact and joined by the column reinforcement. These columns came to rest on top of the floors they once supported. C19 is unique in that it was connected to the top of the North Core and so could support some loads even if the columns below had been lost.

Interior Columns

95. A number of the interior columns above Level 6 remained intact after the collapse, with some maintaining their connection with the Level 6 floor slab and interior beams.

96. Below Level 6, I gained an overall impression that the columns had largely been reduced to reinforcement and rubble (e.g., Figure 35 [**WIT.HEYWOOD.0001.64**]) with an occasional short length of column.

Beam-Column Joints

97. Most beam-column connections on the eastern elevation had all been removed before I arrived. No observations are made in relation to these joints.

98. Some of the edge beams on the western side of the South Wall were still in place on my arrival (refer Figure 7 [**WIT.HEYWOOD.0001.36**]). Figure 7 and Figure 37 [**WIT.HEYWOOD.0001.66**] show that many of the beam-column connections had pulled apart and the concrete had been lost from the connections.

99. Some of the beam-column connections between the top floor (Level 6) and the columns, which supported the roof, remained intact.

100. It is likely that the connections between the rectangular columns on the western elevation and the adjoining precast interior beams, narrow edge beams and sill beams disintegrated during the early stages of collapse because:

a. The rectangular columns had generally become detached from the Building;

b. The lengths of the intact segments of the columns observed were similar to the clear storey heights. If the column segments, which were in sound condition, had remained attached during the collapse, they would have suffered considerable damage as the floors fell or the floor slabs would not have come to rest one above the other as observed;

c. The concrete had either been lost from the ends of the precast beams or the end of the beam had pulled out of the column.

101. The interior beams generally remained connected at the interior columns through the top reinforcement in the beams that continued across the column. However, the concrete within the joint had been lost. I did not observe an interior beam column connection where the concrete in the joint immediately above and below the beams had not been lost or rendered ineffective other than on Level 6.

Concrete

102. With respect to the concrete in the Building, I observed that the concrete disintegrated into rubble more readily than I would have expected (refer Figure 38 [**WIT.HEYWOOD.0001.67**] and Figure 42 [**WIT.HEYWOOD.0001.71**]). Many people at the Building site noted how readily the concrete turned to rubble. It was surprisingly difficult for a machine to lift any substantial piece of concrete slab without it breaking into pieces and rubble. This made the concrete challenging to remove. This was true in all areas – those that had been and had not been affected by the fire.

103. Towards the end of the search and rescue operation, all of the rubble was being sifted and tested as part of the search for human remains. Consequently it was desirable to remove concrete without it turning into rubble. I observed three machines operating in unison in an attempt to load one slab of concrete at a time onto trucks so as to minimise its disintegration into rubble (refer Figure 39 [**WIT.HEYWOOD.0001.68**]). I

considered this to be unusual and a consequence of the poor tension strength of the concrete.

104. The coarse concrete aggregates were rounded gravel up to about 20 millimetres in size. The aggregates were observed in the rubble or in any of the surfaces where the concrete had failed. Figure 40 **[WIT.HEYWOOD.0001.69]** and Figure 41 **[WIT.HEYWOOD.0001.70]** show typical failure surfaces, which are characterised by the rounded aggregate projecting from the surface or the rounded impressions of aggregate. Almost all the coarse aggregates had pulled from the matrix rather than the pieces of coarse aggregate breaking. This indicates that increasing the bond between the matrix and the coarse aggregate would have increased the concrete's tensile strength.

105. The combination of the pulling of coarse aggregate from the concrete matrix, the propensity for the concrete to disintegrate into rubble and some reinforcement/drag bars being pulled from the concrete rather than breaking, raised my concern about both the compression strength and the tension strength of the concrete. Concrete with poor tension strength also has reduced shear strength and reduced ability to anchor the reinforcement in the concrete.

Floor Reinforcement Mesh

106. The notes on Drawing S 15 state: "Reinforce slab with 1 layer of 664 mesh throughout, 30 mm top cover". Drawing S 15 shows the 664 mesh extending into edge beams and the South Wall. The 664 mesh consists of 6 millimetres (1/4 inch) diameter wires welded together at 150 millimetres (6 inch) centres to form a square grid. D12 reinforcing bars perpendicular to edge beams, internal beams, and the South Wall supplement the mesh and complete the top layer of reinforcement in the floor slabs. There was no additional top reinforcement in the east-west direction.

107. The reinforcement in the bottom of the slabs was provided by the Hi Bond metal decking. There was no additional bottom reinforcement specified on the drawings except for 2 bars along the western edge of Level 4.

108. The mesh was typically at the top of the floor slab in the vicinity of the beams, but away from the beams it could be seen at the bottom of the floor slab rather than in the top of the slab as specified (refer Figure 42 **[WIT.HEYWOOD.0001.71]**).

109. The mesh, which is less ductile than the reinforcing bars, provided approximately half the tension capacity of the reinforcement used to tie the floor slabs to the edge beams. The connections between the floor slabs and the edge beams were all severed during the collapse. These connections are discussed below.

Edge Beams

110. Edge beams (960 millimetres wide) trimmed the north, east and south edges of the Building except for the narrow edge beams between Lines A and B on the north and south elevations. They were formed from precast concrete "shell" beams made integral with the cast-in-situ slab. The edge beams spanned between circular columns and supported precast spandrels.

111. All observed connections between the edge beams and the floor slab were severed during the collapse with the two observed failure surfaces illustrated in Figure 43 [**WIT.HEYWOOD.0001.72**]. Both failure surfaces went around the end of the Hi Bond metal decking.

112. Examples of the vertical failure surface indicated by the solid line in Figure 43 are shown in Figure 22 [**WIT.HEYWOOD.0001.51**], and Figure 40 [**WIT.HEYWOOD.0001.69**]. In these cases, both the mesh reinforcement and the supplementary reinforcement have failed in tension.

113. An example of the vertical and horizontal failure surface, indicated by the dashed line in Figure 43 [**WIT.HEYWOOD.0001.72**], is shown in Figure 26 [**WIT.HEYWOOD.0001.55**]. In these cases, both the mesh reinforcement and the supplementary reinforcement have torn from the top surface of the beam, possibly promoted by low concrete cover.

114. The edge beams were typically extracted from the rubble in one piece with little sign of structural damage to the precast elements except at their ends (refer Figure 26 and Figure 37 [**WIT.HEYWOOD.0001.66**]).

115. The top reinforcement connecting the edge beams to narrow edge beams and internal beams needed to be cut to allow each precast element to be removed. Similarly, some of the connections between the edge beams and the North Core and South Wall needed to be cut, but others pulled apart during the collapse.

116. It is not possible to comment on the connectivity of the edge beams on Line F as most had been removed before my arrival.

Narrow Edge Beams

117. Narrow edge beams (400 millimetres wide) trimmed the north and south edges of the Building between Lines A and B. They were formed from precast concrete beams made integral with the cast-in-situ slab. The narrow edge beams spanned between a rectangular corner column and a circular column. They did not support precast spandrels.

118. All observed connections between the narrow edge beams and the floor slabs were severed during the collapse with the two observed failure surfaces illustrated in Figure 44 [**WIT.HEYWOOD.0001.73**].

119. An example of the vertical failure surface illustrated in Figure 44 (solid line) is shown in Figure 45 [**WIT.HEYWOOD.0001.74**]. In this case, only one wire from the mesh crossed the failure plane. This suggests the mesh had practically zero overlap of the reinforcement cage.

120. An example of the vertical and horizontal failure surface, indicated by the dashed line in Figure 44 [**WIT.HEYWOOD.0001.73**], is shown in Figure 46 [**WIT.HEYWOOD.0001.75**]. In this case, the failure surface includes a horizontal surface between the mesh and the top of the beam reinforcement cage, indicating that even if the overlap were sufficient, the structure may fail around the reinforcement.

121. The narrow edge beams were typically extracted from the rubble in one piece with little sign of structural damage to the precast elements except at their ends (refer Figure 45 [**WIT.HEYWOOD.0001.74**]).

122. The top reinforcement connecting the narrow edge beam to the edge beam needed to be cut to allow each precast element to be removed, although large changes in angle occurred at the joint, for example as illustrated in Figure 22 [**WIT.HEYWOOD.0001.51**].

Internal Beams

123. Internal beams supported the floor slabs along Lines 2, 3 and 4. They were formed from precast concrete beams made integral with the cast-in-situ slab. The beams at the eastern and western ends are tapered in elevation. The precast beams were supported on rectangular columns on Line A, the North Core and 400 mm diameter circular columns elsewhere. The ends supported on circular columns were cylindrically recessed (refer Figure 49 [**WIT.HEYWOOD.0001.78**]) to provide formwork for the columns for the depth of the beams.

124. The internal beams separated from the floor slabs during the collapse with very few exceptions. The observed failure surface is shown in cross-section in Figure 47 [**WIT.HEYWOOD.0001.76**]. Often the beam had rotated so it was no longer perpendicular to the floor slab, as illustrated in Figure 48 [**WIT.HEYWOOD.0001.77**]. "Hinges" also formed in the thin residual slab (refer Figure 33 [**WIT.HEYWOOD.0001.62**]).

125. Other examples that show the internal beams separated from the floor slabs are presented in Figure 17 [**WIT.HEYWOOD.0001.46**] and Figure 26 [**WIT.HEYWOOD.0001.55**]. The failure surface can be seen on an internal beam after its removal from the rubble in Figure 49 [**WIT.HEYWOOD.0001.78**].

126. The internal beams were typically extracted from the rubble in one piece with little sign of structural damage to the precast elements except at their ends (refer Figure 49).

127. The top reinforcement connecting the internal beams needed to be cut to allow each precast element to be removed (refer Figure 49).

North Core

128. On arrival, I observed the visible portions of the North Core were still in good condition other than where damage had occurred from the edge beams detaching from the walls. A closer inspection of the northern face of the North Core revealed two horizontal cracks in the rendered concrete within 1 metre of ground level (refer Figure 50 [**WIT.HEYWOOD.0001.79**]). The cracks indicated that either the North Core may have been bent towards the south so as to crack the concrete or that this was a pre-existing crack. The staining below the crack supports the view that the crack was a pre-existing

crack. In either case, the North Core appeared to have performed well during the earthquake. I did not inspect the tower closely after the debris was removed.

South wall

129. The South Wall collapsed on top of the rubble, bringing with it the escape stair (refer Figure 5 [**WIT.HEYWOOD.0001.34**], Figure 8 [**WIT.HEYWOOD.0001.37**] and Figure 9 [**WIT.HEYWOOD.0001.38**]). The cracking at the base indicated that the shear wall had suffered damage (Figure 9) but withstood the east-west loads applied by the earthquake.

130. The South Wall collapsed by bending about an axis parallel to Line 1 at about Level 2 (refer Figure 13 [**WIT.HEYWOOD.0001.42**]). As previously observed, the floor slabs are likely to have become detached from the South Wall before the South Wall collapsed (refer paragraph 38).

131. When the South Wall collapsed, it dragged at least some of the edge beams that were directly attached to it northwards by distances consistent with the height of the point of attachment above the "hinge" line.

General

132. Ductile structures are desirable, especially in earthquake regions, because of the large deformations that occur before they fail. This provides a warning of impending collapse as well as the opportunity for the structure to find alternative load paths to support the load. Normal grade steels are considered ductile because they deform by 20% or 30% before they fail.

133. Brittle structures provide no warning of collapse and often no opportunity to redistribute the load. A stick of chalk, for example, is brittle when it is bent. One moment the chalk is carrying the load and the next moment it has failed without warning. Columns can also behave in a brittle manner, even if made from a ductile material. If a column is too slender, it can suddenly lose strength (buckle), even though there are considerable reserves of strength in the material.

134. Reinforced concrete structures are formed by combining concrete, which is brittle in tension, with reinforcing steel, which is ductile in tension. The concept is to provide

sufficient steel reinforcement bonded within the concrete so that the reinforcement supports the internal tension forces that the concrete cannot sustain. The internal tension forces generated within the concrete are transferred to the steel reinforcement when the concrete cracks. In this way, brittle concrete is supported by ductile steel so that the combination of the two – reinforced concrete – is ductile. Considerable care in design and construction is necessary to ensure a ductile result, especially in a situation where a structure is pushed back and forth many times by an earthquake.

135. Sometimes the steel reinforcement is bonded to the outside of the concrete. The Hi Bond metal decking used in the Building provided both the formwork for the construction of the concrete in the floor slabs and the steel reinforcement.

136. Reinforced concrete structures rely on the tension as well as the compression strength of concrete. For example, the bonding of reinforcement to the concrete relies on the tension strength of the concrete. If the bond between the steel and the concrete fails, then the structure will fail, at least locally.

137. Combining ductility and alternative 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). For example, the loss of a column due to a vehicle impact or an explosion should not cause a significant portion of the structure to collapse.

138. The Building collapsed quickly (refer eyewitnesses and the Hyland Report) and catastrophically with many elements becoming detached from each other – possibly in a brittle manner. The following elements of the Building may have contributed to the brittle nature of the collapse:

    a. The columns;
    b. The beam-column connections when subjected to lateral movements;
    c. The connections between the precast elements;
    d. The connections between the floor slabs and the South Wall, the edge beams and the North Core;
    e. The lack of continuous bottom reinforcement in the beams over columns and in the slabs over the internal beams;
    f. The apparently low tensile strength of the concrete;
    g. The low ductility of the mesh reinforcement.

CONCLUSIONS

139. No observations were made in relation to the eastern elevation of the Building as most of the debris had been removed before my arrival.

140. The collapse of the Building reduced 5 suspended floors to a height of less than one storey, except where the floors lent up against the North Core.

141. From my observations detachment was common throughout the Building:

   a. The floor slabs detached from the North Core, South Wall, the edge beams, the internal beams and the sill beams that supported the block walls on the western wall.
   b. The precast edge beams detached from each other, from columns, the North Core and possibly the South Wall.
   c. The Hi Bond metal decking debonded from the floor slabs and detached from its supporting beams and walls.

142. I observed the total loss of concrete from within most beam-column connections.

143. I observed that the concrete turned to rubble readily, that reinforcement debonded from the concrete and that the concrete failure surfaces were characterised by a loss of bond with the coarse aggregate.

144. Based on what I observed, I consider it likely that:

   a. The southern end of the Building has fallen essentially vertically but there have been northward movements up to 4 metres at the northern end of the floor slabs;

   b. The floors have been extended by up to approximately 3 metres in the north-south direction by one or more tears in the floor slab;

   c. Line 3 of the Building collapsed before the floor slabs were detached from the North Core;

   d. The floor slabs detached from the South Wall prior to the South Wall collapsing;

e. The rectangular columns on the western elevation became detached from the Building at an early stage in the collapse.

Signed: ....
ROBERT JAMES HEYWOOD

Date: 22 May 2012

Figures in Brief of Evidence Robert James Heywood In relation to the CTV Building

Figure 1

Schematic plan of the Building showing the general layout of the structural elements including: North Core, South Wall, columns, internal beams and edge beams.

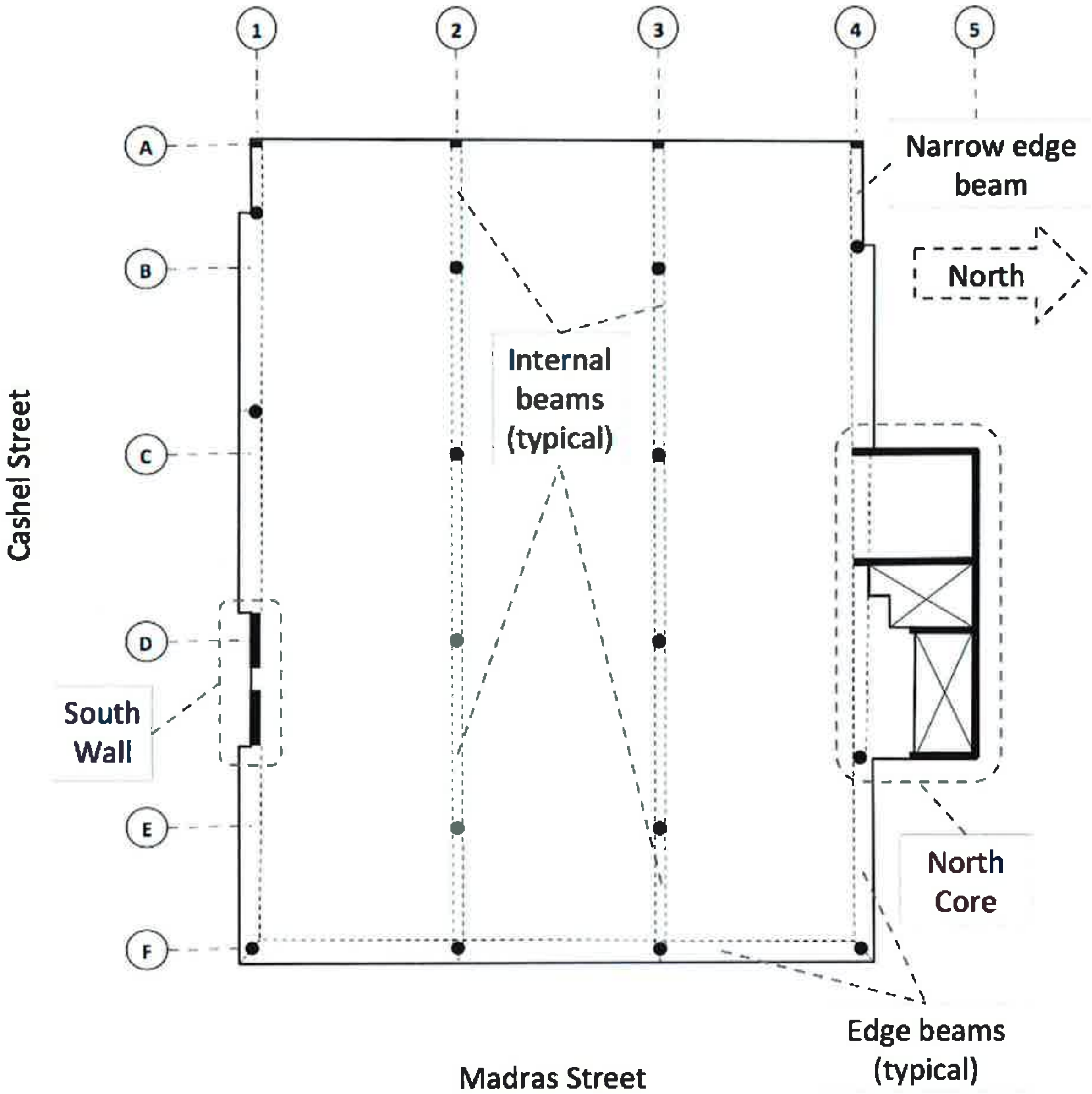

Figure 2

Precast beam location plan (Extract from Drawing S 18)

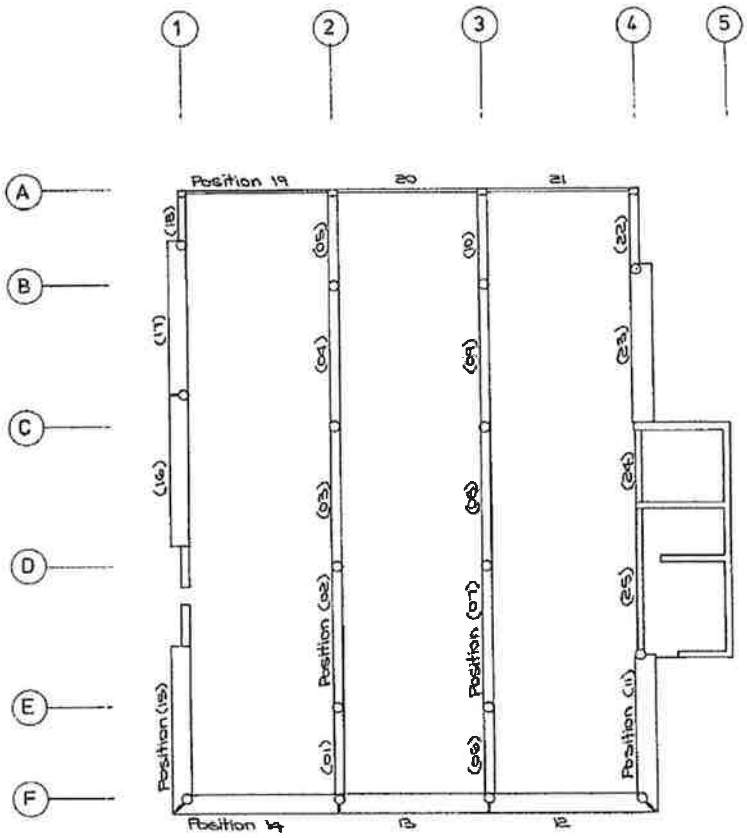

Figure 3

Column location plan (Extract from Drawing S 14)

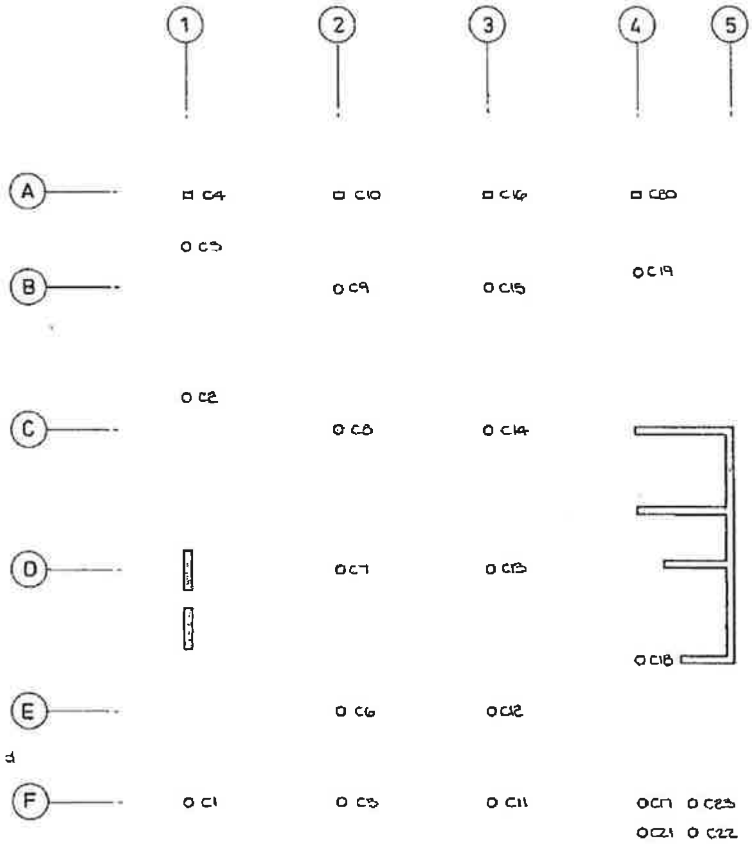

Figure 4

My initial observation of the CTV Building from Madras Street looking southwest. Some remnants of the floor slabs were hanging from the North Core on Level 6. Concrete beams and precast panels were on top of the pile of debris adjacent to the North Core. An excavator was stockpiling debris on Madras Street. (4:00 AM 24 Feb 2011)

Figure 5

The CTV Building seen from the corner of Cashel and Madras Streets on arrival. The burnt North Core remained standing and smoke was rising from the Building rubble with the fire brigade in attendance. An excavator was working removing rubble and stockpiling it in Madras Street. The external stairs and their supporting South Wall had collapsed onto the Building. Most of the material in the foreground appeared to have been relocated (4:00 AM 24 Feb 2011).

Figure 6

An excavator removing a column from the western face (Line A) of the CTV Building viewed from midway between Line 1 and 2 towards the north. There is one segment of the block wall standing in the background and one other segment on the ground. The "smooth" edges of two floor slabs (Line A) are evident with the Hi Bond metal decking separated from the underside of the lower slab (6:30 AM 24 Feb 2011).

Figure 7

Southern elevation of the Building to the west of the South Wall showing precast spandrel beams and their supporting edge beams, and an external circular column (6:20 AM 24 Feb 2011).

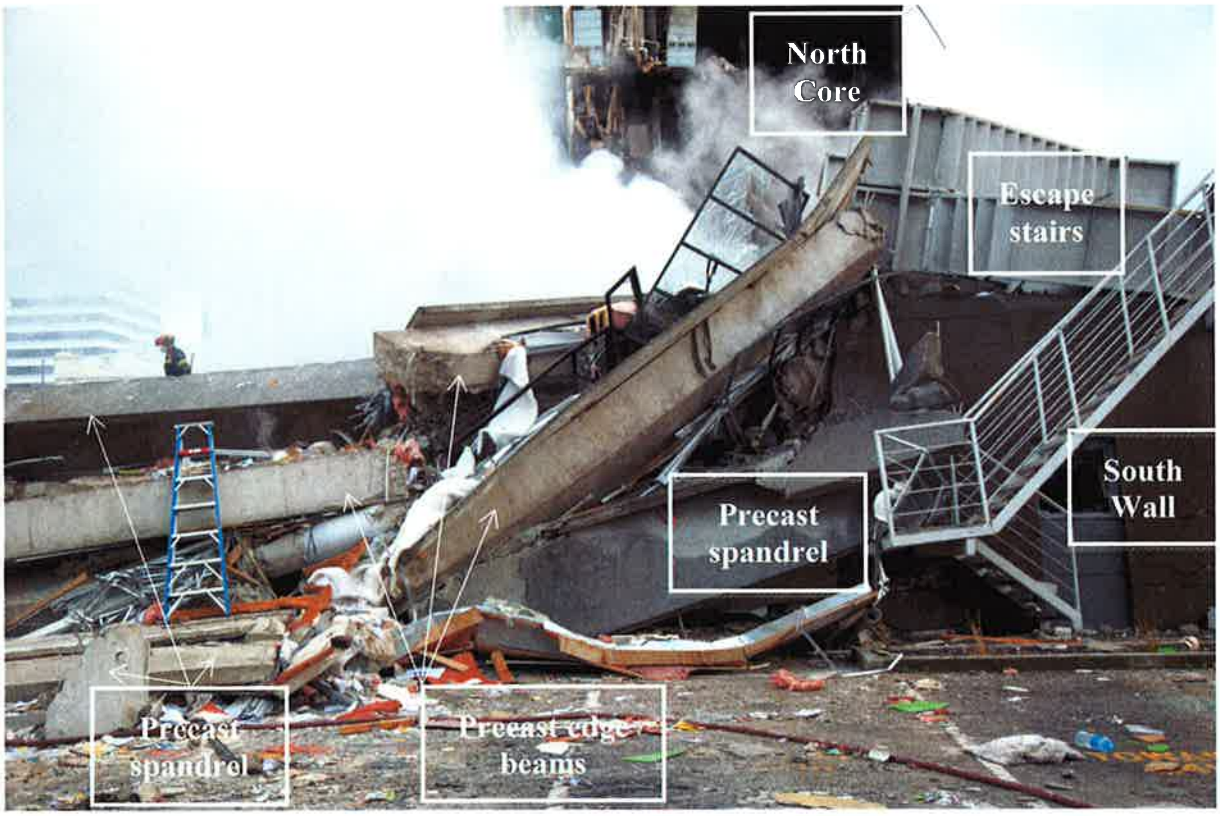

Figure 8

The collapsed South Wall lies on top of the debris. A Line 1 edge beam is in the foreground. A second line 1 edge beam lies on top of the debris almost parallel to the collapsed South Wall. A Line 4 edge beam lies on the rubble in front of the North Core and beside Column C18. The arrows indicate the edge beams (8.30 AM 24 Feb 2011).

Figure 9

View of South Wall and fire escape stair collapsed onto the Building. The wall "hinged" about a line parallel to its base at about Level 2. The edge beams are no longer attached to the lower levels on the eastern side of the South Wall (4:30 AM 24 Apr 2011).

Figure 10

Two slabs were visible through the opening in the South Wall immediately below the escape stair landing at Level 2. The Hi Bond metal decking is visible below the two levels of slab.

Figure 11

A close-up of the upper slab showing fracture surface through the concrete, the "664" mesh reinforcement and the Hi Bond metal decking.

Figure 12

Southern edges of floor slabs from Levels 2 (L2), 3 (L3), 4 (L4) and 5 (L5) immediately to the east of the South Wall on Line 1 (7:20 AM 25 Feb 2011).

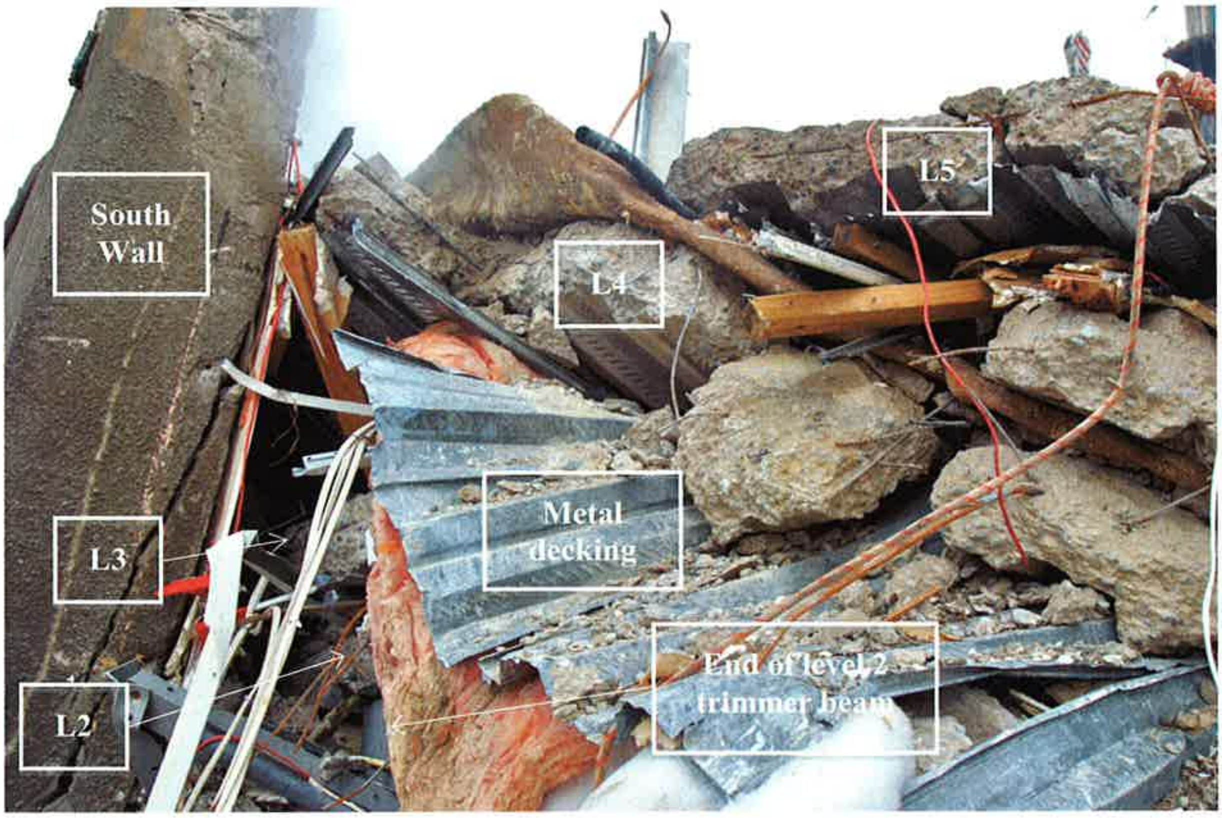

Figure 13

Southern edges of L2, L3, L4, L5 and L6 floor slabs immediately to the east of the South Wall on Line 1 (4:30 AM 24 Feb 2011).

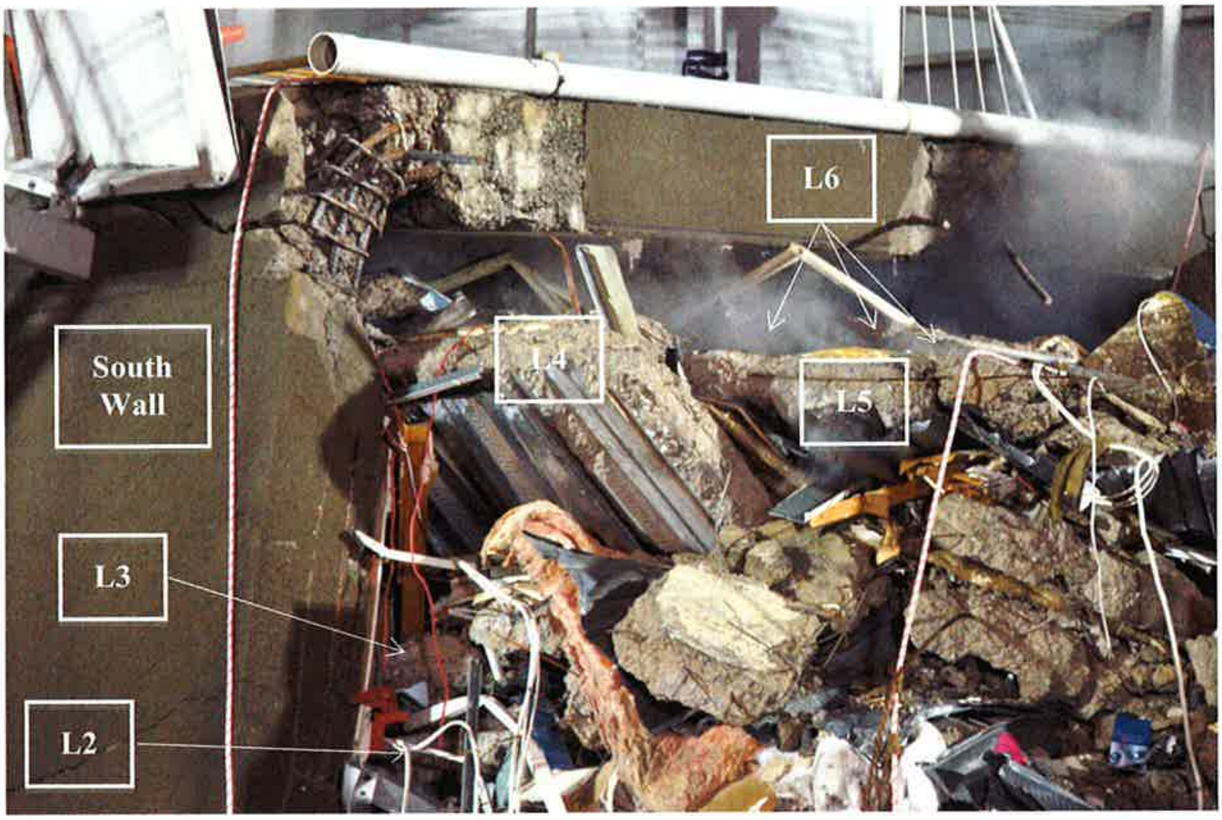

Figure 14

Segments of Line A columns connected by exposed steel column reinforcement (6:30 AM 24 February 2012).

Figure 15

Collapsed slabs lie on top of each along Line A. The ends of the Line 3 internal beam reinforcement have been circled and floor levels of the slabs labelled.

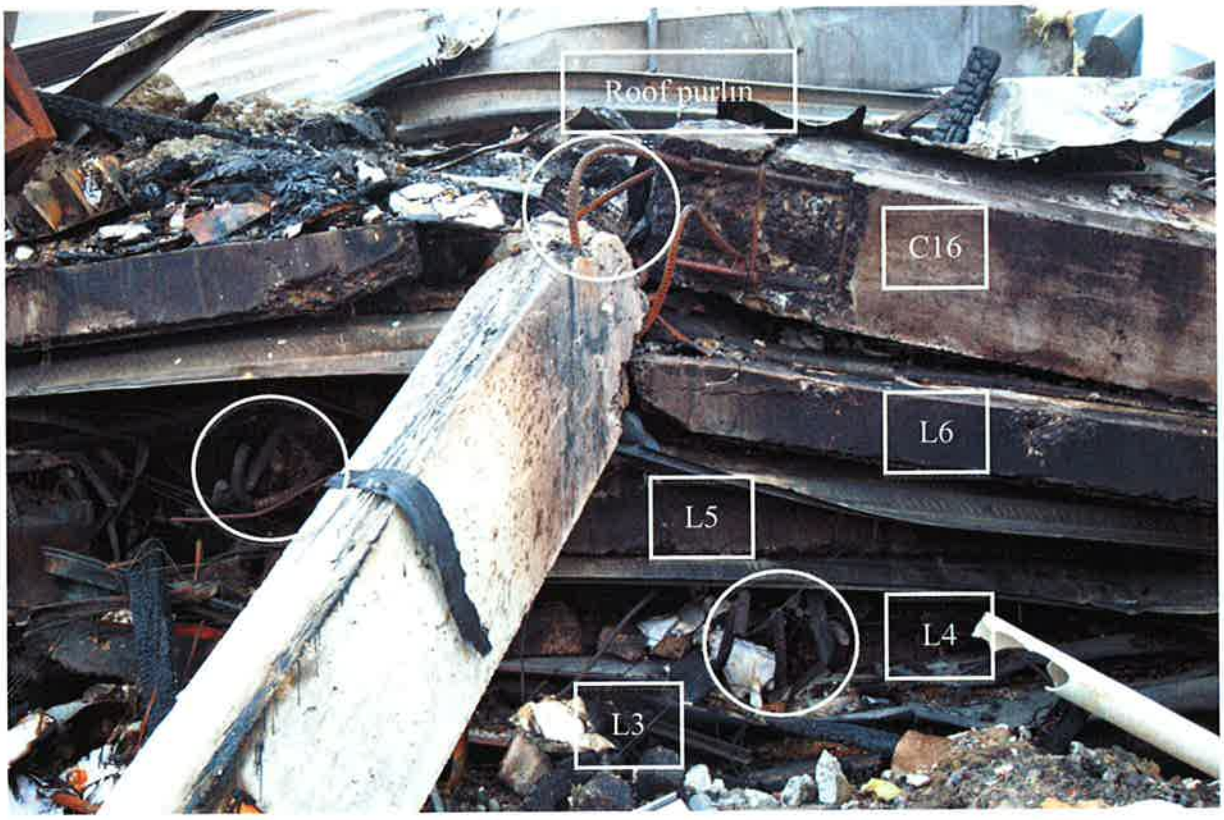

Figure 16

Sill beam starter bars in the edges of the Level 3 floor slab on Line A (6:30 AM 24 February 2011).

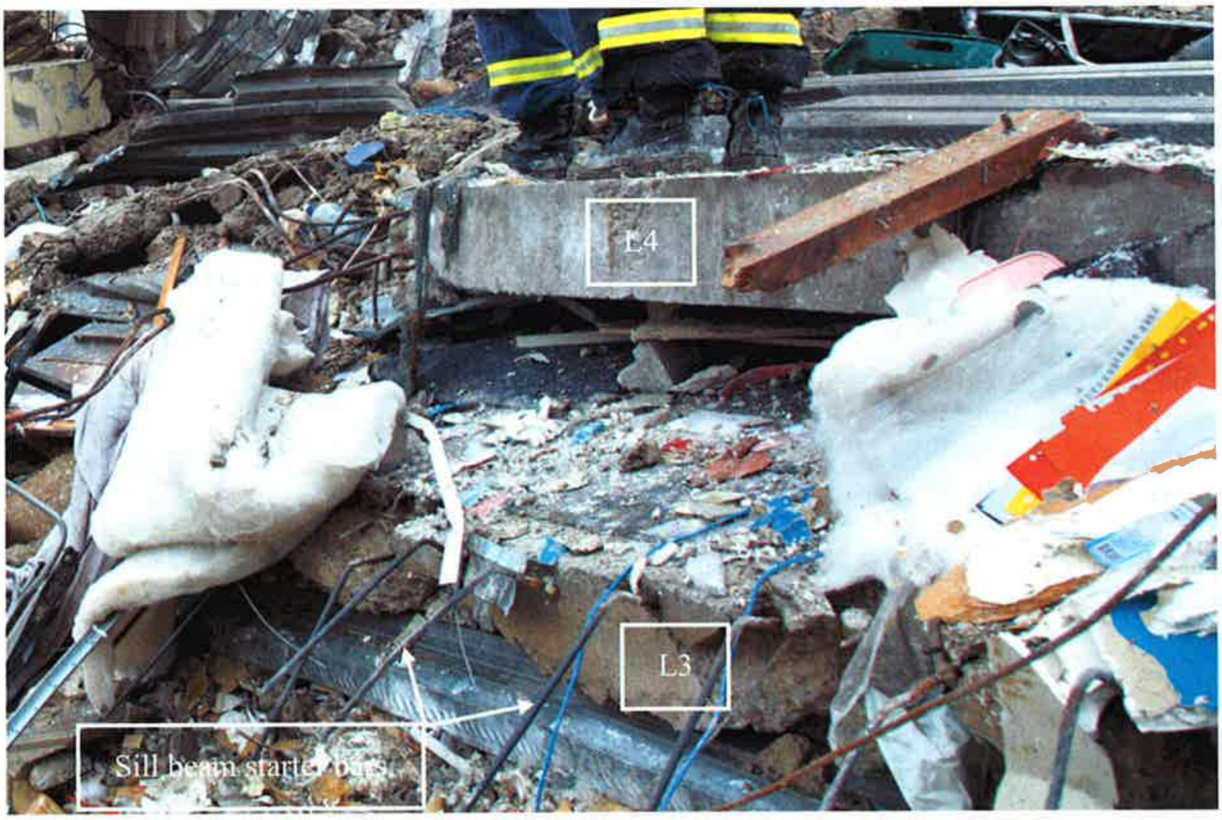

Figure 17

View of the edges of the slabs near Line A looking towards the North Core from near the southwest corner of the Building (6:30 AM 24 February 2011).

Figure 18

Northwest corner of the Building looking towards the North Core.

Figure 19

Northwest corner of the Building looking towards the North Core (9:10 AM 25 Feb 2011).

Figure 20

This western end of a northern narrow edge beam (Line 4) came to rest at the base of the brick wall to the adjacent covered car park.

Figure 21

Western elevation of the North Core and collapsed building (5:45 AM 25 Feb 2011).

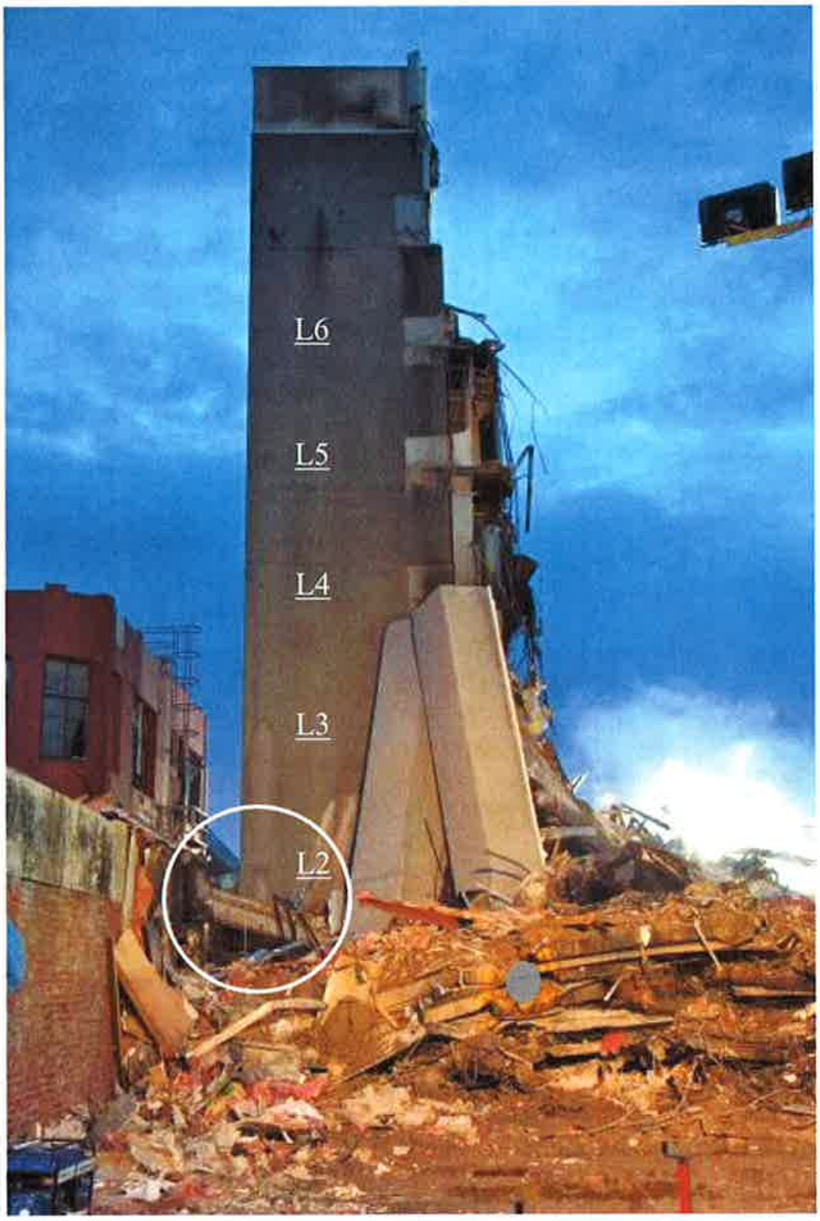

Figure 22

View of the northwest corner of the Building viewed from the collapsed Building looking to the northwest over the undercover car park (12:00 noon 25 Feb 2011).

Figure 23

View of northern wall from Madras Street looking west. An edge beam and precast spandrel lie on top of the rubble. The top of the South Wall staircase and some roof framing are visible on the left and the spandrel of the northwest edge beam that fell onto the undercover car park can been seen at the end of the laneway (7:20 AM 24 Feb 2011).

Figure 24

View of collapsed building and North Core from Madras Street looking west (4:00 AM 24 Feb 2011).

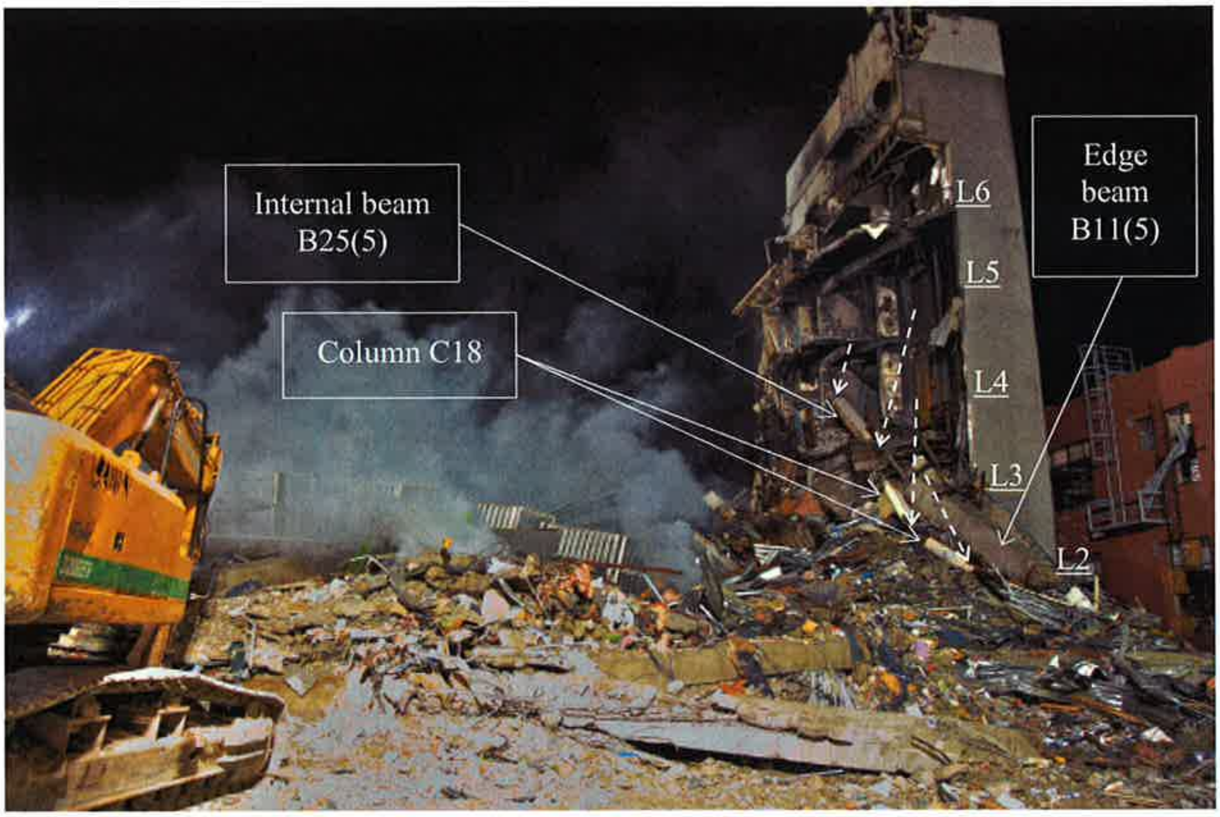

Figure 25

View of North Core showing lift, remnants of Level 5 and 6 floor slabs, supporting internal beams and Level 5 drag bars (10:30 AM 24 Fen 2011).

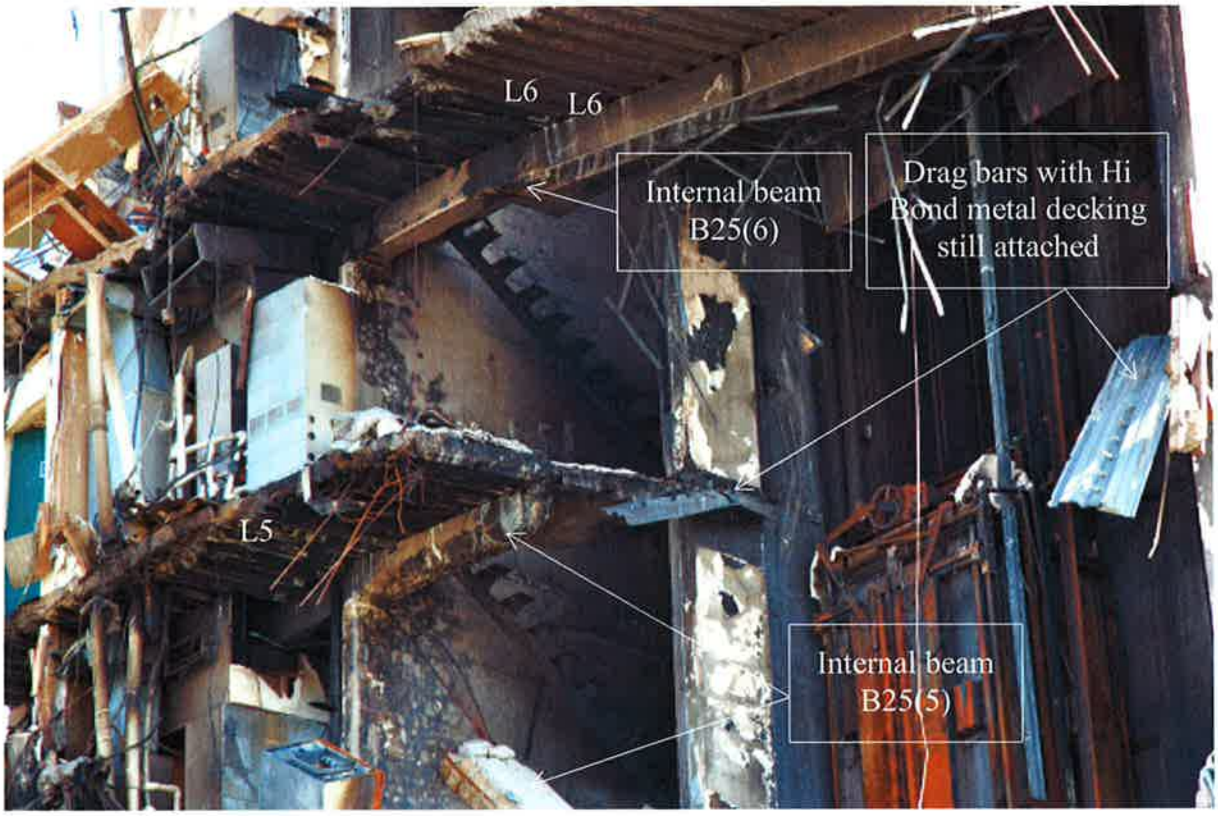

Figure 26

View of North Core looking west after partial removal of rubble (11:45 AM 25 Feb 2011).

*

Figure 27

View of the edge of the severed Level 6 slab to the south of the amenities in the North Core after the slab had been removed from in front of the stair well and the lift well (1:20 AM 26 Feb 2011).

Figure 28

The remnant of the Level 6 slab being cut away from the North Core (1:00 AM 25 Feb 2011).

Figure 29

View looking north of the floors leaning against the North Core showing the Level 6 slab and the top of column C14 (Lines C and 3) still connected to the slab (11:55 PM 25 Feb 2011).

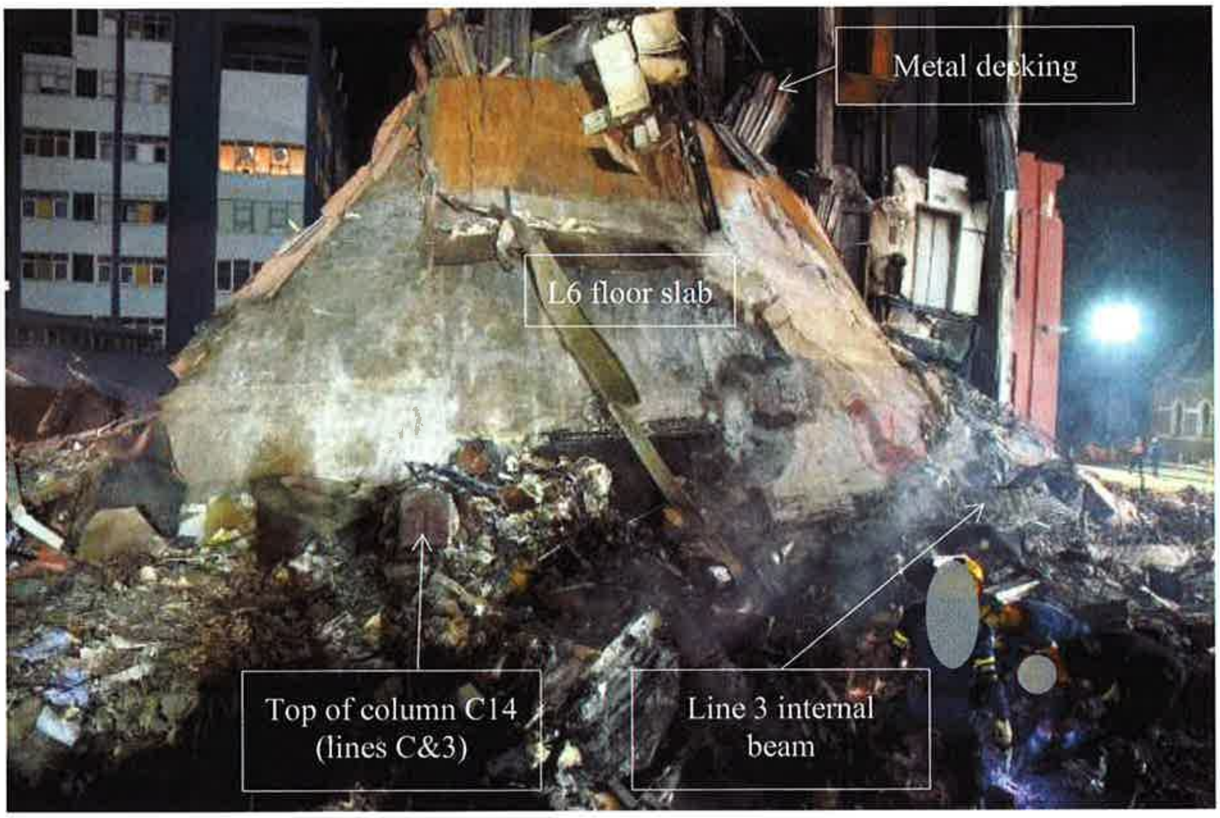

Figure 30

View of the western elevation of the North Core and the collapsed floors leaning against it (3:30PM 1 March 2011).

Figure 31

View of the western elevation of the North Core and the collapsed floors leaning against it – the area contained in the rectangle in Figure 30 (3:30 PM 1 March 2011).

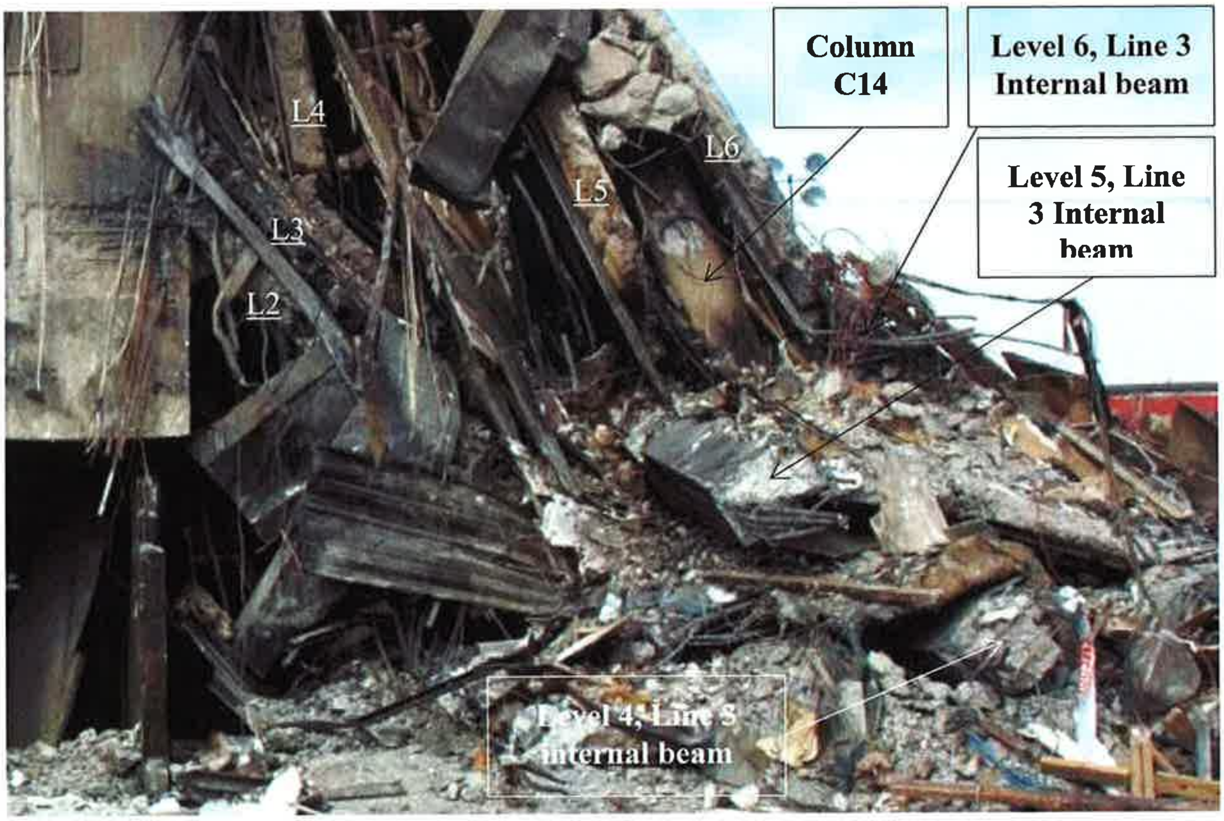

Figure 32

View to the northwest of the slabs leaning against the North Core (4:20 AM 26 Feb 2011).

Figure 33

View of a slab double "hinge" in the Level 6 slab viewed from the western elevation looking northeast towards the North Core (8:20 AM 25 March 2011).

Figure 34

View towards the north of the Line 2 internal beams and the North Core after the South Wall and the floors between Lines 1 and 2 had been removed. A more detailed view of the area approximately enclosed by the rectangle is presented in Figure 35.

Figure 35

Junction between Line 2 internal beams and column C7 looking north towards the North Core. Refer to Figure 34 for a wider view (7:00 AM 26 Feb 2011).

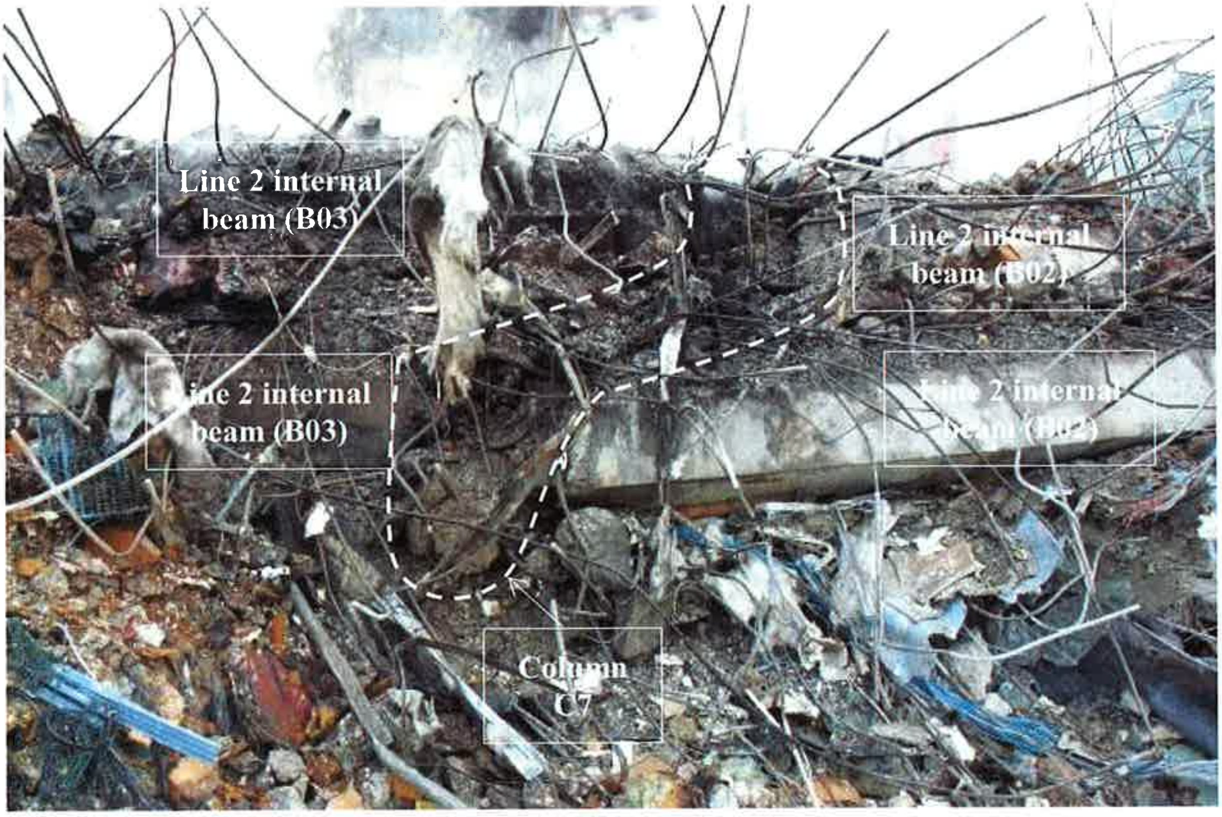

Figure 36

View of the northwest corner of the Building looking towards the North Core after removal of edge beams, spandrel panels and some of the floor slabs. (3:50 AM 27 Feb 2012).

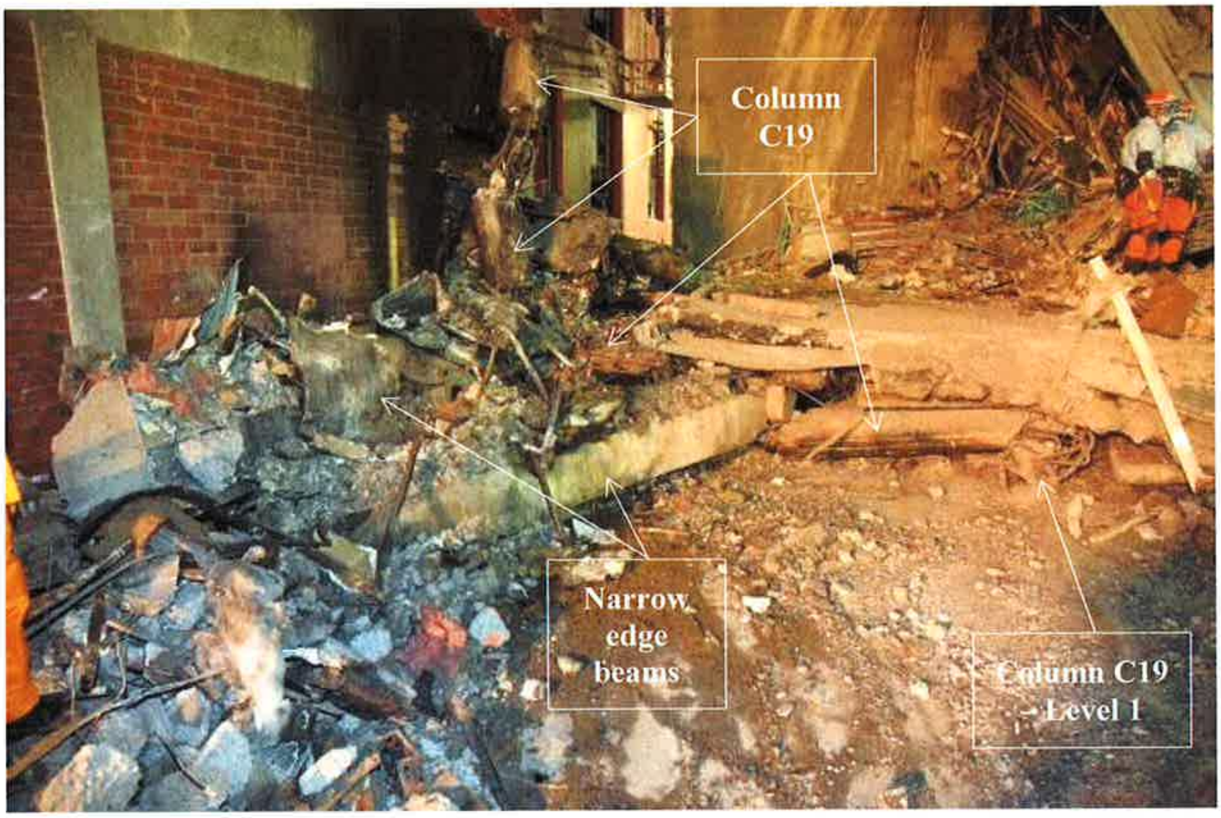

Figure 37

View of an edge beam and a segment of a circular column being removed from the west of the South Wall on Line 1 (10:30 AM 24/ Feb 2011).

Figure 38

View of the concrete breaking down into rubble on the southeast corner of the Building (4:45 AM 24 Feb 2011).

Figure 39

Three concrete demolition machines working in unison to remove a concrete floor slab from the Building while minimising rubble generation.

Figure 40

View of the failure surface where the 200 mm thick floor slab was severed from a north wall edge beam showing the broken wires from the 664 mesh (circled), a broken H12 reinforcing bar and the metal stop used to seal the ends of the Hi Bond metal decking during concreting.

Figure 41

An edge beam after removal from the Building showing the "shell" beam construction, the shelf and seal plates for the Hi Bond metal decking, and the failure surfaces characterised by the aggregate pulling out of the concrete matrix (6:50 AM 25 Feb 2011).

Figure 42

Two examples of concrete floor slabs where the 664 mesh is close to the bottom of the floor slabs and the Hi Bond metal decking.

Figure 43

Precast edge beam to slab connection for perimeter beams except spanning between Lines A and B on Lines 1 and 4 (extract from Dwg S 15). The two failure paths observed are indicated. The "Hi Bond Formwork" noted is referred to in this Brief of Evidence as "Hi Bond metal decking".

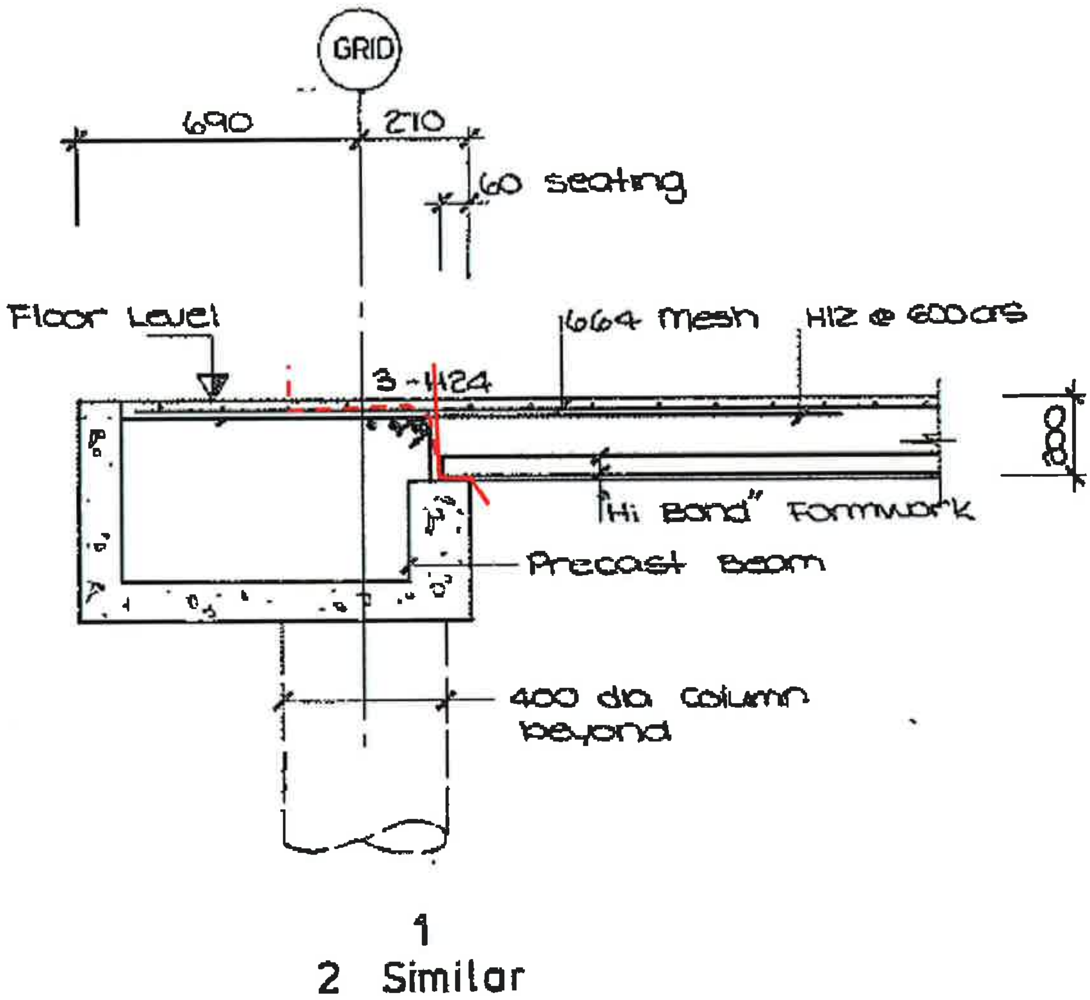

Figure 44

Narrow precast edge beam to slab connection for beams spanning between Lines A and B on Lines 1 and 4 (extract from Dwg S 15). The two failure paths observed are indicated.

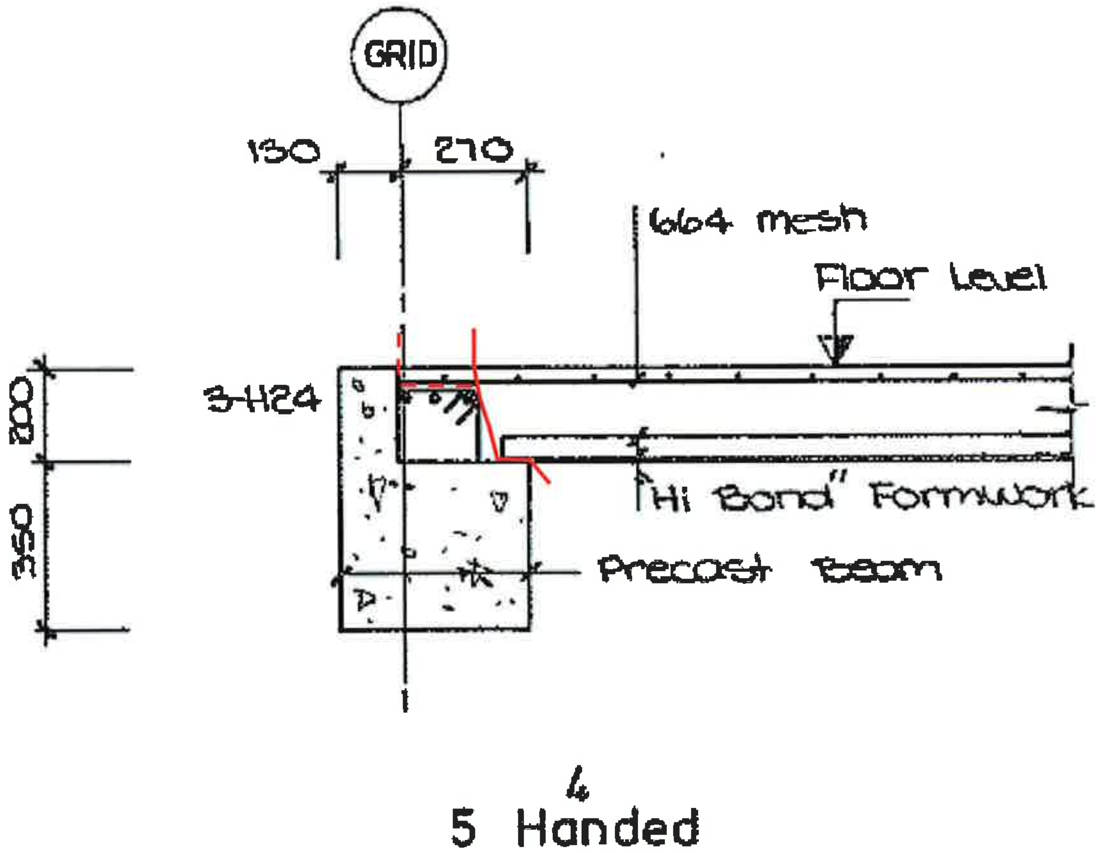

Figure 45

Narrow edge beam from Line 4 between Lines A & B showing failure of the slab to beam connection along a nominally vertical plane (Exhibit E18 with black brown carpet).

Figure 46

Narrow edge beam from Line 4 between Lines A & B showing the concrete broken away from both the front and the top of the reinforcement cage.

Figure 47

Section through precast internal beam to slab (extract form Dwg S 15). The typical failure surface observed is indicated.

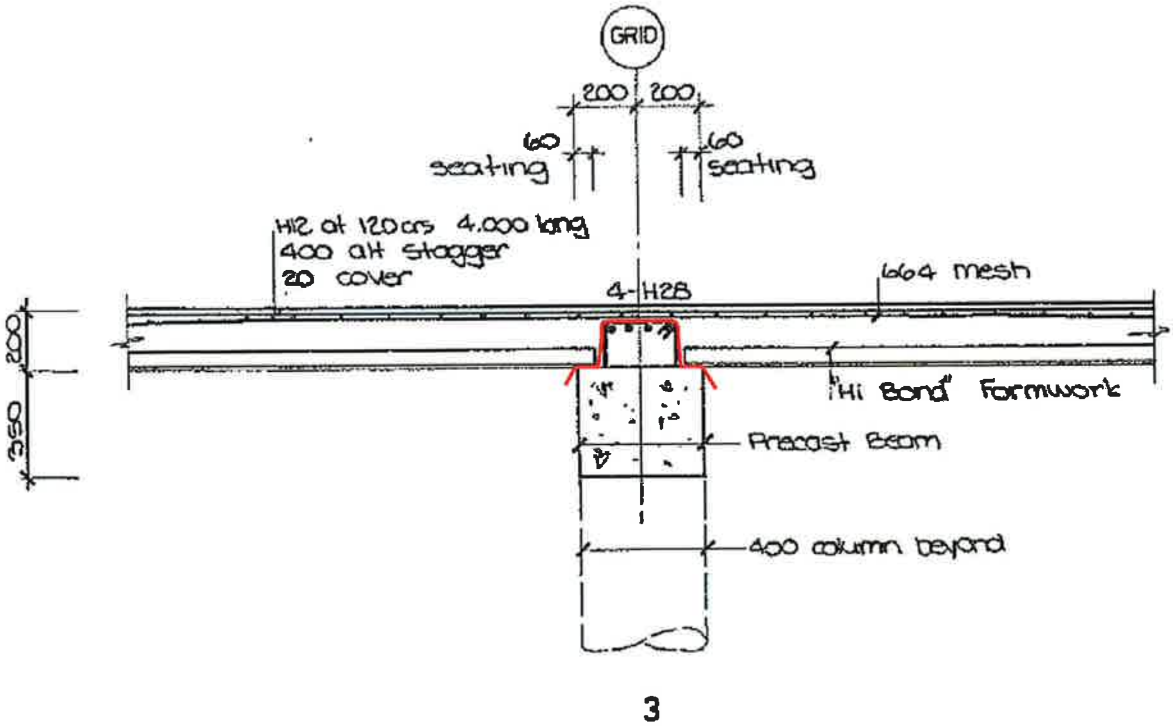

Figure 48

Demolition equipment lifting a slab that had separated from a precast internal beam. The top reinforcement (D12 @120 and 664 mesh) has been cut along the centre of the precast internal beam to facilitate the removal of the slab (8:30 AM 25 Feb 2011).

Figure 49

A precast internal beam lying on its side after extraction from the Building. The beam had completely separated from the slab during the collapse as indicated by the fire blackened concrete on the failure surface. The bent bars protruding from the end, the cylindrical surface on the end of the beam and the oxy-cut top reinforcing bars indicate that the end of the beam to the left of the photograph was supported on a round internal column.

Figure 50

Crack in northern face of the North Core near the western end (7:30 AM 24 Feb 2011).