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 GRAHAM EDWARD FROST IN RELATION TO THE CTV BUILDING

DATE OF HEARING: COMMENCING 25 JUNE 2012

BRIEF OF EVIDENCE OF GRAHAM EDWARD FROST
IN RELATION TO THE CTV BUILDING

1. My full name is Graham Edward Frost. I live in Auckland. I am the Chief Engineer at Fletcher Construction Company. At the time of the response to the February Earthquake I was a USAR Support Engineer for the New Zealand Fire Service. Since November 2011 I have been the nominated Secondee for Fletcher Construction in their Contract with the New Zealand Fire Service to provide consulting engineering services for Urban Search and Rescue (USAR) on an 'as and when required' basis, i.e. a Contracted USAR Engineer.

QUALIFICATIONS AND EXPERIENCE

2. I graduated from the University of Canterbury in 1980 with a Bachelor of Engineering (Civil) and have worked as a Construction/Structural Engineer since 1977 in New Zealand, Malaysia and the USA. While studying at Canterbury I learned about the design and performance of reinforced and prestressed concrete structures from world renowned academics.

3. I hold professional memberships with the Institute of Professional Engineers of New Zealand (IPENZ) and the American Society of Civil Engineers. I am a founding member of the Washington Chapter of the Structural Engineers Institute (of ASCE). I was elected a Fellow of IPENZ in 2008. In 2011 I was awarded the IPENZ Fulton Downer Gold Medal (the Presidents Award). I am a mentor for younger engineers and am a Practice Area Assessor for IPENZ.

4. Early in my engineering career I worked on the redesign of the falsework and construction sequence for New Zealand's highest concrete rail viaduct - the South Rangitikei (Mangaweka) Bridge (that had collapsed during construction early in the 1970's). I also did most of the design of the Temporary Works for New Zealand's longest concrete rail viaduct – the new Hapuawhenua Viaduct on the slopes of Mt Ruapehu. I also advised on and supervised the upgrading measures to the Temporary Works for the iconic LUTH Building in Kuala Lumpur (which had also suffered from collapse problems prior to my involvement). Understanding the performance of concrete structures was also a critical part of my role as Project Engineer on both the Mangere Bridge Project in the early 1980's, the first Tauranga Harbour Bridge in the mid 1980's and then the 1km long Alsea Bay Bridge Project on the Oregon Coast

Highway in the late 1980's. On this last project I was involved in convincing the designers of that bridge that the high concrete stresses we identified in our construction sequence were inherent in any construction sequence; that they hadn't identified these high stresses themselves because that hadn't taken into account the staged construction steps.

5. My experience in the design and construction of reinforced and prestressed concrete structures also includes:

   a) construction supervision on the new Christchurch Hanger Complex for Air New Zealand;
   b) Structural analysis, design and detailing of all structural elements for the 3 level Waikato Valley Authority office building in Hamilton in the late 1970's. This building has an irregular plan with both reinforced concrete and masonry shear walls plus reinforced concrete beams and columns;
   c) Building structure analysis and redesign, detailing and construction supervision of the shear walls for the six level Ambassador Motor Inn building in Hamilton in the late 1970's.

6. I have read and agree to comply with the Code of Conduct for Expert Witnesses.

EVIDENCE

7. My evidence will address the following topics:
   a) My role at the CTV Building site.
   b) My general observations of the Building in its collapsed state.
   c) My observations of parts of the Building which provide some insights into the failure of the Building.

8. I confirm that all of these matters are within my areas of expertise.

MY ROLE AT THE CTV BUILDING SITE

9. After the February 2011 earthquake I spent 5 days in Christchurch as a USAR Support Engineer assisting with the search and rescue and recovery efforts at the Building site. I arrived at the CTV Building site on the evening of 23 February 2011, about 30 hours after it collapsed. The NZ USAR teams typically changed shifts around 6.00am and 6.00pm. The Australian teams typically changed shifts at midnight and noon.

10. My 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.

11. Given the period of time I spent assisting USAR at the Building site and with my role involving the examination of the collapsed Building elements to assess safe removal for USAR and recovery assistance, I made a number of observations about the arrangement and condition of the debris from the Building collapse. I will discuss this in detail shortly.

12. No drawings of the CTV Building were available to me at that time. The only Building details made available to me at handover on 23 February 2011 were sketched floor plans drawn by tenants of CTV for Levels 1 and 2, The Clinic for Level 5 and Relationship Services for Level 6.

13. I also had a copy of a rough sketch John Trowsdale did of his inferred understanding of the structural elements of the Building. [**WIT.FROST.0001.18**]

14. I took several photographs throughout my time at the Building site. The purpose of taking photographs at the site was to capture details of the recovery process, building details, connections and collapse conditions that would be of interest to anyone investigating the Building collapse at a future date. I thought there were many lessons to learn about how some elements of the Building had performed and I felt I had an obligation as a professional engineer to try to capture details of this Buildings performance so the evidence would be available for other engineers. This would help the New Zealand engineering community make informed decisions about future code revisions that may be deemed necessary after this earthquake.

15. As we were perusing the collapse site and going through the process of removing building elements to gain access to spaces where victims may be located it became

more and more clear to me that the Building had failed in a very non-ductile fashion. I was concerned that the multiple handling of building elements by chain from crane hooks and digger buckets, concrete demolition jaws during the deconstruction and removal from site process would increase the damage to those elements. I was also concerned that critical evidence would be destroyed or lost through this multiple handling and disposal process.

16. It was not until I had been on the site for several hours that I started to take photos, initially with just my phone camera. On my second and subsequent shifts I used a digital camera to record the building elements I thought would be of interest to anyone else investigating the collapse. Rob Heywood (Queensland USAR) assisted me with the photographic recording.

17. Counsel Assisting this Royal Commission have shown me Robert Heywood's photographs and I have referred to some of these photographs in this brief of evidence.

18. Believing there would be an investigation into the Building collapse, I arranged for approximately 30 samples from the Building to be kept on site rather than being disposed of immediately off-site with other Building rubble. Each sample was marked with a reference "E" (for Engineering exhibit) plus a number in spray paint.

19. The samples were selected for various reasons. Rob Heywood participated in the selection process for some of the samples. He was concerned about the apparent lack of strength and aggregate composition (round river gravels) of the concrete in the floor slabs and columns, hence the four slab samples.

20. I was able to get photographic records of the:

a) damage to beam ends;

b) smooth finish on precast elements where I expected they would have been roughened;

c) lack of confining reinforcement through beam column joints;

d) lack of reinforcing connecting slab elements to beams;

e) absence of intact beam column joints;

f) very low (and, in some places, very high) concrete cover to reinforcing;

g) delamination of the Hi Bond proprietary steel decking from the floor slabs;

h) sloping of slab, spandrel and beam elements up toward the north core– suggesting disconnection from the core occurred subsequent to the collapse initiation;

i) very light column spiral reinforcing; and

j) lack of overlap of reinforcing bars from precast beams supported at the same column.

21. The police had operational control of the CTV site during the recovery phase. I discussed my concerns with their Site Commander 'MJ', Senior Constable Michael Johnston, and he agreed that samples should be cordoned off with police tape to reduce the likelihood of them being taken to the disposal site before other engineers had had a chance to examine them.

22. On 25 February 2011 I made notes to record the samples taken. [**BUI.MAD249.0329.1** and **WIT.FROST.0001.19**].

23. Specific areas were set up for certain purposes. Police and coroner stations were placed on the empty neighbouring site on Cashel Street close to the northern end of the Building. The samples were placed to the southwest of the Building, partly on the site where a neighbouring building had just been demolished.

GENERAL OBSERVATIONS OF THE BUILDING

24. Despite little of the original structural form being clearly discernable after the collapse, the main horizontal seismic resisting elements were still quite obvious:

a) The strong 'core' structure on the northern side of the Building (the North Core) remained standing.

b) A shear wall on the southern face of the Building (the South Wall) with openings for fire egress doors central in the wall at each upper floor level for access to the fire escape stairs attached to the outer face of the wall.

25. Apart from these two main structural elements, the only other load bearing elements appeared to be 400mm diameter columns at approximately 7.5 metre centres in both directions on all levels plus some rectangular columns from the western face of the building.

26. The South Wall and fire escape stairs attached to it were lying horizontally across, and on top of, the central section of the debris pile. The top end of the South Wall was sloping down relative to the rest of the wall and was partially buried in the debris that sloped up to the North Core. Most of the southern half of the building appeared to have simply collapsed vertically with little horizontal displacement. Some of the slabs and beams from the northern half had moved several metres to the north (relative to their original location) during the collapse process.

27. Photograph **WIT.FROST.0001.26** is taken facing east from the southeast corner of the Building. The South Wall can be seen lying on the debris. Photograph **WIT.FROST.0001.27** is taken from the east side of the Building facing northwest and shows the debris sloping up to the North Core.

28. Most the roof material had been removed from the debris pile by the time I arrived on the site. However, some of the framing and purlins from the southern half of the roof were still visible. It appeared to have been folded in two back over the northern half and was still trapped under the top of the collapsed South Wall section. Photograph **WIT.FROST.0001.28** illustrates this. The arrangement of this debris suggested to me that the central section of the roof collapsed before the South Wall was pulled over on top it.

29. The photograph **WIT.FROST.0001.29** is taken from Madras Street near the southeast corner of the Building just minutes after the collapse, as indicated by the lack of smoke and no fire damage up the North Core. I do not know who took this photograph, but it is helpful to show the location of debris after the collapse. It shows:

   a) The fire escape stairs above the collapsed South Wall;

   b) The top end of the stairs is buried in the roof framing;

   c) Purlins, concrete floor slabs and other debris sloping back up to the North Core;

   d) Some of the 400mm diameter columns;

   e) Precast spandrel panels and edge beams from the east face and eastern end of the south side of the Building; and

   f) Concrete floor slab sections still hanging off the North Core.

30. Most of the roof material east and west of the folded over horizontal South Wall had been removed by the time I arrived at the site on the evening of 23 February 2011.

31. The photograph **WIT.FROST.0001.30** is taken from above St Paul's Trinity- Pacific Presbyterian Church situated on the southeast side of the Madras and Cashel Streets intersection. I sourced this photograph from the internet. I do not know when it was taken. Here, the fire escape stairs are still attached to the South Wall that is bent over horizontally at Level 2. Roof purlins and some pink ceiling insulation can be seen. Also visible are precast concrete spandrel panels and a precast edge beam leaning against the western face of the South Wall.

32. The southern half of the Building appeared to have come down vertically. It appeared the debris to the north-east and north-west of the Building had spilled out a lot more than the southern side.

OBSERVATIONS OF PARTICULAR PARTS OF THE BUILDING

North Core and South Wall

33. The effectiveness of the North Core and South Wall as seismic resisting elements in the Building would have relied on the concrete floor slabs staying intact and attached to those two building elements, and acting as rigid diaphragms to transfer lateral loads to them. However, it was clear from what I observed at the Building site that the concrete floor slabs did not stay intact. No slabs remained connected to the South Wall after the collapse. The upward slope of the floor slabs towards the North Core is a strong indication that separation from the North Core occurred later rather than earlier in the collapse sequence. If the floor slabs had separated from the North Core before they lost support along the central column lines , I believe that we would have found them in a more horizontal orientation, or even sloping down towards the North Core, after the collapse.

34. On inspecting the base of these two main elements I saw a few fine horizontal cracks 0.5 metre to 1 metre above ground level on the North Core, but no evidence of spalling or vertical movement relative to the adjacent ground slab. There was some horizontal cracking and spalling near the base of the South Wall that I thought looked consistent

with the weak axis bending of that wall combined with impact from falling slabs on the inside and falling beams and spandrels at each end. These cracks were widest at the outer face and barely visible on the inside face. I saw no evidence of vertical movement of this wall relative to the adjacent ground slab. The photographs **WIT.FROST.0001.31** and **WIT.FROST.0001.32** show cracking in the east end of the bottom level of the South Wall. Here the cracking is widest at the outer southern face and appears to barely reach the inside face.

North Core

35. Photographs **WIT.FROST.0001.33** and **WIT.FROST.0001.34** are taken looking towards the north-east corner of the north core and shows the base of the North Core. Photographs **WIT.FROST.0001.35** and **WIT.FROST.0001.36** were taken near the northwest corner of the North Core. There is slight cracking along the base of this wall. However, I observed no significant damage to this Building element.

South Wall

36. After the Building collapsed, the bottom of the South Wall up to the (original) first floor level had tilted inwards a few degrees . The higher levels had folded over about its weak axis to just past horizontal and was lying over over the top of the collapsed floors [**WIT.FROST.0001.37**].

37. Photograph **WIT.FROST.0001.38** shows the horizontal South Wall sections. Most South Wall sections were lifted out between 7.00pm on 24 February and 5.00am on 25 February 2011. Photograph **WIT.FROST.0001.39** is taken from a crane suspended man basket located near the east end of the North Core in the early hours of 25 February 2011. In this photograph you can see that most of the South Wall has been removed. The remaining top section of the South Wall is located near the centre of this photo and has the oxygen and acetylene bottles for the gas cutting set sitting on it. You can also see a door opening in this section of the wall and the top of the wall still buried in the debris sloping down from the North Core.

38. Photograph **WIT.FROST.0001.40** also shows the upper section of the South Wall still buried in the debris pile sloping down from the North Core. Much of the fire escape stair structure has been cut free of the South Wall in this shot.

39. Photographs at **WIT.FROST.0001.41** show the base of the South Wall before its removal. I observed there was no sign of diagonal cracking in the 'coupling' sections of the shear wall between the doorway openings. From what I observed it may be that the cracking near the bottom of the outside face was due to the South Wall being pulled over by the collapsing floor slabs and bent about its weak axis.

40. I believe this observation may also demonstrate that the Building collapsed vertically almost immediately before much horizontal shaking had occurred, due to overloading the Building elements designed for gravity loads. I believe it is also possible that little or no horizontal loading was taken by the South Wall and it may be that it did not have an opportunity to perform as it should have.

41. Photograph **WIT.FROST.0001.42** shows the topmost section of the South Wall being removed. It had been too difficult and too dangerous to remove this section earlier because it was partially buried in the rubble that sloped up to the lift core.

<u>Columns</u>

42. Photograph **WIT.FROST.0001.43** shows a typical column of unknown location. I observed the columns were poured in-situ.

43. I also observed that the only real damage to the columns was at the ends. The central length of the columns was largely intact. Photograph **WIT.FROST.0001.44** is of column specimen E33. You can see where a precast spandrel panel had been located adjacent to this column. There appears to be no damage to this column at the level adjacent to the top of the spandrel panel.

<u>Beams</u>

44. Most of the beams in the Building appeared to be precast, 'log' beams for interior grids and shell beams around the perimeter. Shell beams had been infilled during slab pours. As with the columns, the beams were also largely intact, with the only damage typically at the ends of each beam. Most beams I saw had the corners spalled off with very little damage along the length of the beam.

45. The formed surfaces to the inside of the shell beams were very smooth in appearance, not rough. My observation was that this reduced the bonding between the in-situ fill and

the pre-cast. In some photos it appears that the infill concrete may have slipped longitudinally relative to the precast concrete shell [**WIT.FROST.0001.45**].

46. Photographs in **WIT.FROST.0001.46** show one of the interior precast beams near the east face of the Building. The location of a construction joint in the slab pour can be seen from the smooth, formed face over the full depth of the slab concrete that has remained inside the beam stirrups when the slab and beam separated. Also visible in these photographs is the absence of any slab reinforcing passing beneath the top of the longitudinal beam reinforcing. Having some slab reinforcing beneath the top longitudinal beam reinforcing would have reduced the universal tendency I observed for the slab concrete to completely detach from the beam.

47. Photograph **WIT.FROST.0001.47** is of engineering/forensic specimen E18. This is the beam marked B18 in the Department of Housing and Building Report by Hyland Fatigue and Earthquake Engineering (the Hyland Report) located near the southwest corner of the Building. This is a narrow L-shaped precast edge beam with the seat for supporting the ends of the metal decking. Section 5 on S15 [see copy attached **WIT.FROST.0001.48**] shows only 664 mesh being required to be lapping onto the perimeter L-shaped beam at this location. I observed an absence of both mesh and supplementary rebar running from the slab into the beam and lapping with the beam stirrups. I also noted how little damage this beam had sustained as it was torn from the column (C3).

48. Photograph **WIT.FROST.0001.49** shows engineering/forensic specimen E9 (a B16 or B17 beam from the south face of the Building). Here you can see the separation of the beam core from the shell, the lack of steel in the beam core and the bottom longitudinal bars. Two of these longitudinal bars are bent up and 2 stop short of the column location. Since these two L-shaped/bent up bars have to lap with matching bars entering the joint from the beam on the opposite side and pass between the six vertical column bars, there is insufficient room to have more than two bottom bars from each beam entering the joint.

49. Photograph **WIT.FROST.0001.50** shows the end of a typical interior beam. You can see a smooth formed rebate in the end of the beam where the column bars can pass through, the 2 bent up bottom longitudinal beam bars and little damage to the beam save the sides being spalled off and the longitudinal bars spread apart.

50. I have done a sketch [**WIT.FROST.0001.51**] which demonstrates that the two bars from each beam entering the joint have a very short lap length with each other. In my opinion, it is not possible to get adequate anchorage for straight bars with this short lap length. Having the 90 degree bends improves their anchorage in the joint zone. The addition of trimmer bars would also enhance anchorage.

51. Photograph **WIT.FROST.0001.52** is taken looking towards the west of the South Wall. It shows edge beams from Levels 2 and 3 plus slabs from higher levels now visible to the west of the South Wall after the spandrel panels that were leaning against the Wall have been removed.

52. Photograph **WIT.FROST.0001.53** shows a typical interior beam on the western side of the Building before removal. No interior beams were found with the concrete slab still attached. All slab reinforcing was above the beam steel.

53. Photograph **WIT.FROST.0001.54** is another typical interior beam where the concrete floor slab has completely separated from the beam.

54. Photograph **WIT.FROST.0001.55** shows how little cover some of the supplementary, negative moment D12 bars had that linked the slab to an edge beam. Here, they have ripped out of the concrete in the top of the beam.

55. Photograph **WIT.FROST.0001.56** is taken from the northeast corner of the Building looking back towards the north corner. It shows one precast spandrel section from the northwest corner of the Building. You can see that it has ended up several metres north of its original location, which was behind the left-hand side of the North Core structure, and was found leaning over the neighbouring building. In the foreground you can see another spandrel panel, a beam sloping up the eastern side of the North Core and other debris that has spilled around the eastern end of the North Core.

<u>Beam-Column Connections</u>

56. While most beams survived the collapse intact, except for their ends, I saw no intact beam-column connections. The beam-column connection components I observed had fallen apart and had no confining steel to contain the concrete within the joint. Photograph **WIT.FROST.0001.57** is taken looking at the northern end of (level unknown) Beams 22 from Drawings S18 and S22 of the Hyland Report [see attached

copy **WIT.FROST.0001.58**] that would have framed into Column C20 at intersection of Grids A and 4. You can see the concrete still intact behind the bent up bottom longitudinal bars and bent down top longitudinal bars and the impression left where this beam has pulled away from one of the vertical C20 column reinforcing bars. There is no sign of any reinforcing from beam B21 or C20 columns that interlocked or lapped with this beam steel and clearly no column stirrups. There is no other real damage to this beam.

57. The end unreinforced corners of the precast beams appear to have broken off at the joints. I observed there were no stirrups in the beams within the beam-column connection. Spiral reinforcing also stopped at the bottom of the beam level and then recommenced above each slab. I have prepared a sketch **[WIT.FROST.0001.60]** which demonstrates a possible failure mechanism. If the unconfined 'wings' at the ends of the precast beams split off, as I postulate was a very likely scenario if the Building was subjected to very high vertical accelerations during the February earthquake, the concrete in the compression zone between the bottom of the narrow remaining section of beam and the column infill concrete would have been under much greater compression than ever anticipated in the design. This compression is probably high enough to burst out of this unconfined zone. Losing this concrete would also result in the loss of any shear capacity where the beams met the column, leaving the magnified gravity loads from each beam having to be transferred through a very small bearing area onto the cover concrete of the column below. I also refer back to **WIT.FROST.0001.51** for a sketch depicting the bearing area available on this cover concrete, with and without the beam 'wings'; and for comments on the possible intensity of these bearing stresses.

58. Photographs **WIT.FROST.0001.61** and **WIT.FROST.0001.62** are good examples of what one of these joints looked like after the collapse at the junction of Column C19 with Beams B22 & B23 near the northwest corner of the Building. Photograph **WIT.FROST.0001.63** is another good example of the end of a log beam that has lost its 'wings'. You can also see the very smooth formed surface to the remaining curved end face.

Concrete Floor Slabs

59. From my observations, the concrete floor slabs were 200mm thick panels with composite Hi Bond metal decking (metal decking) supported on the top edge of the precast shell beams and precast log beams. The top cover to the mesh in the floor slabs varied from 10mm to often about 140mm **[WIT.FROST.0001.64** and **WIT.FROST.0001.65**] were taken through the ground level door opening in the South Wall and show two of the concrete floor slabs. In **WIT.FROST.0001.64** a transverse wire of the 664 wire mesh can be seen as well as at least one hole where one of the mesh wires running perpendicular to the South Wall has failed in tension and pulled out of the slab concrete. None of the 'H12 @ 600' bars detailed in Section 6 on Drawing S15 of the Hyland Report [see copy attached **WIT.FROST.0001.66]** can be seen in either of these photos. At the North Core there were still several remnants of the 'Drag' bars that were used to anchor the floor slabs to the North Core walls.

60. I did not see any concrete floor slab to which the metal decking was still attached, even in areas without fire damage. This observation raised in my mind the possibility that vertical accelerations in the earthquake 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 loss of bond between the metal decking and the slab concrete, or through tension failure of the metal decking. Since the metal decking is work hardened in the manufacturing process it possesses little ductility, so even a temporary overload pulse in this material would be likely to lead to a brittle failure under fairly small displacements of the concrete floor slab. Conventional reinforcing has quite a long yield plateau and can deform plastically, absorbing energy and thus allowing quite large deflections before failure occurs. Determining the moment demand, and the associated vertical accelerations that would cause those moment demands, that would have led to each of these failure modes occurring is not within my field of expertise. However, I raised them as possibilities that should be explored by engineers more experienced in these structures than me. Both of the slab failure modes postulated here would have resulted in the sudden catastrophic failure of the floor slabs. And either of these failure modes would help to explain the apparent simple, vertical collapse of the southern half of the Building while the northern half of the Building elements that collapsed ended up displaced to the north.

61. Photographs in **WIT.FROST.0001.67** are taken from the Madras Street (east) side of the Building and show the intact lift core plus the top 3 floor slabs leaning against it. De-bonding between the metal decking and slab concrete can also be seen.

62. As discussed above, all interior beams and most exterior beams had completely separated from the concrete floor slabs. In some locations the concrete floor slab reinforcing had pulled away from the perimeter beams, or else there was no reinforcing where the floor slab met the beam.

63. I observed from the arrangement of debris that the concrete floor slabs appeared to have lost support further south in the Building. Floor slabs were found leaning against the North Core and slightly higher against the South Wall. This suggests that collapse of the floor and beam elements started near interior columns before the north and south wall strong elements. This was also supported by the fact that most of the concrete floor slabs in the southern half of the Building appeared to have dropped with very little horizontal displacement. The slabs and beams in the northwest corner ended up several metres north of the original Building line suggesting rupture of the floor plates at a very early stage of the collapse.

64. Photograph **WIT.FROST.0001.68** is taken from the western side of the Building and **WIT.FROST.0001.69** is taken from eastern side of the Building. Both of these photos show how midway between the 2 main structural elements the floor slabs and beams had pancaked down to a thickness that was less than one storey high.

65. Photograph **WIT.FROST.0001.70** shows all 5 floor slabs still in their collapsed location immediately above propping that was installed by USAR Teams searching this void at the eastern side of the South Wall, giving an idea of how thinly the structure pancaked down to.

66. Photograph **WIT.FROST.0001.71** is taken looking towards the western side of the Building. Vehicles that were in the ground level car park are visible, crushed below multiple burnt out slabs. At least 6 vehicles were removed from the internal car park area on the ground floor, Level 1 during the Recovery. All vehicles removed were burnt out. I think it is possible that the fire may have started in these vehicles, providing a fuel source for some time.

67. This Photograph shows evidence of:

   a) Fire damage on the western side of the Building above where the crushed vehicles were found;

   b) Delamination of the Hi Bond metal decking from the concrete slabs. This is typical of what was seen throughout the collapsed structure;

   c) The small space between slabs. Timber blocking inserted between floor slabs in some areas while a digger temporary lifted the edge of the slab to see if any victims were located near the outer edge; and

   d) The smooth outer edge to some of the slabs here indicates the westernmost extent of those slabs (referred to as being at Grid A in the Hyland Report).

68. Photograph **WIT.FROST.0001.72** shows the recovery team attempting to use digger jaws to lift out crumbling, brittle sections of concrete floor slab and the 664mm welded wire mesh slab reinforcing. Two almost intact spandrel panels can be seen in the background still leaning against the western side of the North Core.

69. Photograph **WIT.FROST.0001.73** shows one of the precast spandrel sections being lifted out. The chain slings have almost ripped through all of the panel reinforcing. This photo was taken to give an indication of the difficulty and risks associated with deconstructing elements such as this which are very lightly reinforced.

THE HYLAND REPORT

70. Clark Hyland asked me to review the debris removal sequence of the draft Hyland Report. A copy of the *Removal of Debris* section of the draft Hyland Report was forwarded to me for review in August 2011. I prepared a document setting out the Removal of Debris as I could recall them and forwarded this to Mr Hyland.

CONCLUSION

71. The evidence I found at the Building site indicated to me that there are three very brittle/non-ductile failure mechanism possibilities that could have initiated the collapse or at least contributed to the scale of the collapse and the degree of building disintegration. All three of these failure mechanisms could have been triggered by the

very high vertical accelerations reported for the CBD area during the February 22 earthquake:

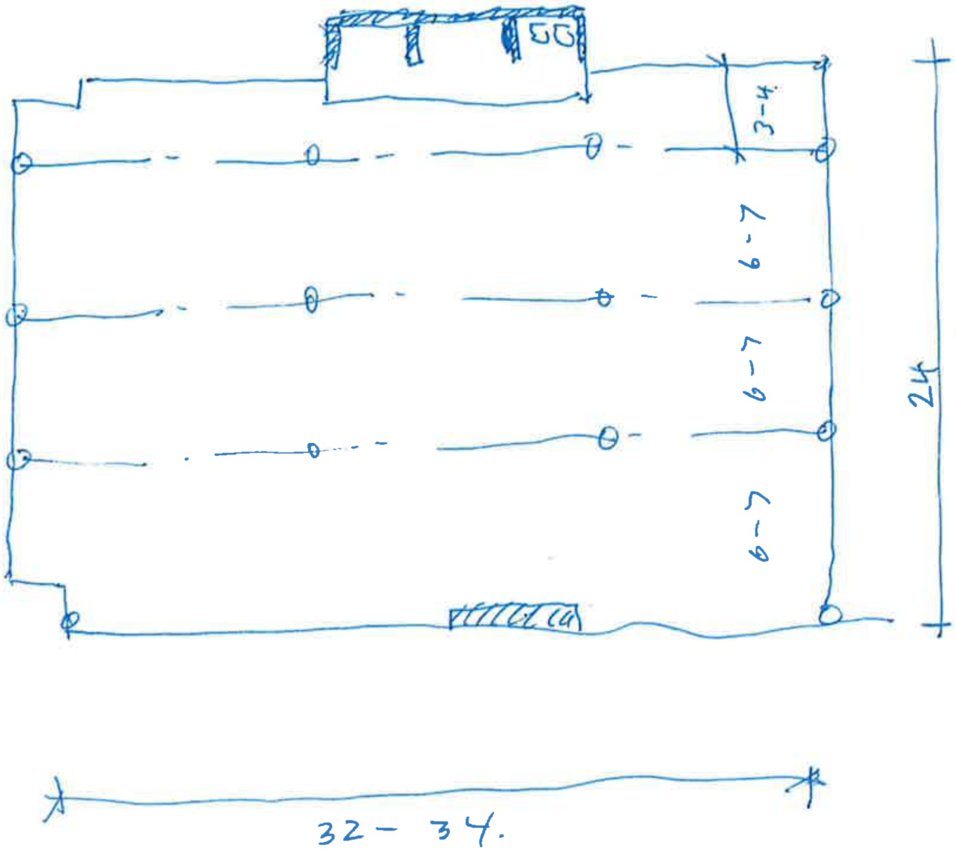
a) Failure of one or more beam-column connection due to lack of confining steel in beam ends and beam column connections, exacerbated by beam compression stresses having to be transferred across smooth formed surfaces that were not perpendicular to the line of action of those forces;

b) Total loss of slab moment capacity associated with bond failure between the metal decking and the 200mm thick composite concrete floor slabs;

c) Tension failure of the metal decking at midspan of the floor slabs.

Signed: G. E. Frost
GRAHAM EDWARD FROST

Date: 19 May 2012

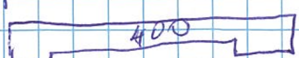

3-4
6-7
6-7
6-7
24
32 - 34.

Fletcher CONSTRUCTION
ENGINEERING
Private Bag 92059, Auckland, New Zealand
Tel +64 9 579 9979 Fax +64 9 525 0302

Calculation Sheet

JOB NAME CTV Bldg		PAGE 1 OF
SECTION		
JOB No.	DESIGNED	G. Frost
DATE Fri 25/02/2011	CHECKED	

Structural Samples (Forensic)

E1 Shear wall SW side Bldg - GL - 1
E2 Ditto FL1 → FL2
E3 Ditto FL2 → FL3
E4 Ditto FL3 → FL4
E5 Ditto FL4 → FL5
E5A Ditto Above FL5
E6 N/S Edge Beam (Short) near NE Cnr
E7 E/W Edge Core → NE Cnr
E8 Ditto
E9 ~~Deep~~ Wide N/S Edge Beam near NE Cnr
E10 Edge Beam @ Core → NE Corner
E11 Short E/W Edge Beam near NE corner
E12 E/W Edge Beam
E13 E/W Edge Beam Core → NW
E14 Ditto
E15 Edge Beam
E16 E/W Edge Beam
E17 E/W " "
E18 Short E/W Edge Beam west of Core
(Black/Brown carpet)
E19 Top Level Column to Roof
E20 760 

760 × 950 × 7m long
E21 Parapet Panel ~~NW~~ West of Core
E22 E/W Edge Beam
E23 EW " " Shear Wall → West
Green Carpet, slab still connected

Fletcher CONSTRUCTION
ENGINEERING
Private Bag 92059, Auckland, New Zealand
Tel +64 9 579 9979 Fax +64 9 525 0302

Calculation Sheet

JOB NAME	CTV		PAGE 2 OF
SECTION			
JOB No.		DESIGNED	GF
DATE	25/02/2011	CHECKED	

E24 Metal Decking / Bondeck

E25 Round column - top + 2nd to top level

E26 Interior E/W Beam ~8m long (broken / missing concrete central)

E27 Tapered E/W Interior beam West end of bldg. 4·5m long overall

E28 Slab Sample 1

E29 " " 2

E30 " " 3

E31 " " 4

E32 Floor Slab - level 3

E33 400ø Column section

Fletcher
CONSTRUCTION
ENGINEERING
Private Bag 92059, Auckland, New Zealand
Tel +64 9 579 9979 Fax +64 9 525 0302

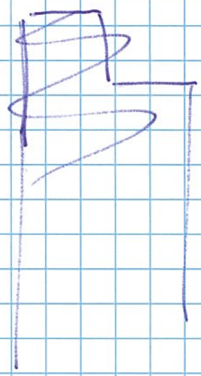
Calculation Sheet

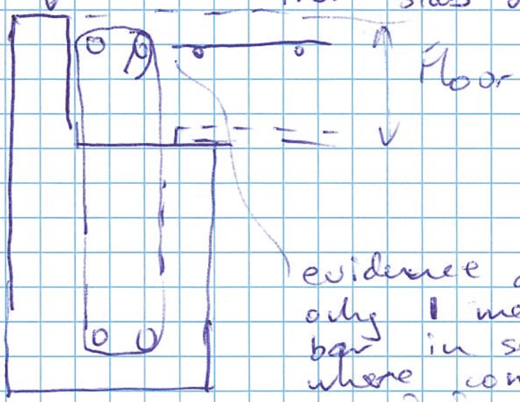
JOB NAME CTU		PAGE 3 OF	
SECTION			
JOB No.	DESIGNED	GF	
DATE 25/02/2011	CHECKED		

E18

Black carpet

No sign of mesh or rebar from slab overlap into beam.

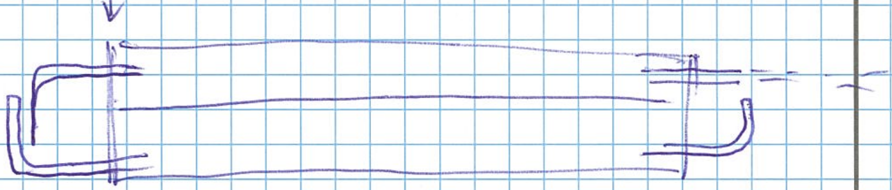


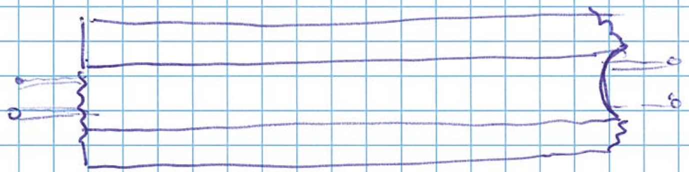


Floor

evidence of only 1 mesh bar in section where concrete remaining extends out from stirrups approx 60 mm (see photo)

sq end to precast L-shape





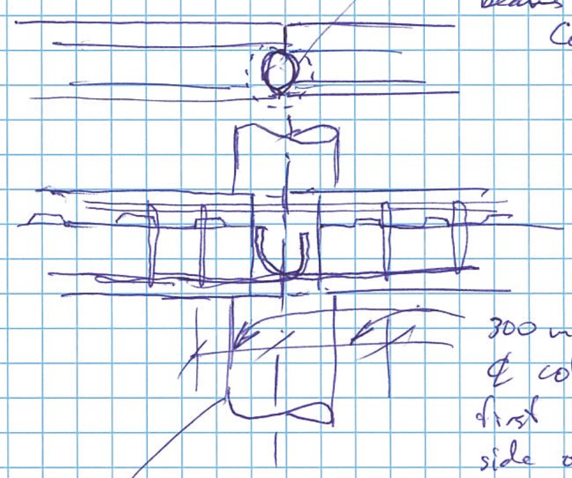
Fletcher
CONSTRUCTION
ENGINEERING
Private Bag 92059, Auckland, New Zealand
Tel +64 9 579 9979 Fax +64 9 525 0302

Calculation Sheet

JOB NAME CTV Bldg		PAGE 1 OF ___	
SECTION			
JOB No.	DESIGNED	G. Frost	
DATE Sun 27/2	CHECKED		

Found first beam - beam - column joint where components still 'linked' near NW cnr of bldg.

~~300~~ 300ϕ recess in beams at junction - Column eccentric to ℄ beam.



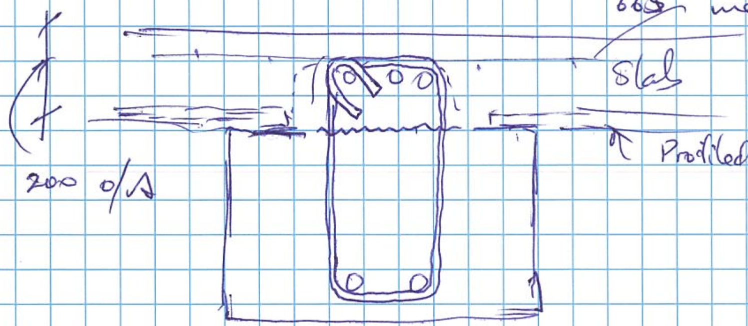
300 mm from ℄ column + first stirrup ea side of joint

400ϕ column

(to allow precast beams to be lowered over column starter bars.)

No column spiral through depth of beams.

No beam stirrups within 300mm of centreline of column.

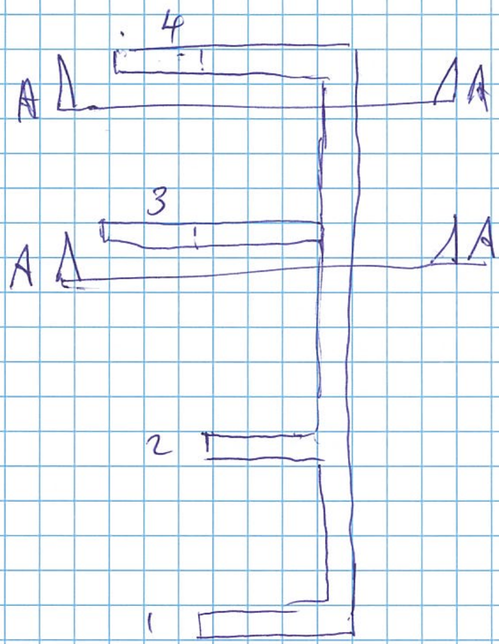


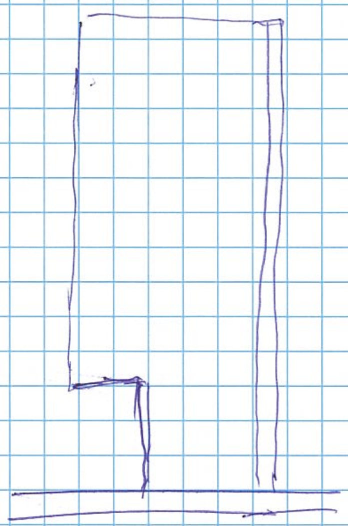
Fletcher
CONSTRUCTION
ENGINEERING
Private Bag 92059, Auckland, New Zealand
Tel +64 9 579 9979 Fax +64 9 525 0302

Calculation Sheet

JOB NAME CTV		PAGE 2 OF	
SECTION			
JOB No.	DESIGNED	CoF	
DATE Sun 27/2	CHECKED		

Still no sign of any slabs still bonded to 'Tray Deck'





A-A

Western most two blade walls have doorway in southern edge a Ground floor level - reduces width of blade to similar to Eastern most two blades.

No sign of damage at base of blade walls 1, 3 + 4 or back wall. other than minor horizontal cracks near base of wall - not enough to spall & plaster forward

My Impression:
Collapse mechanism initiated by either:
1. slab failure (non-ductile). bond to tray deck?
or
2. Column or column/beam joint failure. (also non-ductile)

WIT.FROST.0001.24

Fletcher CONSTRUCTION
ENGINEERING
Private Bag 92059, Auckland, New Zealand
Tel +64 9 579 9979 Fax +64 9 525 0302

Calculation Sheet

JOB NAME	CTU		PAGE 3 OF
SECTION			
JOB No.		DESIGNED	GF
DATE	Sun 27/2	CHECKED	

- 95% of beams are completely separated from column components
- slabs separated from all interior beams.
- slabs separated from most exterior /edge beams.
- no tray deck / bondeck metal found still bonded to slabs.
- all interior columns 400 ϕ.
- exterior columns reported to have been square or rectangular .. I saw ~~no~~ none on site (since arriving on Wed night).
- ~~At SW cnr most slab~~ Right along southern side of bldg slabs and edge beams have collapsed almost vertically i.e. v. little N-S drift.
- At NW corner slabs + edge beams ~~etc~~ found several metres north of original edge of bldg.
- In vicinity of main core( ) slabs were found leaning against core at approx 60° angle (from horizontal).
- Adjacent to shear wall on south ~~f~~ edge of bldg slabs there sloping up towards inside face of shear wall (slightly)
- all strong evidence ~~of~~ slab collapse started near interior columns.

WIT.FROST.0001.25

Fletcher CONSTRUCTION

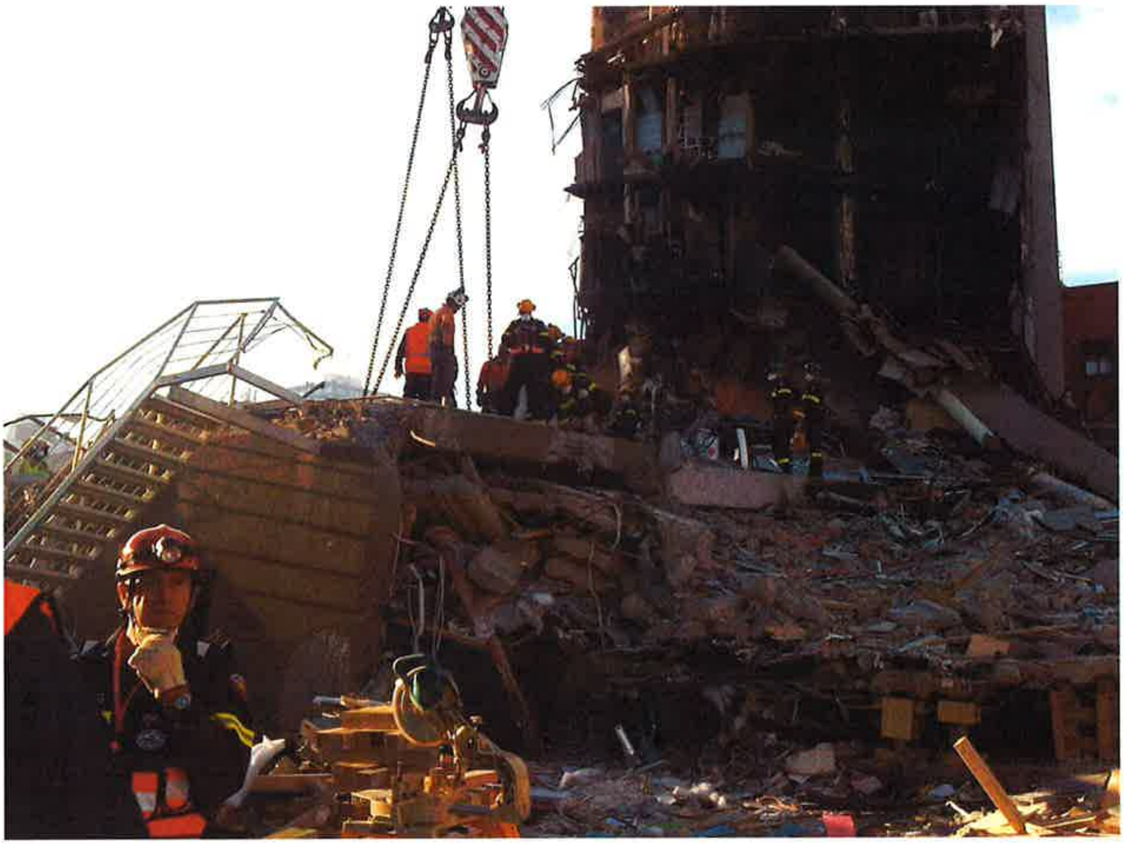
ENGINEERING
Private Bag 92059, Auckland, New Zealand
Tel +64 9 579 9979 Fax +64 9 525 0302

Calculation Sheet

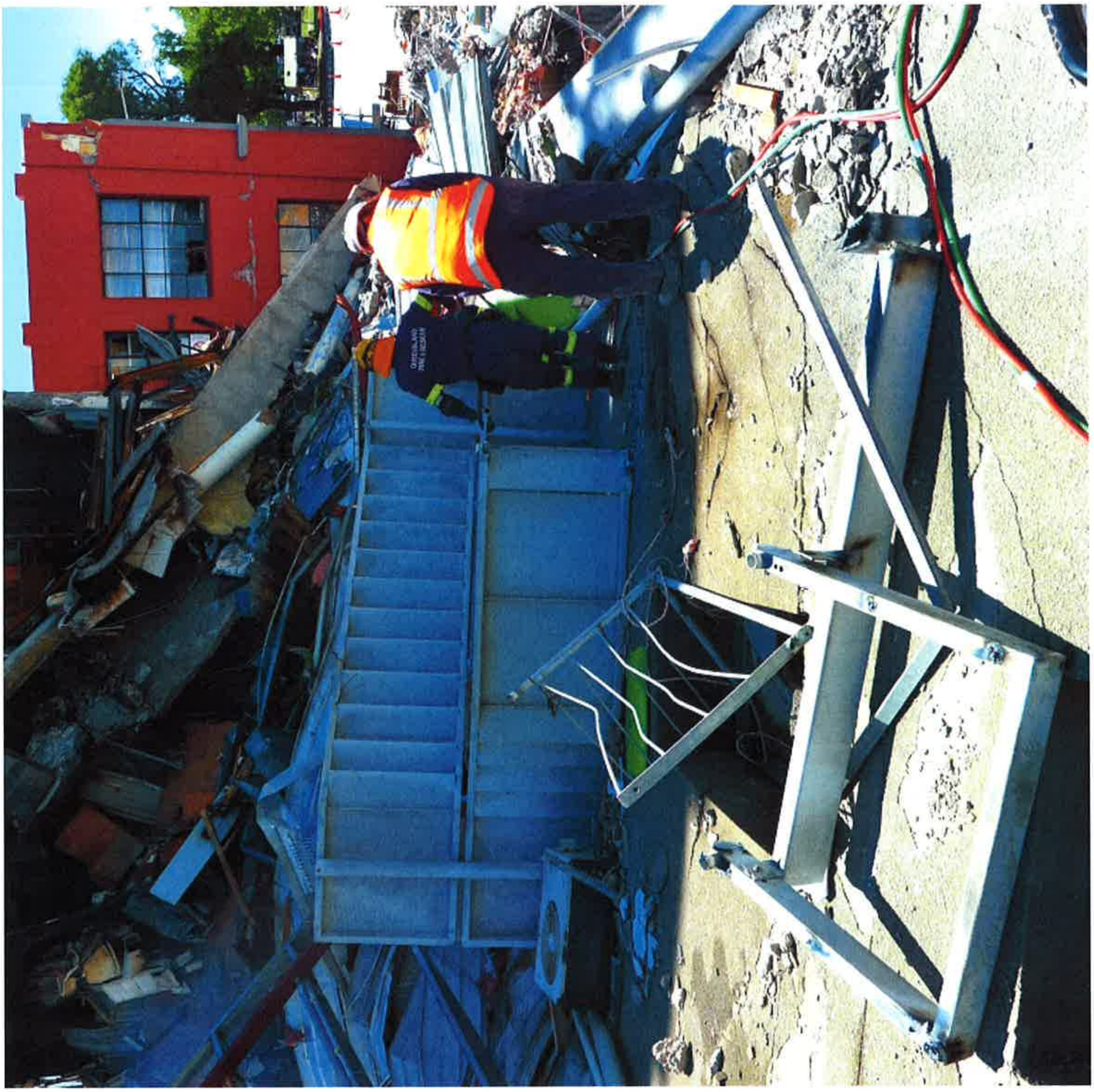
JOB NAME	CTV		PAGE 4 OF
SECTION			
JOB No.		DESIGNED	GF
DATE	Sun 27/2	CHECKED	

- Shear wall remained near vertical over bottom level only.
- ~~Above level~~ Approx 3m above G.L. shear wall was forced folded over top of the collapsed floor slabs
- Over most of building plan area all 5 slabs + beams had pancaked down to a combined thickness of approx 3m.
- Precast shell beams for external beams. expressed outward relative to column / shear wall support.
- No reinforcing connection between shell + infill concrete.
- v. smooth inside (formed) surfaces to shell beams - including the circular formed hole (for column rebar starters) at each end.
- some edge beams have ~~mesh~~ no mesh overlap with precast beam and no added D12 bars added.
- mesh commonly ~~has~~ has up to 150mm top cover.
- over some edge beams added D12 bars (@ 600 crs) have less than 10mm cover.


CTV
your home channel




DORMER

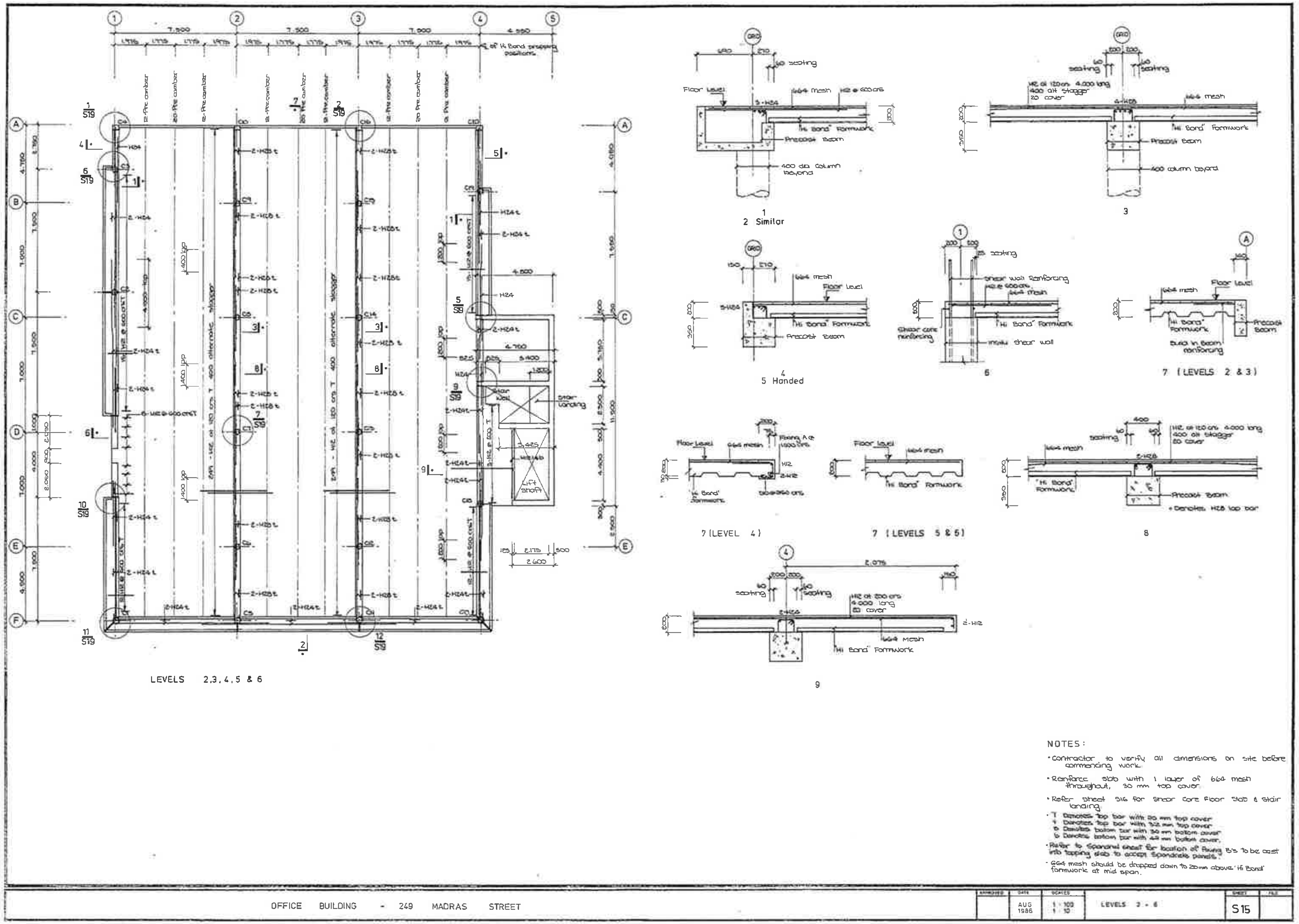



E1
22:00
160

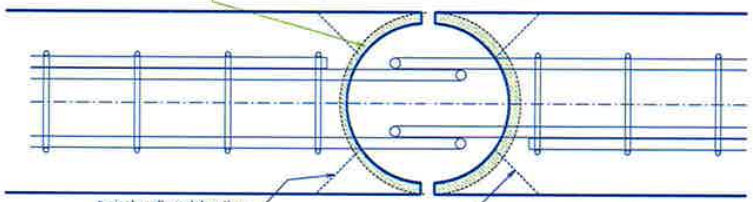

SMITH


E33

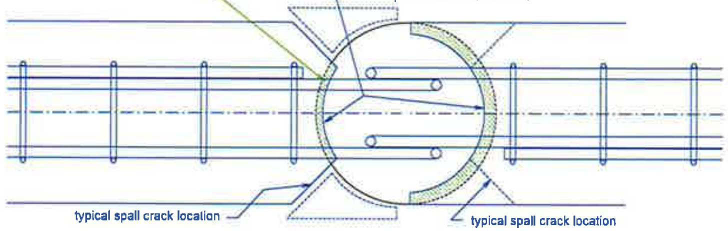

E18


LEVELS 2,3,4,5 & 6
1
2 Similar
3
4
5 Handed
6
7 (LEVELS 2 & 3)
7 (LEVEL 4)
7 (LEVELS 5 & 6)
8
9
NOTES:
• Contractor to verify all dimensions on site before commencing work.
• Reinforce slab with 1 layer of 664 mesh throughout, 30 mm top cover.
• Refer sheet S16 for shear core floor slab & stair landing.
• T Denotes top bar with 30 mm top cover
t Denotes top bar with 32 mm top cover
B Denotes bottom bar with 30 mm bottom cover
b Denotes bottom bar with 44 mm bottom cover.
• Refer to Spandrel sheet for location of fixing B's to be cast into topping slab to accept Spandrels panels.
• 664 mesh should be dropped down to 20mm above 'Hi Bond' formwork at mid span.
OFFICE BUILDING - 249 MADRAS STREET
AUG 1986
1:100
1:10
LEVELS 2 - 6
S15

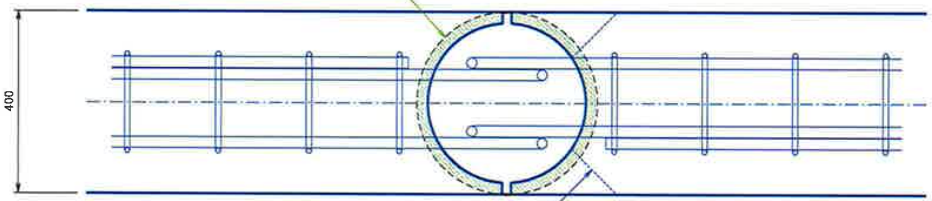

E3



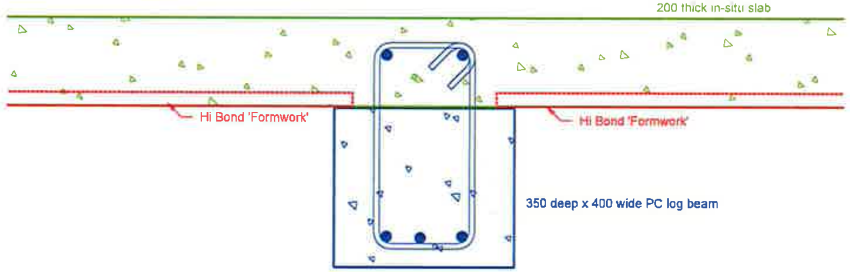
PLAN SECTION 2-2 - INTERIOR BEAM-COLUMN JOINT
(ALLOWING 10mm CONSTRUCTION TOLERANCE)



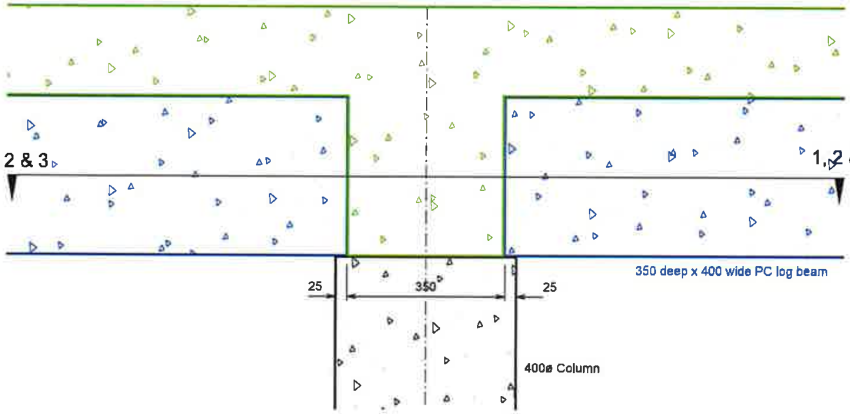
PLAN SECTION 3-3 - INTERIOR BEAM-COLUMN JOINT
(ALLOWING 10mm CONSTRUCTION TOLERANCE + CORNER SPALLING)



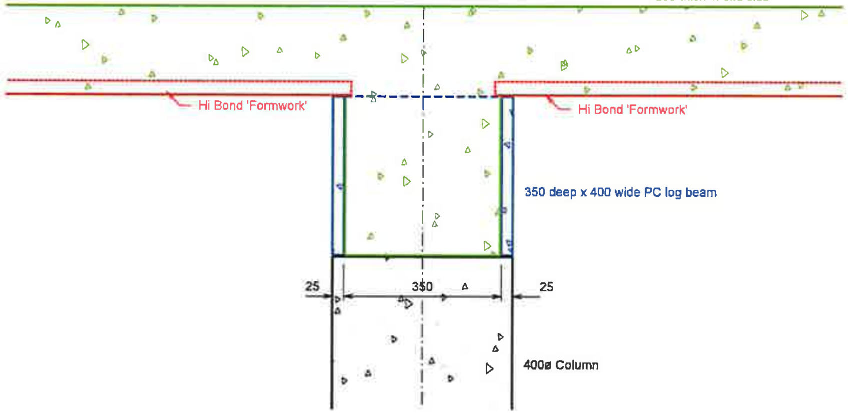
PLAN SECTION 1-1 - INTERIOR BEAM-COLUMN JOINT
(AS DESIGNED)



Typical Section thru PC Log Beam

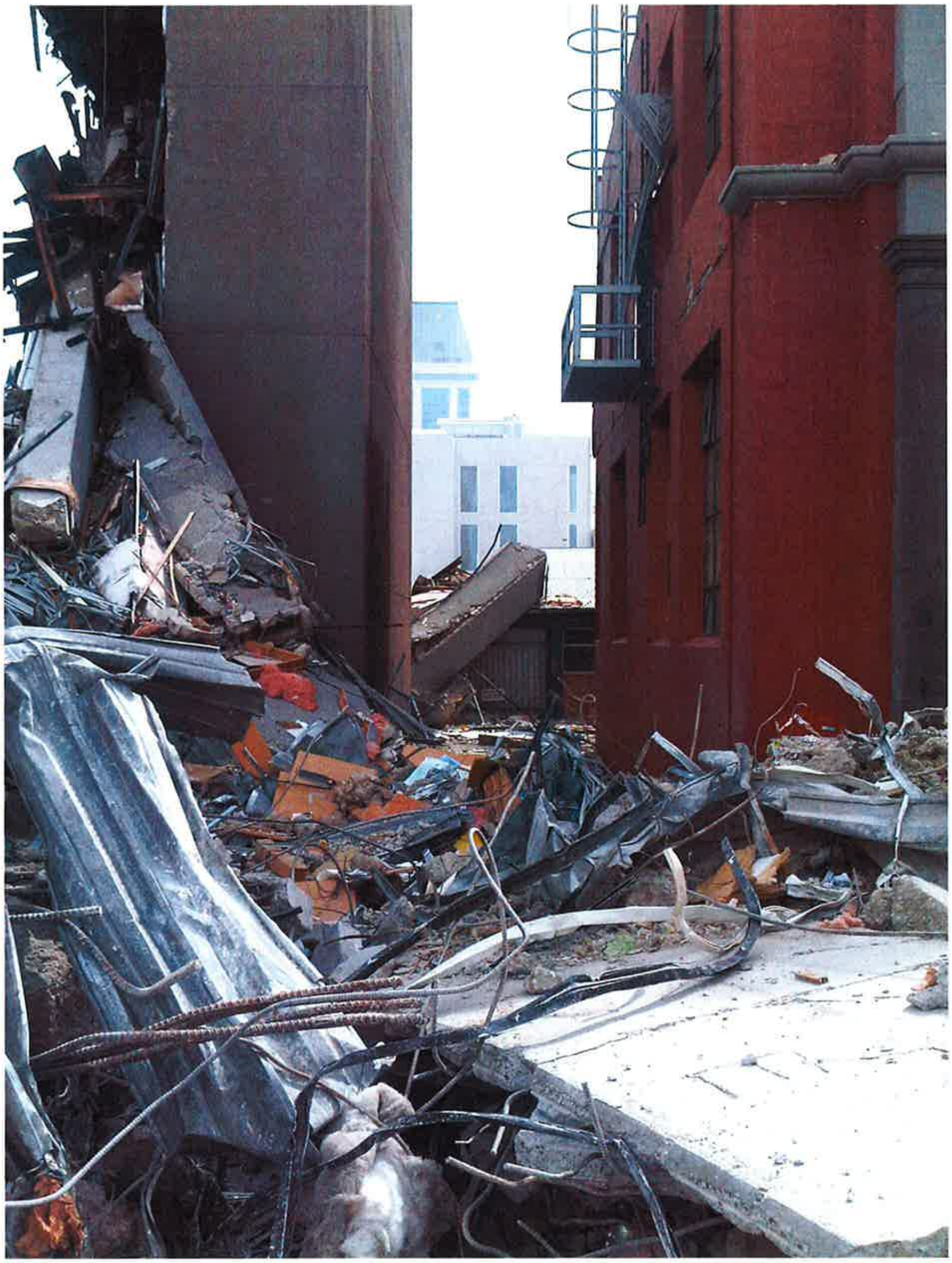


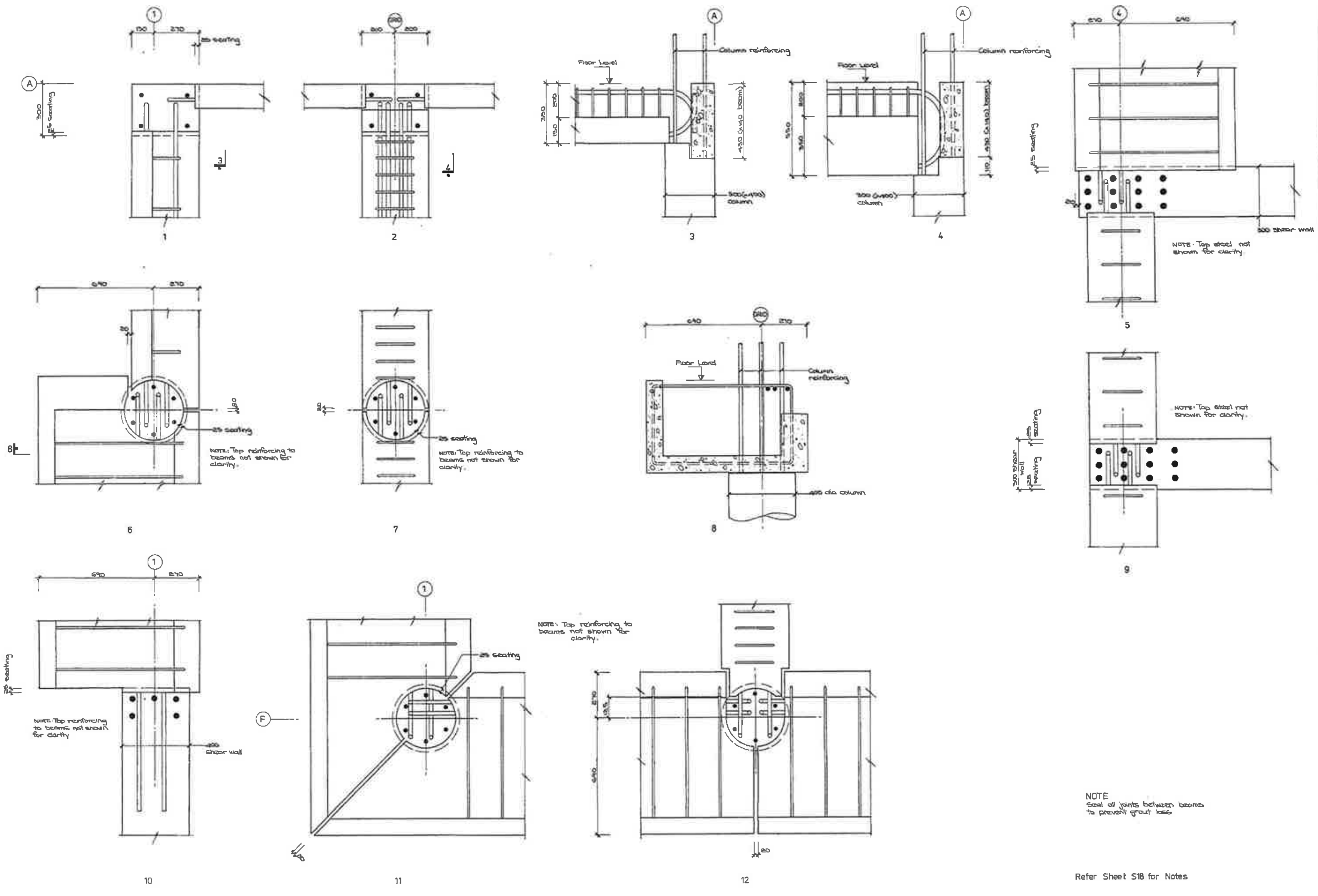
LONGITUDINAL SECTION - TYPICAL INTERIOR BEAM-COLUMN JOINT



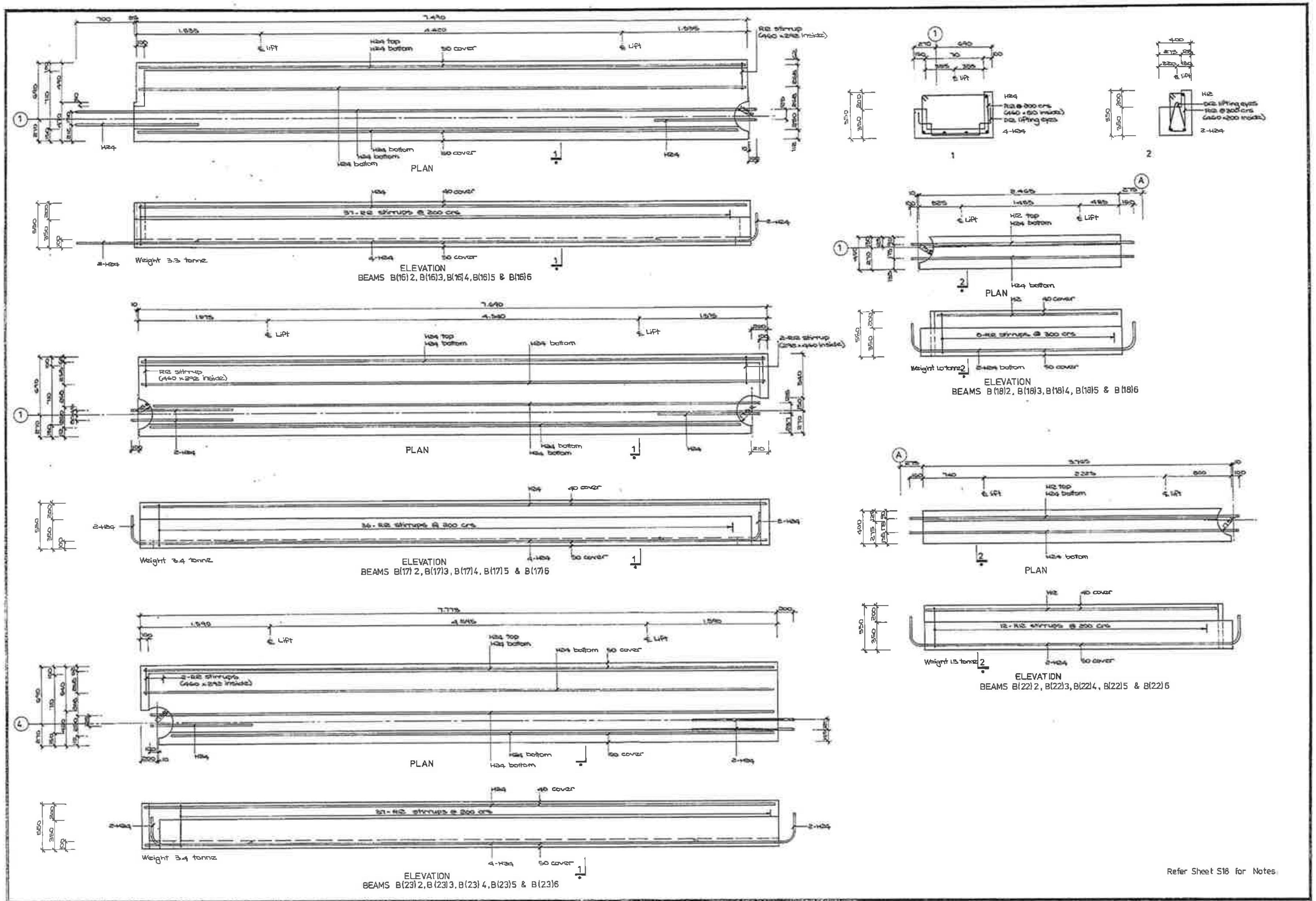
SECTION NEAR COLUMN CENTRELINE

File: CTV Bldg - Interior Beam-Column Joint - GF 20120217.dcd



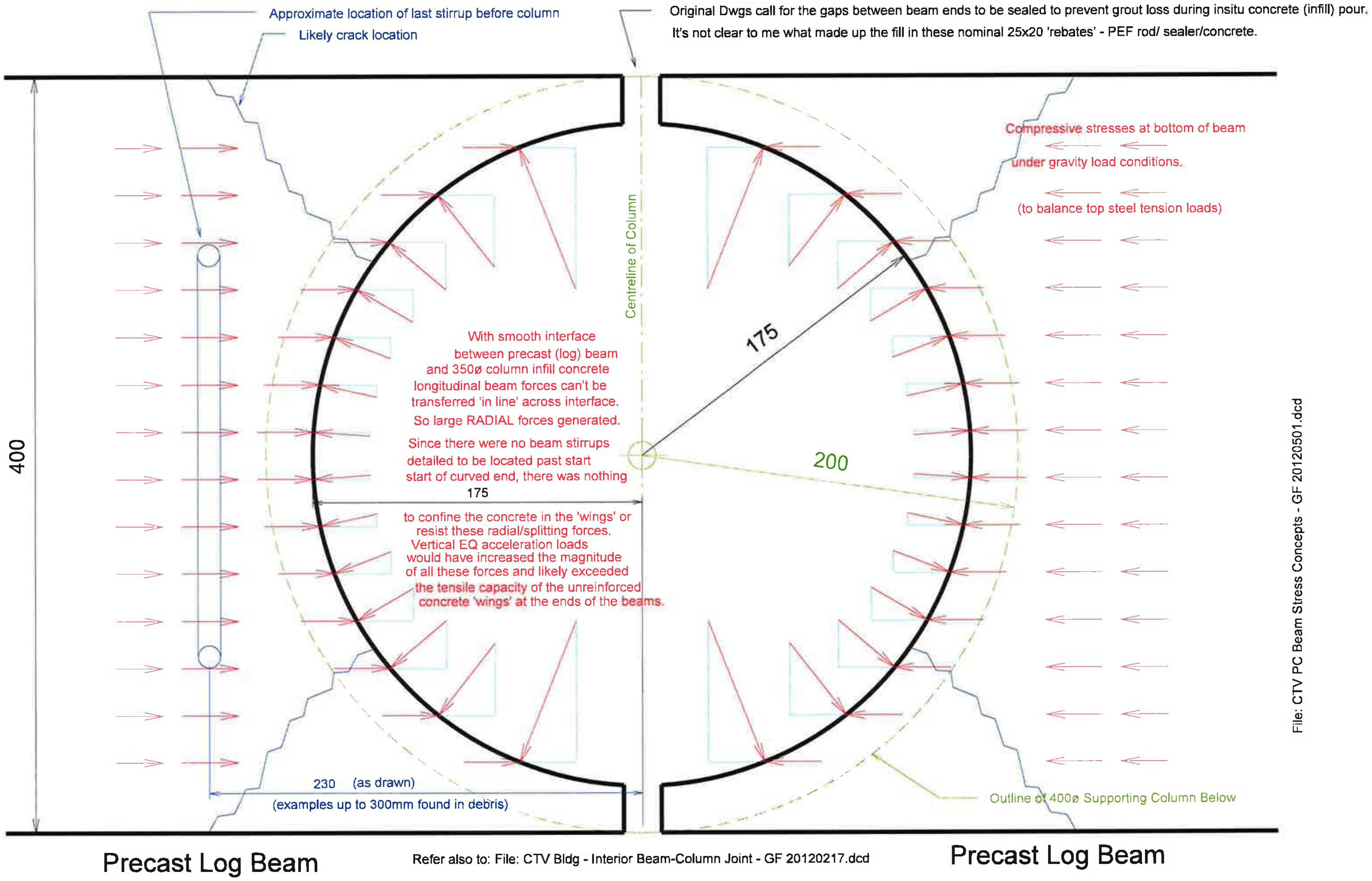


OFFICE BUILDING - 249 MADRAS STREET	APPROVED	DATE AUG 1986	SCALES 1:10	BEAM / COLUMN JOINTS	SHEET S19	FILE



OFFICE BUILDING - 249 MADRAS STREET

APPROVED	DATE	SCALE		SHEET
	AUG 1985	1:20	PRECAST FLOOR BEAMS	S22



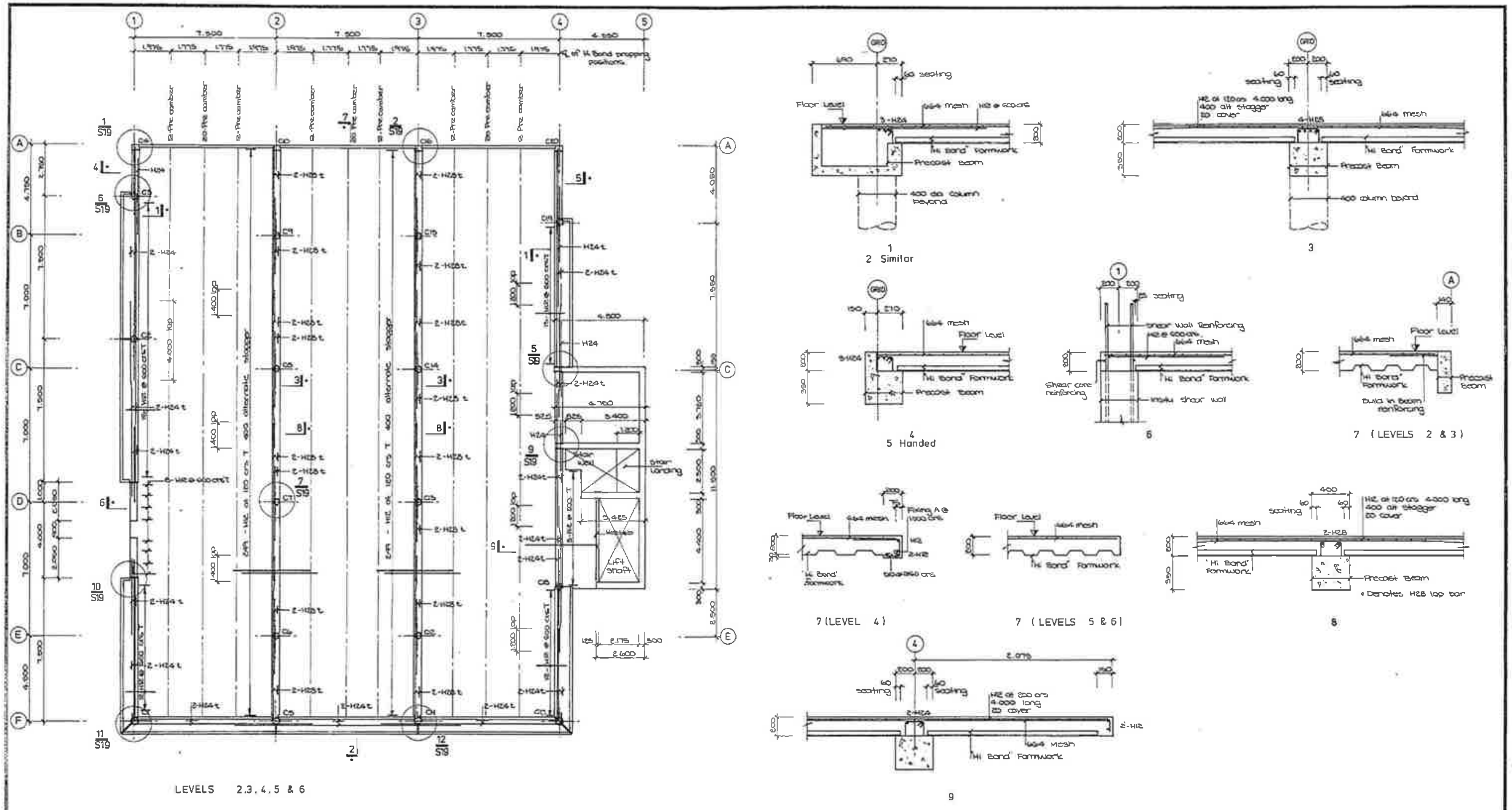
Refer also to: File: CTV Bldg - Interior Beam-Column Joint - GF 20120217.dcd

File: CTV PC Beam Stress Concepts - GF 20120501.dcd

generate large RADIAL forces in
at the interface.

Plan Section at Typical Interior Column
(thru Bottom of Precast Log Beams)

SK GF 20120501



NOTES:

- Contractor to verify all dimensions on site before commencing work.
- Reinforce slab with 1 layer of 664 mesh throughout, 30 mm top cover.
- Refer sheet S16 for shear core floor slab & stair landing.
- T Denotes top bar with 30 mm top cover
  Denotes top bar with 52 mm top cover
  b Denotes bottom bar with 30 mm bottom cover
  b Denotes bottom bar with 42 mm bottom cover.
- Refer to Spandrel sheet for location of fixing B's to be cast into topping slab to accept Spandrels panels.
- 664 mesh should be dropped down to 20mm above 'Hi Bond' formwork at mid span.

OFFICE BUILDING - 249 MADRAS STREET	APPROVED	DATE: AUG 1986	SCALES: 1 : 100, 1 : 10	LEVELS 2 - 6	SHEET: S15	FILE


FREWS