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**
AND IN THE MATTER OF	**THE CTV BUILDING COLLAPSE**

STATEMENT OF EVIDENCE OF WILLIAM HOLMES
IN RELATION TO THE CTV BUILDING

DATE OF HEARING: COMMENCING 25 JUNE 2012

STATEMENT OF EVIDENCE OF WILLIAM HOLMES IN RESPECT OF THE CTV BUILDING

1. My full name is William Thomas Holmes. I reside in California. I am a Senior Consultant with Rutherford & Chekene, Consulting Engineers, San Francisco.

2. In 2011 I was appointed by the Canterbury Earthquakes Royal Commission to peer review the reports prepared for the Department of Building and Housing into the collapse of the CTV Building and three other buildings.

3. I have been asked to give evidence of an assessment that I carried out for the Royal Commission to calculate if the penetration made in the Level 2 diaphragm in 2000 could have had any effect on the seismic performance of the CTV Building. My review of this matter is annexed to this statement of evidence.

4. The calculations attached were prepared by William McVitty, a Royal Commission Staff Engineer, under my direct supervision, due to my unfamiliarity with New Zealand Codes and Standards. I have converted several of the calculation results into English units, with which I am most familiar, and the values are reasonable.

Signed: William T. Holmes

Dated: 19-7-12

Review of Structural Alterations in 2000 to Add New Stair from Level 1 to Level 2 by William T. Holmes

1. The Royal Commission has asked me to review the penetration cut in the Level 2 floor diaphragm at the south end of the CTV Building **(the Building)** in 2000.

2. In preparing this review have referred to the drawing submitted to the Christchurch City Council for this work by Falloon and Wilson **[BUI.MAD249.0151B.2]**. I have also referred to the original structural drawings and calculations for the Building.

3. Prior to carrying out calculations I considered that there were two issues related to this alteration that could influence the seismic performance of the Building:

   a. Did the apparent reduction in shear capacity in the Level 2 diaphragm as a result of the stair penetration affect the seismic performance of the building in any significant way?

   b. Did the drilled holes in the shear wall associated with the connection of the steel beam trimming the penetration on the west side cut any important vertical flange bars in the shear wall and if so, did this have an important influence on the performance of the building?

Reduction of Diaphragm Shear Capacity

4. Seismic shear to the shear wall on line 1 (**the coupled shear wall**) is transferred from the Level 2 floor slab to the wall through direct connection along the wall plus the perimeter beams on line 1 acting as collectors. The total length of this shear transfer is essentially the full width of the building from line A to line F, 30250 mm. The length of slab removed was 3023 mm or about 10% of the total length. I note that 10% is the current limit for alterations without analysis in the current US code-IBC para 3403.2.3.2.

5. Based on the length of the slab, it is most likely that 5-H12 starter bars (A = 565 sq mm) were removed plus the mesh for the length of the penetration (3023 mm). It could be argued that the 2 connections of trimmer beams to line 1 added a total of 4-M20 bolts (Area = 1256 sq mm) connecting the slab to line 1 as well; these 4 M20 are not included in the calculations.

6. It is my understanding that there is no specific structural alteration trigger in New Zealand to cause overall structural evaluation of the building similar to a change of occupancy, but the building must be shown to still reasonably meet code (presumably the original design code) after the alteration. The code demand for line 1 transfer to the shear wall at level 2 was 470 kN, using a parts and portions co-efficient of 0.15 and 50% of the level 2 floor mass of 6217 kN **[BUI.MAD249.0272.2]**.

7. The original code level capacity of shear transfer from diaphragm to line 1 was 4600 kN, using shear friction along the face of the wall and above the face of beam in other locations along line 1. The code level capacity after the penetration was placed was about 4000 kN, still far above demand.

8. Therefore the level 2 shear connection to the south wall along line 1 was well in compliance with the original design code after the penetration was placed.

9. The ultimate capacity of shear transfer along line 1 before the penetration, taken as 1.5 x the code level, was about 6900 kN, 1250 kN taken directly into the wall, 1950 kN being dragged in from the east, and 3660 kN being dragged in from the west. However, the edge beams on line 1, acting as drag beams, were connected to the south wall by 4-H24 bars with 700 mm embedment. These bars would limit the ultimate tension connection to 860 kN. The connection was also potentially compromised by having less than the required embedment (700 mm vs. 1250 mm). I note that ASCE 41 recommends simply taking a straight proportion in such cases yielding a connection capacity of 700/1250 * (860) = 480 kN in tension.

10. Therefore the ultimate capacity of the transfer from diaphragm to wall towards the west was 1950 (east beam in compression) + 1250 (direct to wall) + 480 (west beam in tension) = 3680 kN. To the east the ultimate capacity was 480 (east beam in tension) + 1250 (direct to wall) + 3660 (west beam in compression) = 5390 kN.

11. The capacity of the east collector beam to transfer load would be affected by the stair penetration and including loss of mesh and starter bars, would have been limited in shear to 780 kN. In that case, the ultimate capacity of the transfer from diaphragm to wall to the west was 780 (east beam in compression) + 1250 (direct to wall) + 480 (west beam in tension) = 2510kN. To the east the ultimate capacity was 480 (east beam in tension) +1250 (direct to wall) + 3660 (west beam in compression = 5390 kN.

12. The only demand values available for the south wall are contained in the Compusoft report. In Table 26, the Darfield demands are given. The maximum shear transfer demand is 713 kN, well below any of the capacities estimated above **[ENG.COM.0001.59]**.

13. For the Lyttleton (Christchurch) results for all three time histories are given in Table 41 **[ENG.COM.0001.72]**. These average 1380 kN to the west and 1582 kN to the east. These are maxima during the entire record, including after collapse initiated which was not modelled, so are clearly upper bound figures.

14. The upper bound Lyttleton demand to the west of 1380kN is still below the capacity after the penetration was placed of 2510 kN. The upper bound demand to the east of 1581 is also well below the estimated capacity of 5390 kN.

15. The only other possible effect of such a penetration in a diaphragm is the creation of horizontally oriented moments in the remaining diaphragm segments. Trimmer bars, or steel beams are often installed at the perimeter of such holes and extended one or more metres beyond the corners. In my opinion the steel trimmer beams, connected to the slab and to each other at the corners of the penetration, would serve a similar purpose and inhibit the tendency for a diagonal crack to form emanating from the corner.

16. I therefore conclude that the installation of the stair penetration at level 2 had no effect on the transfer of seismic loads to the South Wall.

17. It should be noted that these calculations indicate the limiting transfer load is governed by the tension embedment of the 4-H24 bars connecting the edge beams on line 1 to the wall. This suggests that, at their limit, beams would have pulled away from the wall or, given extraordinary embedment values, would have failed the bars in tension. Based on the post collapse diagrams in Figures 109, 110 and 111 of the South Wall in the Hyland Report **[BUI.MAD249.0189.213-215]** both of these failure mechanisms occurred either prior to collapse or during collapse.

Potential cutting of bars in shear wall

18. There are two sets of important reinforcing bars in the vicinity of the west trimmer beam for the floor penetration. First there are the 4-H24 horizontal bars transferring

collector loads into the shear wall. Secondly, there are 5 –H28 vertical flexural reinforcement bars for the east half of the coupled shear wall, 3 near the end of the wall, and 2 an unspecified spacing away, all tied with R 12 stirrups as shown in detail 9 of drawings S10 of the structural drawings **[BUI.MAD249.0284.11]**.

19. The locations of the drilled holes for the beam connection bolts shown in Details 4 and 6 of the Falloon and Wilson drawing are placed to avoid top and bottom steel of the beams and can be assumed to be unaffected by the drilled holes.

20. The potential effect on the vertical bars is less clear, as shown in Figure 1 (below). The holes for the trimmer beam connection bolts would apparently miss the outer 2-H28 vertical bars. As sketched, the inner bolt will also miss the inner set of 2-H28 and 1-H12 bar. However, the dimension X on the sketch is not specified on the structural drawings and was drawn in Figure 1 to scale as shown on the drawings. Therefore, depending on the exact location of the shear wall reinforcing bars, the holes for installation of the trimmer beam connection bolts could have hit the shear wall vertical bars.

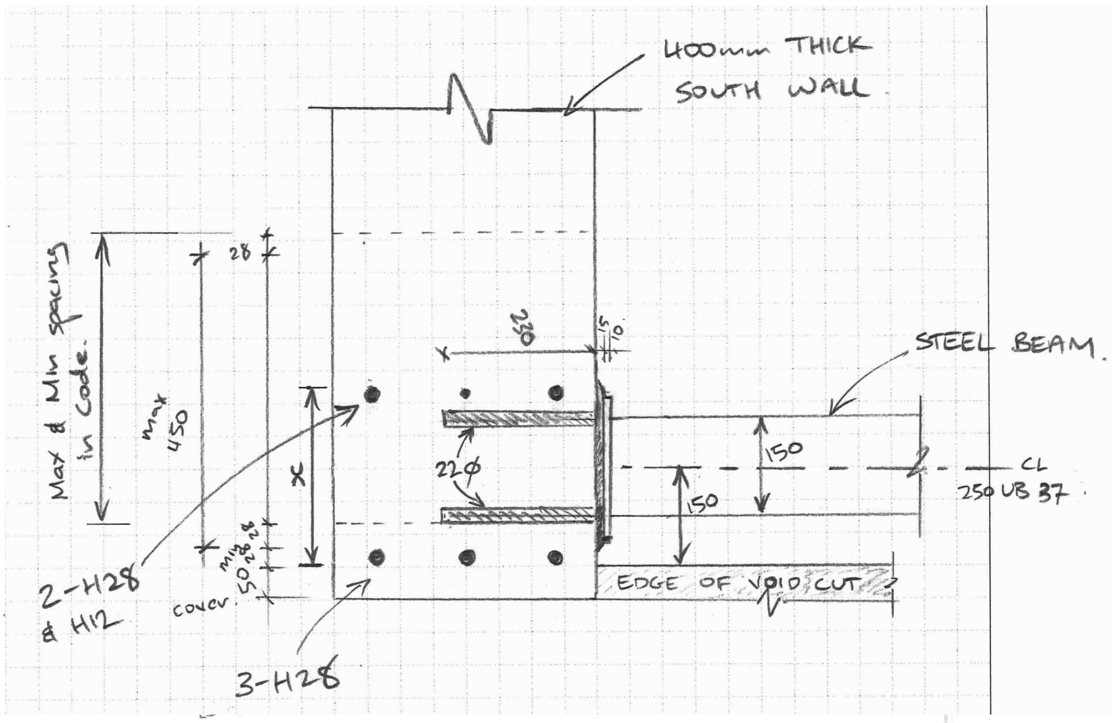


Figure 1 Showing plan of vertical bars in end of south wall and drilled holes from trimmer beam

21. However, sketches of the south wall remnants in appendix C of the Hyland report do not show any indication of heavy cracking or premature yielding of the wall at the location where a cut would have been located (top right of the bottom figure in Figure 109 **[BUI.MAD249.0189.213]**). In fact, the sketches of the south wall indicate that the south wall was not heavily loaded in the east-west direction, either due to failure of one or more levels of drag beams on line 1, or due to early initiation of collapse elsewhere.

22. Figure 13 of the (First) Statement of Evidence of Robert Heywood evidence shows almost the exact location of the connection of the trimmer beam connection to the South Wall and the bars are intact **[WIT.HEYWOOD.0001.42]**. Unfortunately, the inner 2 bars cannot be seen.

23. Most significantly, Figure 9 of the Second Statement of Evidence by Robert Heywood **[WIT.HEYWOOD.0002.13]** clearly shows the western trimmer beam and its connection after the collapse. The trimmer beam had pulled out of the wall. The eastern bolt, nearest the camera in the bottom photograph, has been moved from its original location and installed slightly to the west and below the original position that is identified by the hole in the mounting plate. This indicates to me that the original hole in fact hit reinforcing and the hole was moved and the bolt installed in the alternate position, free of the reinforcing. Such changing of the hole location to miss reinforcing is common practice among knowledgeable contractors.

24. Finally, there is no evidence that the collapse was initiated by premature yielding of the south wall in the east-west direction.

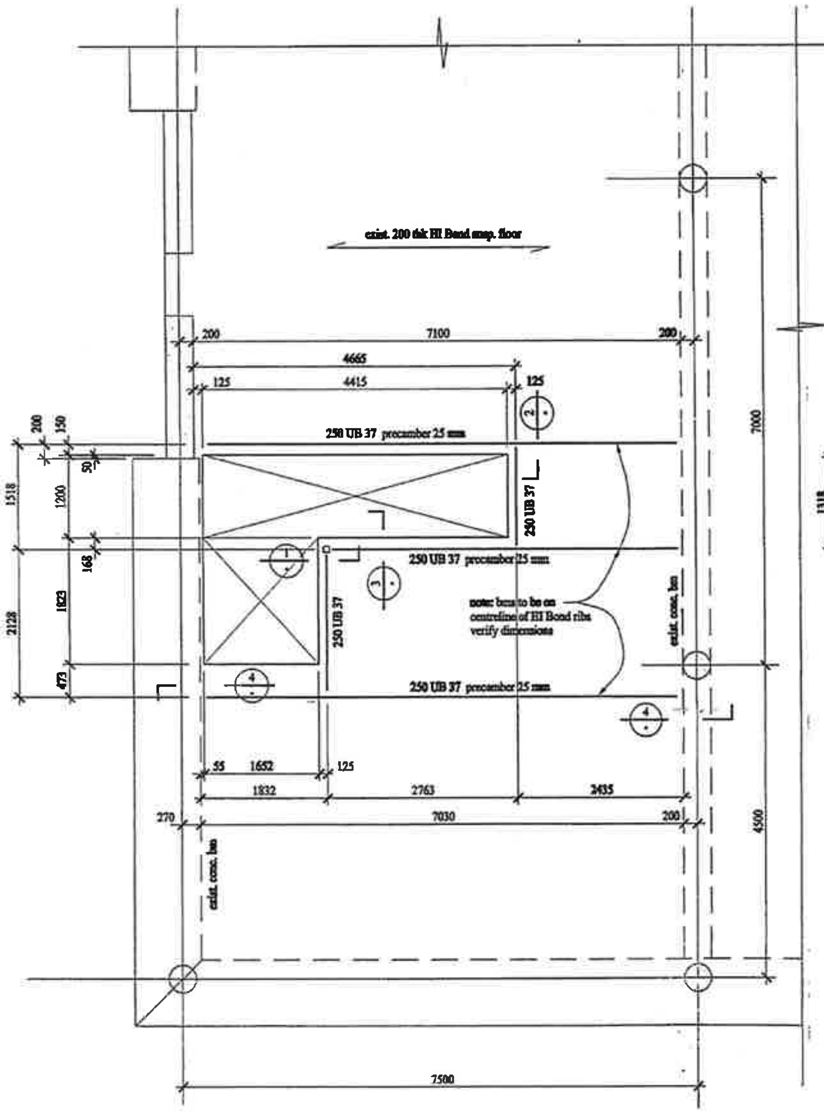
25. I have therefore concluded that the vertical flange reinforcing was not cut by the installation of the western trimmer beam and therefore this installation had no effect on the seismic performance of the building.

26. It should be noted that the connection of a trimmer beam with drilled-in dowels at the end of a major shear wall should have included a note to carefully monitor drilling and avoid cutting of bars.

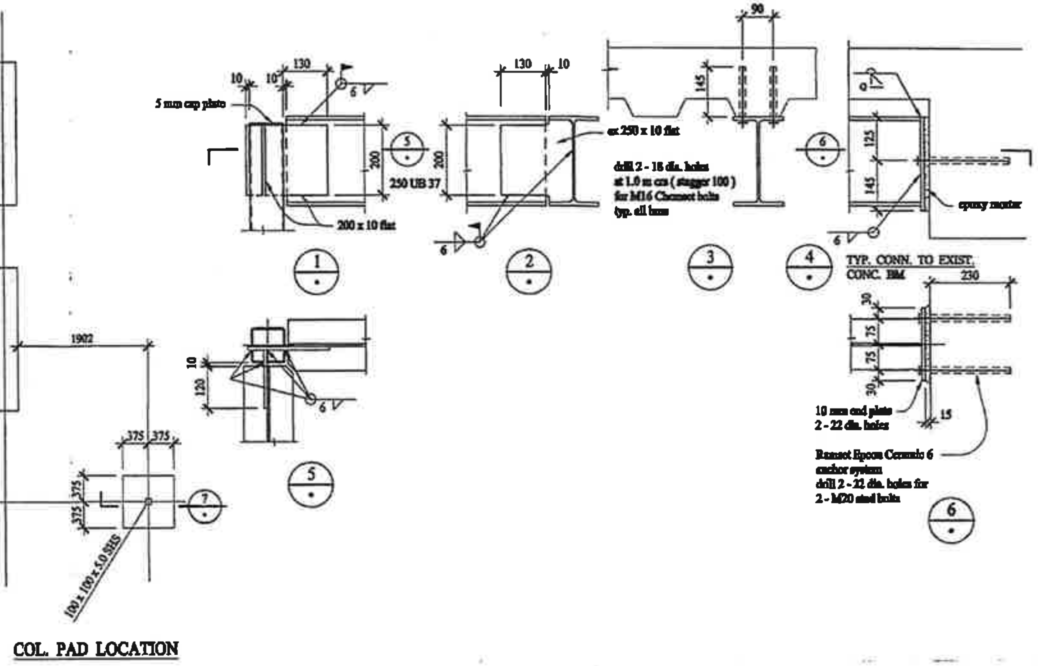
William T. Holmes

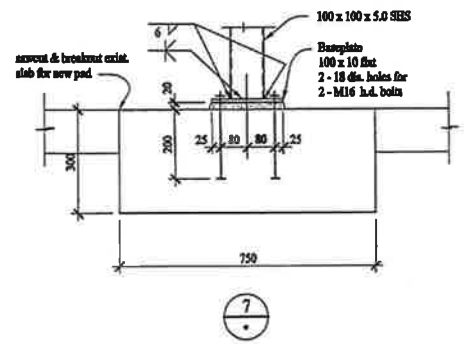
19 July 2012

BUI MAD249.0151B 2



PARTIAL FIRST FLOOR FRAMING PLAN
AT NEW STAIR OPENING





STRUCTURAL NOTES

- Concrete compressive strength (f'c) shall be 25 MPa at 28 days. Materials & workmanship to be in accordance with NZS 3109.
- Steel workmanship to be in accordance with NZS 3404.
- Flats shall comply with AS 3679, grade 250.
- Sections shall comply with AS 3679.1 grade 300.
- Hollow sections shall comply with AS 1163 grade 350.
- Steelwork to be powerwire brushed to remove all loose mill scale & rust prior to the application of zinc chromate priming paint.
- Propping and maintenance of stability of the existing structure shall be the contractors responsibility at all times.
- Refer to Arichitects dwgs for fire protection requirements to bms and post.
- The structural drawing shall be read in conjunction with the architectural drawings.

CHRISTCHURCH CITY COUNCIL
CONSENT DOCUMENT
10 MAY 2000
All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

contractor shall verify all dimensions before commencing work

issue date amendment
drawing status: CONSENT

ALTERATIONS TO BUILDING AT 245 - 249 MADRAS STREET - CHRISTCHURCH

NEW FLR OPNG FOR STAIR
PARTIAL FIRST FLR FRAMING PLAN
DETAILS

FALLOON & WILSON LTD
CIVIL & STRUCTURAL
CONSULTING . ENGINEERS

designed: CWT
drawn: GWL 5/2000
checked:
scale: 1:50, 1:10
file: 2774

sheet S1
rev:

CALCULATIONS

ALAN M. REAY
CONSULTING ENGINEER
CHRISTCHURCH

PAGE S2
SECT
FILE
DATE

Typical Floor

		DL	LL
DL : Floor + S.DL $4.5^{kPa} \times 779$	=	3505	
wide edge beam } = (6.95 − (4.5 × .86))			
+ p/c panels } = $13.1^{kN/m} \times 64^{m}$		838	
narrow int beams = $4.1^{kN/m} \times 75^{m}$	=	308	
columns = $(8^{kN} \times 16) + (8 \times 4)$	=	160	
conc walls [(5 × .4) + (25 × .3)] × 23.5 × 3.24 =		723	
curtain wall $1.0^{kN/m} \times 33^{m}$		33	
LL Floor = $779^{m^2} \times 2.5/3$			650
		5567	650
		6217	

∴ $W_T = (5 \times 6217) + 681 = \underline{31,766}$ kN

Shear Distribution :

H/D of system is approx $\frac{20}{2.5} = 8.0$

∴ distribute 10% at top of bldg & distribute other 90%.

Level	W_x	h_x	$W_x h_x$	$\frac{W_x h_x}{\Sigma W_x h_x}$	F_x	H_x	$F_x h_x$
R	681	20.50	13,960	.044	444	444	9102
5	6217	16.20	100,715	.319	912	1356	14774
4	6217	12.96	80,572	.255	729	2085	9448
3	6217	9.72	60,429	.191	546	2631	5307
2	6217	6.48	40,286	.127	363	2994	2352
1	6217	3.24	20,143	.064	183	3177	593
			316,105	1.000	3177		41,576 kNm

revised p. S9

$V = C_d W_t = 0.100 \times 31766 = 3177$

$0.1 V = 318$

$0.9 V = 2859$

Table 26: South wall diaphragm connection forces - Darfield

Level	East/West Actions			
	Model A		Model B	
	Maximum Westward (kN)	Maximum Eastward (kN)	Maximum Westward (kN)	Maximum Eastward (kN)
Level 6	522	-499	584	-653
Level 5	599	-707	628	-681
Level 4	596	-705	741	-678
Level 3	646	-628	641	-594
Level 2	615	-544	713	-583

9.1.3. Inelastic Wall Demands.

Results have shown that inelastic demand for the cantilever bending of the north core and the south wall only occurs in the lower part of level 1. Table 27 below presents the peak strains that occur during the Darfield CBGS event. Strains listed have been taken from the bottom shell elements at the extremities of each wall, and have been averaged over the height of the shell (998mm for the north core walls, and 1150 mm for the south wall piers). Note that $\varepsilon_y = 0.00219$ for the wall longitudinal reinforcement.

Table 40: Slab 4 C to C/D diaphragm connection forces - Lyttelton.

Level	In-Plane Moments					
	CCCC		CHHC		CBGS	
	Maximum + Moment (kNm)	Maximum -Moment (kNm)	Maximum + Moment (kNm)	Maximum -Moment (kNm)	Maximum + Moment (kNm)	Maximum -Moment (kNm)
Level 6	20656	-12495	15717	-8151	14397	-11083
Level 5	9522	-9433	9143	-5553	8251	-6248
Level 4	19450	-7883	5777	-5948	8732	-6194
Level 3	28094	-5834	4523	-5789	7138	-5077
Level 2	26354	-6134	4152	-5611	7343	-5190

Table 40 presents the diaphragm connection actions at each floor level of the south wall located on grid 1. Results presented are the enveloped maxima recorded over the duration of the time-history record analysed.

Table 41: South wall diaphragm connection forces - Lyttelton.

Level	East/West Actions					
	CCCC		CHHC		CBGS	
	Maximum Westward (kN)	Maximum Eastward (kN)	Maximum Westward (kN)	Maximum Eastward (kN)	Maximum Westward (kN)	Maximum Eastward (kN)
Level 6	970	-929	1017	-1401	1076	-1200
Level 5	1168	-1245	1163	-1153	1092	-1097
Level 4	1083	-1228	1098	-1164	1145	-1331
Level 3	1138	-1063	1328	-1462	1080	-1681
Level 2	1313	-1388	1480	-1315	1347	-2042

10.1.3. Inelastic Column Actions.

Results presented in this section are for the CBGS record only.

Analysis results indicate that the onset of column hinging commences at 2.25 seconds of the run time history record with minor column hinging occurring in up to 15 columns after 4.75 seconds has been run. During this time frame hinges are predominantly located in the eastern perimeter frame (Frame F), with hinge formation initiated in the level 5 columns. Hinge formation progresses to lower levels as the displacement demand on the frame increases. Between 4.75 and 5 seconds an additional 61 columns (61% of the total number of structural

LINE I SOUTH WALL

The Line I South Wall that extended from Level I on the ground to the roof had been broken up into single story components during de-construction.

Level I to 2 (Item E1)

This panel showed flexural cracking patterns typical of cantilever shear walls rather than coupled shear walls (Figure 109). This was likely due to the effect of the Level I doorway having been in-filled with reinforced concrete masonry.

Reinforcing steel taken from the east end of the wall was found to have yielded and elongated prior to the collapse of the building.

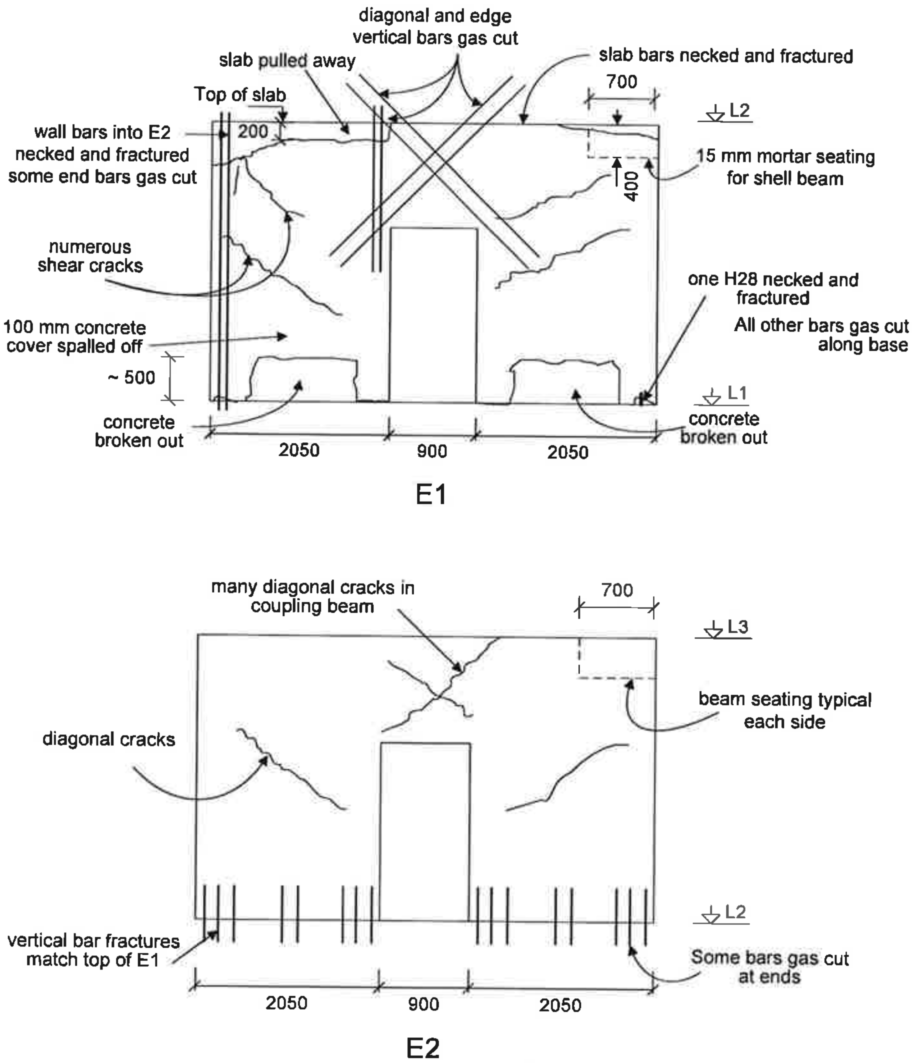


Figure 109 - Line I South Wall remnants (top) E1 Level I to 2; (Bot) E2 Level 2 to 3.

Level 2 to 3 (Item E2)

This panel had diagonal cracking in the piers consistent with cantilever wall behaviour and two way diagonal cracking in the door head coupling beam (Figure 109).

Level 3 to 4 (Item E3)

This panel had dominant uni-directional diagonal cracking running from the bottom west corner to the top east end (Figure 110).

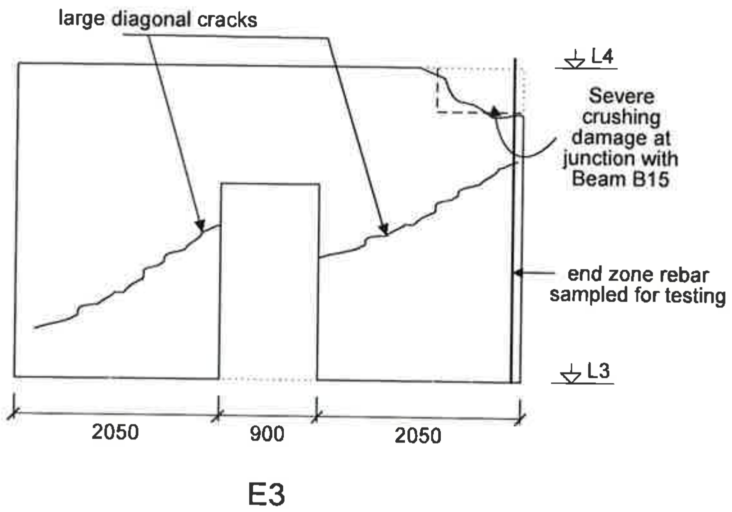


Figure 110 - Line I South Wall remnant E3, Level 3 to 4.

Level 4 to 5 (Item E4)

Severe two-way diagonal cracking in east pier and loss of cover to vertical reinforcing steel on east edge.

Smooth mortar construction joints rather than roughened at junctions with pre-cast shell beams B15 and B16 (Figure 111).

The cracking may have been caused on impact with the ground during the collapse.

Level 5 to 6 (Item E5)

Weak concrete in west pier adjacent to top of doorway that was able to be dislodged by boot (Figure 111).

The top surface of wall was smooth rather than a roughened construction joint for slab seating.

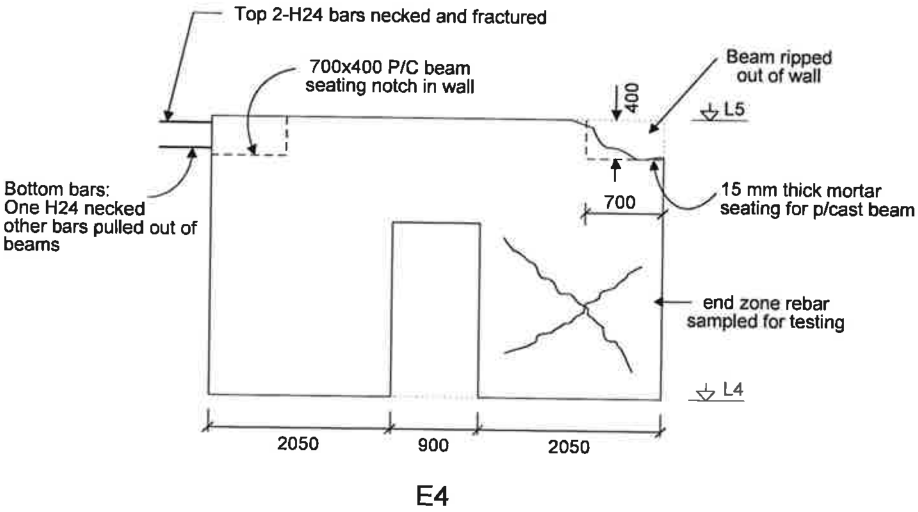
This may have led to slippage on these joints potentially contributing to greater than intended inter-storey drifts.

Bars from wall into attached pre-cast beam had fractured.

No obvious cracking had occurred in the wall or the door head coupling beam.

Level 6 to Roof (Item E5A)

No obvious cracking had occurred in the wall piers or door head coupling beam.



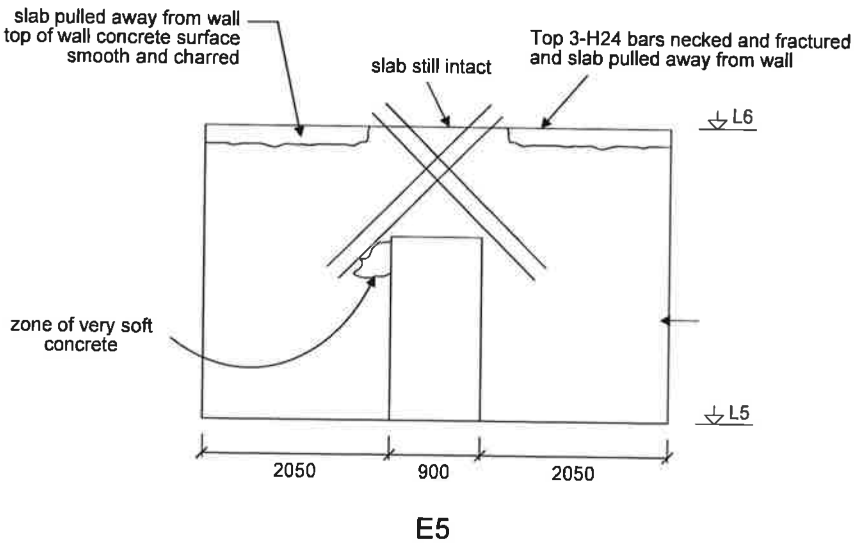
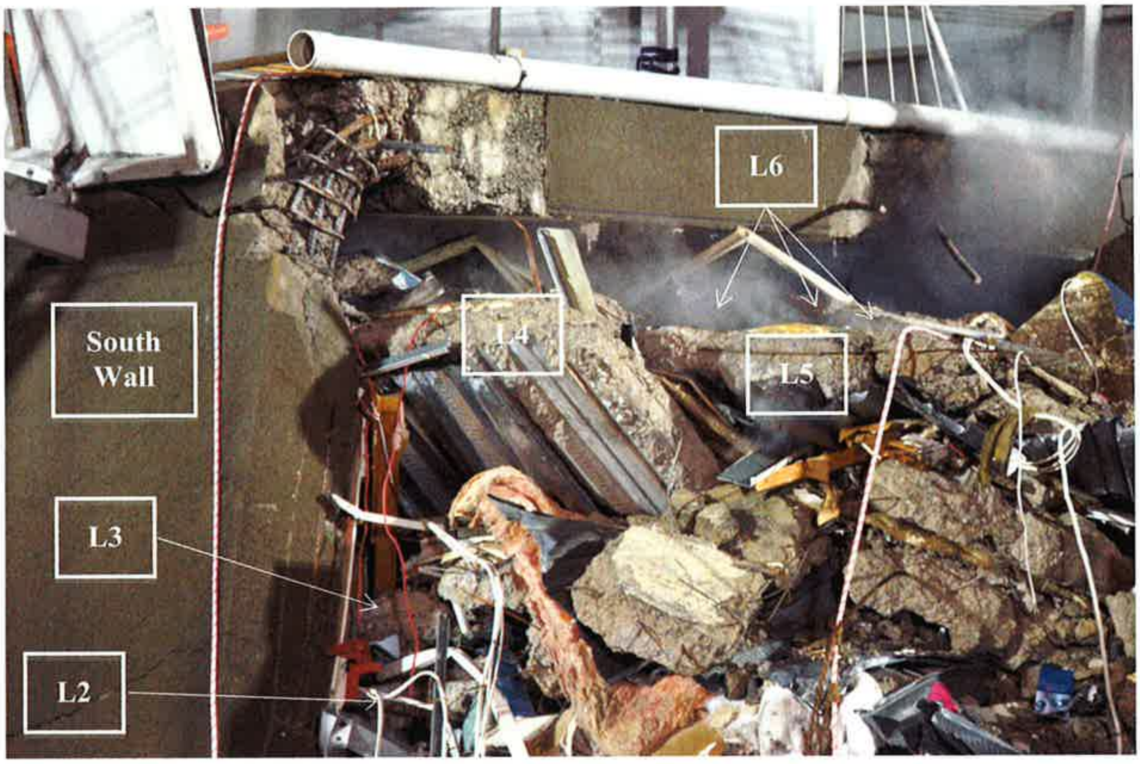


Figure 111 - Line 1 South Wall remnants E4 Level 4 to 5 and E4 level 5 to 6.

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).



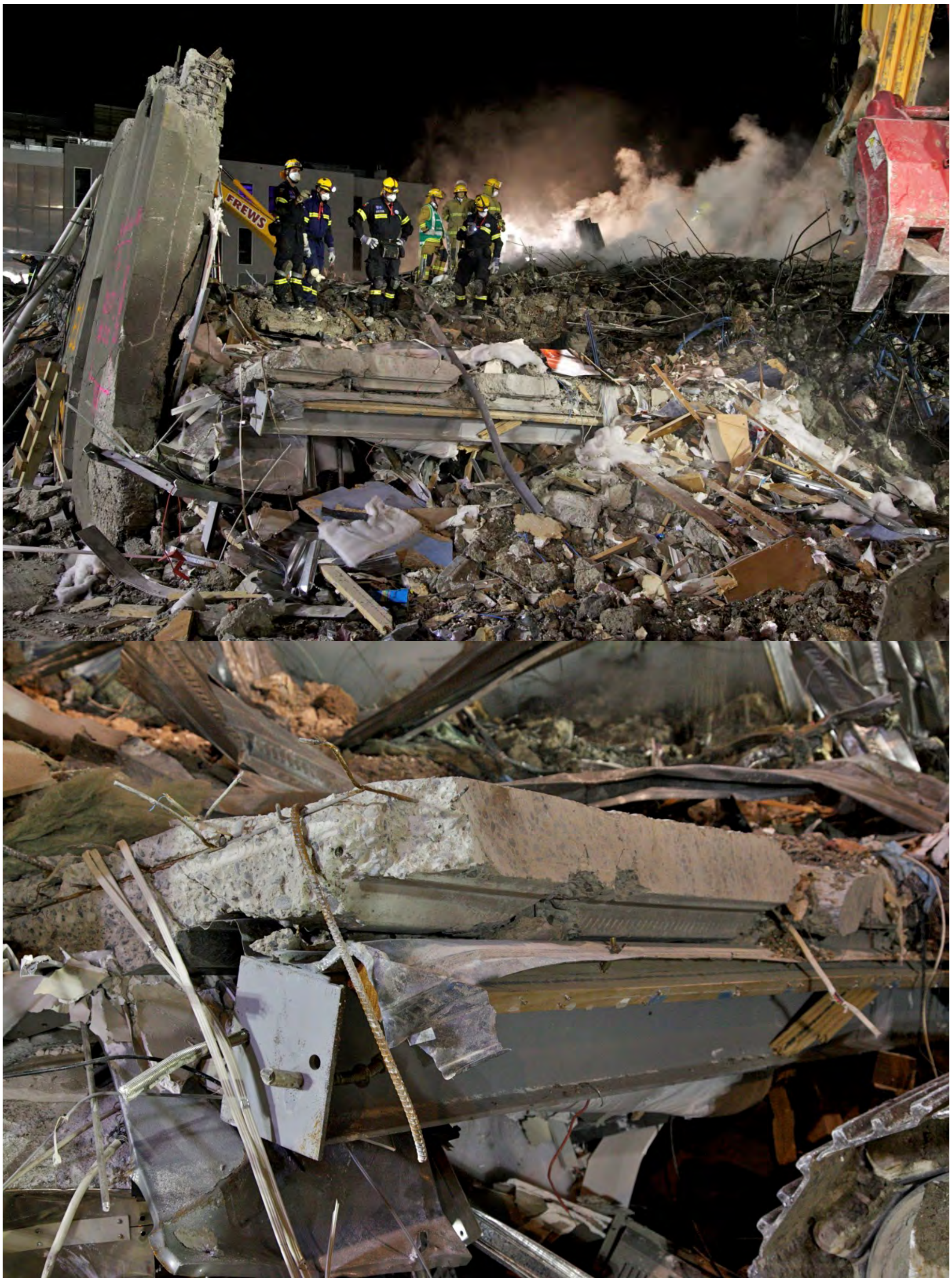


Figure 9 Steel trimming beam: Level 2, south-east corner of the Building