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Collapse of the CTV Building in Christchurch During the Seismic Event on 22 February, 2011

Observations and Opinions by Graham Frost, CPEng - who spent 5 days assisting with the rescue and recovery efforts as a USAR Support Engineer.

While my primary role on this site in the days after the collapse was to assist in recognising and minimising the risks to the USAR and police teams while they performed their rescue, recovery and identification work I noticed several disturbing things about the collapse and felt obligated to record and pass on these observations to other professional engineers, academics and code reviewers in the hope that we can learn from how this building collapsed and reduce the risk of similar collapses in future.

Although the seismic event experienced in Christchurch on 22 February may have had a return period in excess of 1500 years, the number of lives lost in the CTV building may have been much lower if the building had failed in a more ductile fashion. And I'm not talking about ductility in response to lateral loads, but ductility under the demands of short pulse VERTICAL accelerations. I've heard that some area of Christchurch experienced vertical accelerations in excess of 2g.

No drawings of the building were available to the rescue and recovery teams (I understand the Chch City Council building where the records were stored was one of those with no safe access after the February 22 earthquake). And while little of the original structure form was clearly discernable after the collapse, the main horizontal seismic resisting elements were still quite obvious:

- The strong 'core' structure on the northern side of the building which would have been designed to resist all north-south lateral seismic demand, and
- A coupled shear wall running along the southern face of the building, which in tandem with the strong core on the northern face, would have resisted any east-west lateral seismic demands (I say 'coupled' shear wall because there were fire egress doors central in the wall at each floor level).

Unfortunately, both of these lateral seismic load resisting elements relied on the floor slabs staying intact and acting as rigid diaphragms to transfer lateral loads to them – but the slabs did NOT stay intact. On inspecting the base of these two main elements I could see no evidence that they were ever really 'tested' in their primary design mode. There were a few fine horizontal cracks just above ground level, but no evidence of spalling or vertical movement relative to the adjacent ground slab.

The evidence found during the rescue and recovery efforts at the site suggests that the collapse mechanism(/initiation??) included the very brittle/non-ductile failure of the beam-column joints and the total loss of slab moment capacity associated with the bond failure between the profiled steel soffit (which I refer to as 'Tray-dec' although I haven't had time to research which proprietary system was used in this building) and the 200mm thick composite concrete floor slabs which spanned somewhere between 6m and 7.5m. During the recovery effort this author found not a single section of slab to which the tray-dec was still attached – even in areas without fire damage. And while most *beams* survived the collapse intact (except for their ends), no intact beam-column joints were found. Similarly, all interior beams and most exterior beams had completely separated from the concrete floor slabs.

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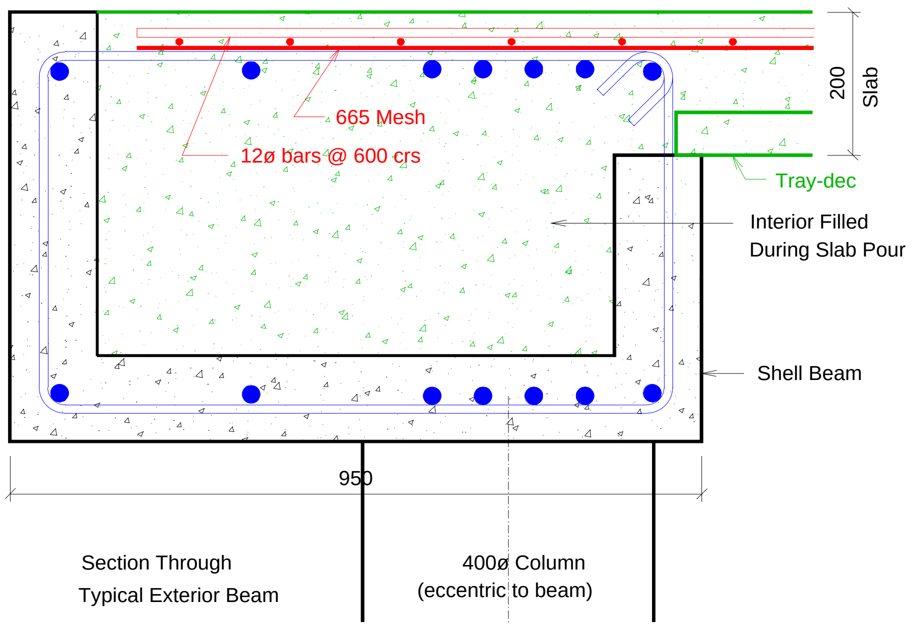
Other important observations include:

1. Apart from the two main structural elements discussed above, the only other load bearing elements appeared to be 400ø columns at approximately 7.5m centres in both directions on all levels
2. The confining steel for these columns appeared to be just an R6 spiral with a pitch of at least 250mm – again, on all levels. (Does this even meet code requirements?)
3. The typical external beams are precast shell beams approx 950 wide by about 600 deep
4. The R12 stirrups to these beams typically stopped 300mm from the column centreline



5. The formed surfaces to the inside of the shell beams are smooth.
6. There was no reinforcing added to the interior of the shell beams.
7. Formed block-outs at the end of the shell beams and interior 'half-beams' are also smooth.
8. When adjacent beams butt together above a column, these semicircular blockouts for a 300ø hole through which the column starter bars pass.
9. With the **TINY** R6 spiral in the columns omitted through the depth of the beam and with the beam stirrups stopping 300mm from column centreline, there is now confining steel through the beam column joint.
10. The sides of the precast beams appear to have 'exploded' away from the column core at the joints.
11. Midway between the two main structural elements the floor slabs and beams had pancaked down to a thickness that was less than the height form ground slab to 1st floor level
12. The floor slabs were found leaning against the main core and slightly higher against the coupled shear wall – suggesting that collapse of the floor and beam elements started near interior columns before the north and south wall strong elements.
13. This was also supported by the fact that most of the slabs in the SW corner of the building appeared to have dropped with very little horizontal displacement. The slabs and beams in

the NW 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.

14. The coupled shear wall remained near vertical up to the (original) first floor level. Above that level it had been folded through 90ø about its weak axis and pulled over the top of the collapsed floors
15. Bottom longitudinal steel to both exterior and interior beams that extended into the joint zone consisted f just two (25ø?) bars bent up though 90ø and lapping approximately 150mm with the two bottom bars from the beam on the opposite side of the column. There were no trimmer bars inside these 90ø bends

Other observations of poor work/detailing on this building (but which I don't believe contributed significantly to the collapse) included:

a. On some edge beams the mesh in the slab did not overlap the precast beam
b. On some edge beams there was no supplementary 12ø negative moment rebar
c. On some edge beams the supplementary 12ø negative moment rebar had less than 10mm cover and had frequently been ripped out of the concrete during the collapse
d. Cover to the mesh in the slab varied from 10mm to (often) about 140mm.

Because there will be other engineers who will want the chance to look at some of the building elements and form their own opinions as to what may have been contributing causes of the collapse – and because we suspected there would likely be an enquiry into the collapse of this buidling, Rob Haywood (Support Engineer for the Queensland USAR team), John Trowsdale(another support engineer with NZ USAR) and I have arranged for approximately 30 samples from the structure to be kept on site (rather than being disposed of off-site with other building rubble). 'MJ' – Site Commander with NZ Police is also aware of the significance of these samples and has agreed that they should be preserved for the same reasons. I have run out of time to provide a tabulated list of what these samples and their approximate location but I have retained my hand written notes and have many photographs of the specimens and the collapse site. Rob Haywood also has details of what most of these specimens are (and his own photos).