CTV Building - GF calculations relating to possible collapse scenarios

Further to Clark Hyland's DBH Report on the CTV Bldg collapse, I think the points below could have received a little more attention in that report - or offer alternative views to some of the report findings.

Point 1 - Weak Beam-Column Joints.
Rough estimate of the gravity load from one floor going into a single interior column is:

$P_g := 7.5m \cdot 7m \cdot \left(0.2m \cdot 25\frac{kN}{m^3} + 0.5kPa\right) = 289 \cdot kN$ or $0.5P_g = 144 \cdot kN$ per beam end.

Post collapse photos show that the ENDS of the precast units were made with a very smooth formed finish. Consider also that with very little inter-storey drift, the vertical joint between a beam end and the column core concrete would, on one side, be opening up below slab level (since the bottom longitudinal beam bars were inadequately anchored beyond that joint) - reducing any available friction on the curved vertical beam end face to zero AND reducing what was initially designed as a very narrow seating width (for the precast beam on the column cover concrete below). If all this single floor dead load is transferred through the 25mm wide (as designed) seating for the precast log beams on the 400ø column, we would have a bearing stress of at least:

$$f_p := \frac{P_g}{\pi \cdot \left[(0.2m)^2 - (0.175m)^2\right]} = 9.8 \cdot MPa$$

If the outer corners of the log beams have spalled off (all the POST-collapse specimens exhibited this situation - and there was no confining steel detailed to prevent this occurrence - see Point 3 below), the available bearing area reduces significantly. Many of the beam ends were left with less than 200mm width at the bottom after the corners had spalled off.
And the bearing stress calculated above assumes columns and beams were built **and located** in PERFECT accordance with the plans. It is quite conceivable that the 25mm bearing width detailed on the drawings could have ended up being much lower. Allowing for just 10mm construction tolerance, support area at one beam end could easily have been as low as $A_{min} := 3510mm^2$. (see CAD File: *CTV Bldg - Interior Beam-Column Joint - GF 20120215.dcd)*

This would have resulted in an unfactored bearing stress in the order of $\frac{0.5P_g}{A_{min}} = 41 \cdot MPa$

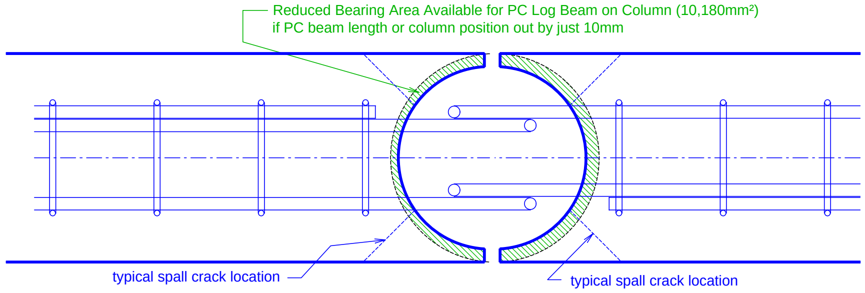
Point 2 - Strain hardening in South Wall Shear Wall.
I also suspect that the strain hardening of bars in the bottom level of the South (shear) wall may have occurred during the final weak axis bending of the wall as it collapsed over the pancaked floor slabs - especially if the bars tested were taken from the outer face.
(In photos IMG00120-20110224-1243.jpg and IMG00121-20110224-1329.jpg - taken before the bottom section of the South shear wall was disturbed - the inclination of the bottom section of fire escape stairs can be seen. The south core wall to which it is still attached in these photos is parallel to the stair structure. In photo IMG00151-20110225-0832.jpg the lean of this wall section is also clearly visible - and the cracks that are so clear in the outer face do not appear to propagate right through to the inside face. While we were preparing upper sections of this wall for removal, I recall commenting that most of the cracks across the coupling beams appeared to have been caused by out-of-plane bending and twisting rather than shear wall action. When we first uncovered the *folded over* shear wall there seemed to be very little diagonal cracking in the coupling beams.

Point 3 - Lack of Confinement at Beams Ends (and in Beam-Column Joints).
End stirrups in precast log and shell beams were often not located close enough to the end of precast - i.e. too far away from column centreline. Some measured up to 300mm! (See photo DSCF 6888). This exacerbates the condition discussed in Point 1 above.

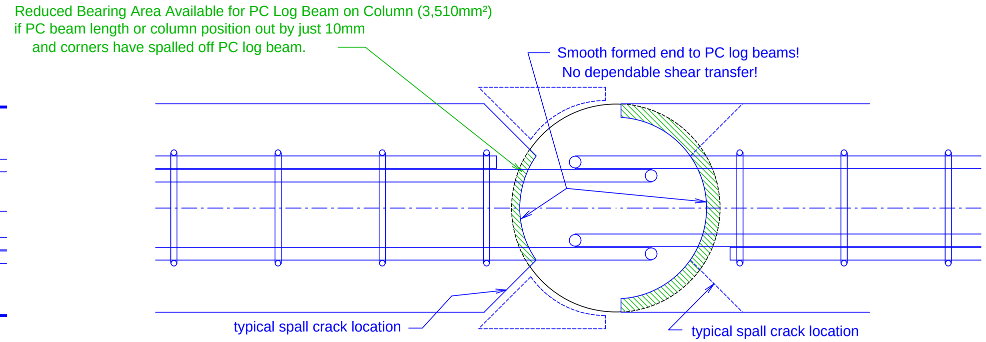
BUI.MAD249.494B.8

CTV Bldg - Interior Beam-Column Joint - GF 201.pdf

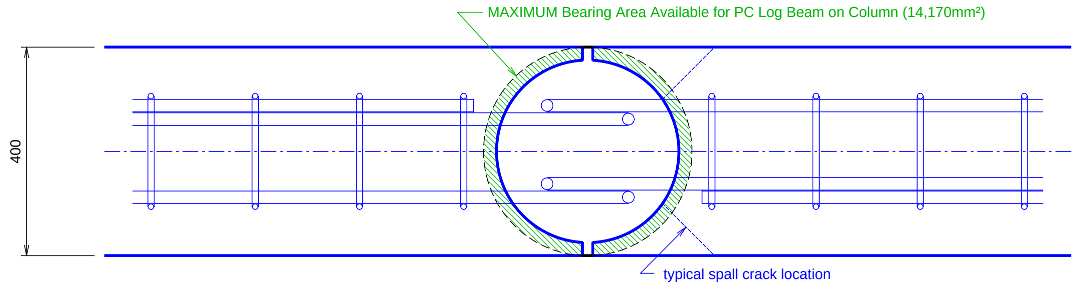
BUI.MAD249.494Bb.pdf



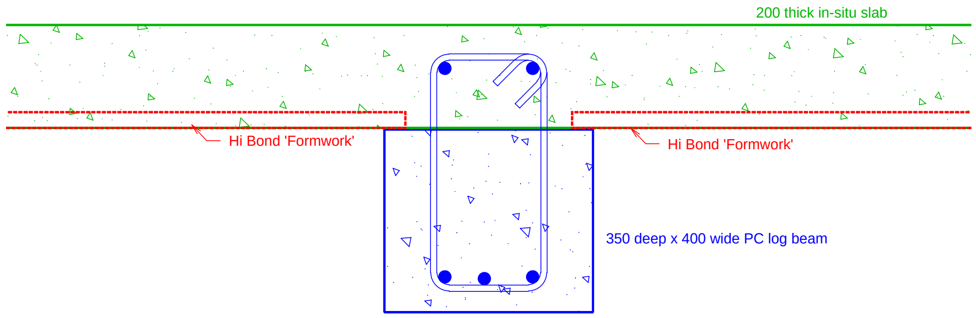
PLAN SECTION 2-2 - INTERIOR BEAM-COLUMN JOINT
(ALLOWING 10mm CONSTRUCTION TOLERANCE)



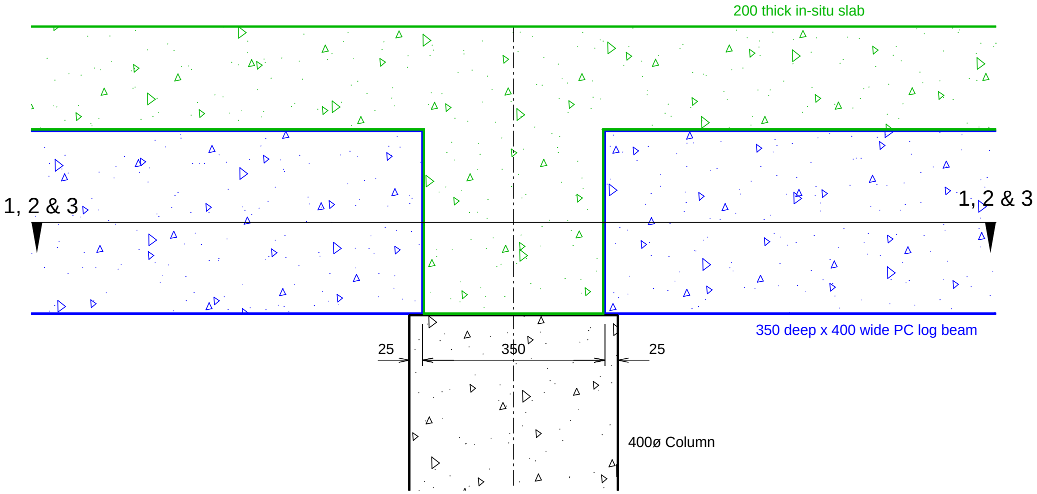
PLAN SECTION 3-3 - INTERIOR BEAM-COLUMN JOINT
(ALLOWING 10mm CONSTRUCTION TOLERANCE + CORNER SPALLING)



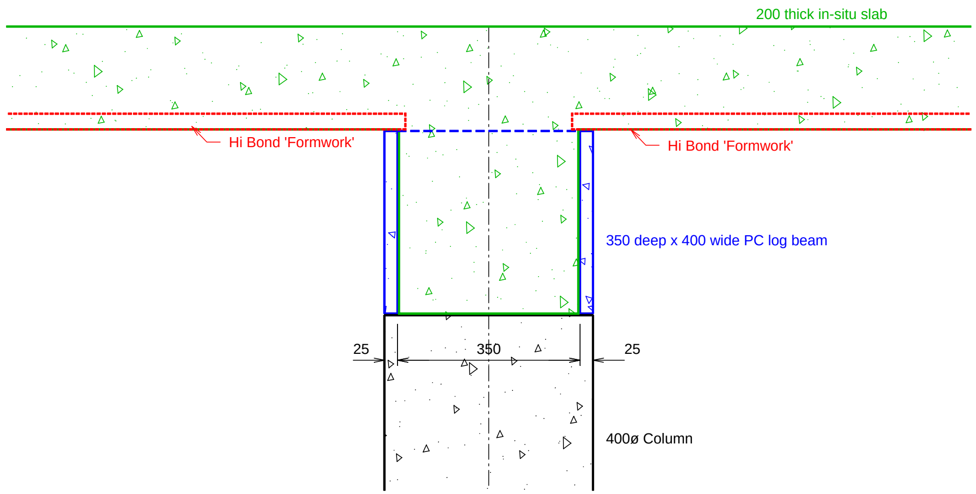
PLAN SECTION 1-1 - INTERIOR BEAM-COLUMN JOINT
(AS DESIGNED)



Typical Section thru PC Log Beam



LONGIITUDINAL SECTION - TYPICAL INTERIOR BEAM-COLUMN JOINT



SECTION NEAR COLUMN CENTRELINE

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