Hyland
FATIGUE+EARTHQUAKE ENGINEERING

Response to Items 6b to 6c

of

Royal Commission Request Concerning

Interpretation of Second Compusoft NTHA

23rd July 2012

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Item 6b.

Do the results of the second Compusoft NTHA change any opinion they have expressed in evidence given or to be given to the Royal Commission in relation to:

i. The response of the CTV Building to the earthquake at 4.35am on 4 September 2010?

No. The calibration of the NTHA to the reported damage remains problematic.

No effects of damage to or effects of internal partitioning on floor response was able to be made on the horizontal or vertical response of the NTHA model.

Both ground motions indicate inelastic behaviour in the line F columns (i.e. yielding of vertical bars and / or concrete compression strains exceeding 0.002).

Figures 35 and 36 in NTHA 2 illustrate the need for caution in interpreting the NTHA results in an absolute rather than a comparative or relative sense.

In Figure 35 the inter-storey drifts in September 2010 would have achieved full contact with the infill masonry on Line A even if the specified gaps had not been compromised by mortar as reported by eyewitnesses.

In Figure 36 the Drifts on Line F in September 2010 would have been sufficient to exceed the column ecu=0.004 failure criteria in the CCCC record. For both the CCCC and CBGS records the columns would have well exceeded their elastic deformation limits ie ec=0.002 concrete and from Level 2 and above the tensile yield limit. Once tensile yield initiated, spalling would have been likely. Obvious and significant levels of spalling would therefore have been expected to be observed from Level 3 and above based on the NTHA 2. No spalling was observed in any of the columns from Level 1 to underside of Level 6 by those who inspected after September 2010. In fact only column F4 at Level 4 was identified as having a crack in it.

The CCCC record indicated disconnection of the drag bars at level 4 only, whereas the CBGS record did not indicate disconnection though level 4 drag bar force on line D/E was up to approximately 95% of the limit force. Movement of the slab relative to the walls would have been required to exceed 2 mm prior to Drag Bar connection failure to account for the bolt hole oversize allowance let alone that required for anchor failure. Such movement would have resulted in significant and observable cracking in linings at Level 4 at the lift doors. However none was reported.

There was inelastic behaviour in some of the two-way beam-column joints in the CCCC record. However, these inelastic beam-column joints did not attain their ultimate strength. However no damage to the beam column joints was reported though it should have been observable.

Drifts on Line 2 compatible with the damage observed after the February Aftershock where estimated by static drift analysis to be less than those necessary to cause net tension in the reinforcing steel of the bottom beam reinforcing steel. This indicates that the beam-column joints may not have been subject to significant internal reversing shear demands and damage in the September Earthquake or prior to collapse in the February Aftershock.

The walls in the South coupled shear wall in NTHA 2 show some inelasticity at the base of the walls though only one fine crack was reported. The drifts shown along Line 1 in Figure

37 exceed those that appear to have occurred in the South Wall prior to its collapse in the 22 February Aftershock estimated to be 0.4 %

ii. The response of the CTV Building to the earthquake at 12.51pm on 22 February 2011?

No. Though the vulnerability of the Level 1 columns to enhanced demand due to two way action has been highlighted if Line A masonry and Spandrel Panel interference is ignored.

The effects of the masonry infill on Line A nor the precast concrete spandrel panels on Lines 1, 4 and F were accounted for in the NTHA 2 analyses. A lower bound cantilever flexural yielding masonry infill panel model was used in the original NTHA. This assumed vertical and horizontal disconnection from the frame of panels 2.39 metres wide. The shear strength and degradation model used assumed significantly lower capacities at 900 kN for the west wall than the NZS 4230:2004 Grade B masonry recommendations and allowing for concrete column shear contribution of 2800 kN.

The stiffness of the Line A masonry infill as observed prior to the February Aftershock including vertical and horizontal contact with frame columns and beams may have been as a panel with effective length up to the of the full depth of the building of 22.5 m and dominated by shear and compression field behaviour until such time as the damage began to degrade its stiffness and strength. The effect of stiffer masonry wall infill in this condition has not been considered in the NTHA. 3 D static analysis and ERSA has shown the effect of a stiffer masonry infill to be significant on torsional response and would also be expected to provide some limitation to north-south drifts on internal columns affecting the 2-way demands on them.

NTHA 2 predicted Drag bar disconnection at all levels however the collapse evidence showed that the Drag Bars only failed after collapse had initiated near the south end of the building. Eyewitness testimony including that of survivors at the Level 4 is in agreement with the collapse evidence. This shows that the NTHA 2 model does not adequately describe in an absolute sense what happened during the collapse on 22 February 2011

iii. Why the CTV Building failed on 22 February 2011?

No. The NTHA 2 shows that there were sufficient drifts for column failure to occur and indicates a number of vulnerabilities were present such as Drag Bar disconnection and beam-column joint fragility. This was also shown by the original NTHA.

iv. The sequence of the failure of the CTV Building to the earthquake at 12.51 pm on 22 February 2011?

No. The sequence of failure presented in the BCR was developed considering convergence of the collapse evidence, eyewitness accounts, 3D ERSA, and static analyses, nonlinear push over analyses, drift compatibility analyses as well as the NTHA. It was recognised that there was difficulty in calibrating the NTHA results to the damage observed in developing the preferred collapse scenario.

Item 6c.

If the results of the second Compusoft NTHA change any opinion they have expressed, in what way has the opinion changed?

No. The NTHA remains useful but problematic in terms of calibration to the damage observed in the September Earthquake and the February Aftershock.

If treated as a closed system of modelling assumptions, and input motions it reflects diligent and best current practice using the SAP software. However the results compared to the reported damage indicate the limitations of current NTHA practice in accurately modelling real building response to earthquakes.

Signed: ______________________

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Date: 23 July 2012