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**

REPORT BY ATHOL J, CARR IN RELATION TO THE INTERPRETATION OF THE SECOND COMPUSOFT NTHA

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1) Compusoft has carried out a Non Linear Time History Analysis (NTHA), the results of which are set out in a report dated 13 July 2012 ('the second Compusoft report').

2) This report addresses the following questions:

   a) Do the results of the second Compusoft NTHA change any opinion 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?

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

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

      iv) The sequence of the failure of the CTV Building on 22 February 2011?

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

UNCERTAINTIES IN THE ANALYSES

   c) NTHA results are highly dependent on the input assumptions made. NTHA results are difficult to calibrate in a quantitative way to actual performance of structures under severe seismic loading. While due care has been taken and expert opinion has been sought in determining suitable inputs, caution must exercised in the way the results are interpreted with respect to the collapse of the CTV Building.

   d) The report on the Revised NTHA of the CTV building describes the changes to the computational model required by the panel of experts and the resulting responses of the building to two of the ground motions recorded in the Christchurch CBD for the September earthquake and for the four ground motions recorded in the Christchurch CBD for the February earthquake. Two further analyses for the February earthquake were also performed using the structures damaged state following the September earthquake in a sequential analysis.

e) It was agreed amongst the expert panel that the concrete strength be taken as 1.5 times the design strength for the purposes of the Revised NLTHA. Varying the concrete strength will have an effect on the computed results. At some point sensitivity analyses should be pursued with respect to variations in concrete strength.

f) It must be emphasised that these are the computational results for the computational model of the CTV building and these computational results must be interpreted back to the CTV building. The results may not describe the actual building behaviour because of:

   i) The limitations of the computational modelling of the structural components. This is in particularly so with respect to the modelling of the beam-column joints.

   ii) The actual ground motion at the CTV site is unknown and the results are for the motions recorded at four sites in the Christchurch CBD. It is hoped that these will give a measure of sensitivity of the results to this uncertainty.

g) No allowance is made for the effects of the masonry infill on Line A nor the precast concrete spandrel panels on Lines 1, 4 and F in the Revised NTHA analyses.

h) The use of EI_{eff} =1872 kNm^2/m for the out-of plane stiffness of the 200 Hi Bond floor slab represents approximately 0.2 I_g or 0.29 I_{av} (the latter being the average of cracked and un-cracked section properties used for deflection calculations per BS 5950.4:1994).

i) Drag Bar strength. Expected strengths used are probably an upper bound. The values provided by BECA are possibly a lower bound, being based on lower characteristic strengths.

j) The beam-column joint model does not consider axial forces on the joint behaviour and does not allow for loss of vertical load carrying ability or for loss of shear strength in the adjoining beams.

k) There is no in-cycle degradation of strength capabilities of the beam-column joint model.

l) A better beam-column joint model is very desirable. However, it should be noted that the CTV joints are considerably different from those discussed and tested in the available literature i.e..precast channel beam shells with small seatings with cast-in-place inner beams. There is a potential loss of shear capability in the beam connections to the joints. An appropriate joint model is not readily available. Most joint models that are proposed in the literature are for two-dimensional joint models. Development and calibration of a three-dimensional

m) Though the joint behaviour in the computed results appears to be better than the computed column behaviour this may be a function of the limited beam-column joint model.

n) The definitions of column failure have been addressed as follows. The first criteria for column failure is deemed to occur when the concrete strain at the column cross-section centroid reaches the crushing strain of concrete. This implies widespread spalling of the concrete and a significant loss of the axial load-carrying ability of the column. Two further potential failure criteria are being monitored at the locations of vertical column bars including;

   i) Compressive strain = 0..0033 as an indicator of potential column bar buckling and

   ii) Ultimate concrete compressive strain as an indicator of significant concrete spalling. (Note: The concrete inside the column reinforcement is very weakly confined and so spalling will not stop at the line of the reinforcement. Thus columns with only moderate axial force could also degrade to the extent that they could not support their axial force -particularly when vertical acceleration effects are considered).

o) The potential pull-out of beams at the exterior joints in lines A and F and of the bottom beam bars at interior joints is not modelled in the revised Compusoft model. A selection of joints has been post-processed to see what is the likelihood of the occurrence of beam pull-out at the joints. This pull-out appears to have occurred for some joints at similar time to the column failures.

p) There needs to be a check carried out to see if the floor diaphragm strength at and between walls C and C/D is sufficient to maintain continuity with the North tower after disconnection of the drag-bars.

3) **The Revised NLTHA responses of the CTV Building to the earthquake records at 4.35am on 4 September 2010 and how they are different from the responses in the first report.**

   a) The first report only considered the CBGS ground motion as input where the second report used both the CBGS and CCCC ground motions as Input.

   b) Both ground motions analysed in the second report indicate inelastic behaviour in the line F columns (i.e. yielding of vertical bars and / or concrete compression strains exceeding 0.002). The responses in the two reports different only slightly.

c) In the first report there was no indication of drag bar failure. In the second report 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 reached approximately 95% of the disconnection force.

d) The CCCC record predicts columns in grid-line F (at multiple levels) and heavily loaded columns (at the base of level 1) exceed their ultimate compressive strain.

e) The first report did not consider inelastic behaviour in the beam-column joints. In the second report there was inelastic behaviour in some of the two-way beam-column joints in the CCCC and CBGS records. Inelastic response is predominantly for rotation about the East/West axis, i.e. due to a North/South building translation. However, none of these inelastic beam-column joints reached their ultimate strengths..

f) The walls in the North tower show some inelasticity at the base of the walls in both reports.

g) The walls in the South coupled shear wall show some inelasticity at the base of the walls in both reports.

h) The coupling beam in the South tower just above level 1 show inelastic behaviour but only for the CCCC record whereas in the first report there is yield for both Model A (without masonry interaction on grid-line A) and Model B (with masonry panel masonry interaction on grid-line A) for the CBGS record.

4) **The Revised NLTHA response of the CTV Building to the earthquake records at 12.51pm on 22 February 2011?**

a) Drag bar disconnection is indicated at all levels 4, 5 and 6 for all ground motions was indicated in both reports.

b) The first report show inelastic column behaviour concentrated in the columns in grid-line F (East side of building) whereas the second report shows inelastic behaviour for the heavily loaded interior columns as well as for the exterior columns. There was failure of the exterior line F columns. Some records predict upper level columns to exceed their ultimate compressive strain prior to the axial load carrying capacity being lost in the level 1 columns.

c) There are significantly large variations in the column axial forces which will contribute to column yield interaction. This was not modelled the the analyses reported in the first report.

d) The joints were modelled as elastic in the first report, so were not able to show inelastic behaviour. There was some post-processing to indicate that they would

e) The analyses reported in the first report only considered the case where the building was assumed to be undamaged for the analyses using the February earthquake. In the second report there were analyses using the sequence of the September earthquake record followed by the February earthquake record only show a noticeable difference in performance for the CCCC record where there was a loss of drag-bar connection indicated at level 4 in the September earthquake. This appears to have delayed the loss of connection of the last drag-bar to fail as it now fails later in the analysis than is seen for the analysis of the undamaged structure model. The displacement history maintains a very similar form and magnitude irrespective of whether the NLTHA model of the structure was assumed to be damaged or un-damaged at the time of the February earthquake.

5) **Why the CTV Building may have failed on 22 February 2011 based on the Revised NLTHA analysis.**

a) The failure of the drag-bar connections at levels 4, 5 and 6 is predicted for all four ground motions. This will add increasing demands on remaining North core diaphragm connections. This result is similar in both reports.

b) The second report indicates the inability of the heavily loaded interior columns to carry their imposed vertical loads when subjected to the effects of the inter-storey drifts was sufficient to lead to loss, or spalling, of the cover concrete. This concrete spalling could penetrate further into the columns beyond the longitudinal reinforcement because of the very limited confinement. With the loss of the cover concrete and the very small amount of confining steel may also allow buckling of the longitudinal reinforcement, particularly if it had earlier yielded in tension. The large variation on the axial forces due to the very large vertical accelerations may also have had a part to play in the column failure. This is not evident in the first report which concentrates more on the failure of the columns on line F.

c) The beam-column joints also indicate large inelastic effects, in many cases exceeding their maximum capacity at the same time or soon after the column failures. This is not modelled in the first report.

d) There is the likelihood of beam pull-out from some of the exterior columns. This is not modelled in either report.

e) The inter-storey drifts on line F without Spandrel Panel interference shown in Figure 55 of the Revised Compusoft report are greater than the columns can sustain. These interference effects were modelled in the first report but not in the second report.

6) **The Revised NLTHA Collapse scenario appears to be.**

a) Drag-bar failure,

b) Potential disconnection of diaphragms to North core.

c) Inter-storey drifts greater than the column drift capacity.

d) Potential failure of upper level columns due to the onset of spalling of concrete.

e) Loss of axial load carrying ability starting with column on line 2 (column A2 with the CBGS record and column C2 with the other three ground motions) at the ground level followed rapidly by the other interior columns. The time interval over which a significant number of column failures occurs appears to be less than 0.3 seconds. Some columns also indicate a loss of axial load carrying ability in the upper floors, such as level 3 for column C2. It should be noted that the computed results after vertical load carrying capacity has been lost should be reviewed with caution as the analysis only considers small displacement effects..The first report demonstrates these effects as occurring in the exterior columns on line F.

f) Failure of beam-column joints in lines 1, F and A. There is a possibility that this may not be relevant as the structure may have already failed and the responses computed after that point may be meaningless. These were not modelled in the first report.

Signed: …………………………………

PROFESSOR ATHOL CARR

Date:…………………………………..