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

JOINT REPORT IN RELATION TO INTERPRETATION OF SECOND COMPUSOFT NTHA

JOINT REPORT IN RELATION TO INTERPRETATION OF SECOND COMPUSOFT NTHA

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) Counsel assisting the Royal Commission asked the following experts to confer:

   a) Dr Clark Hyland

   b) Ashley Smith.

   c) Derek Bradley/Barry Davidson/ Tony Stuart

   d) Professor John Mander.

   e) Dr Brendon Bradley

   f) Professor Nigel Priestley

   g) Rob Jury

3) Professor Athol Carr facilitated discussions between the experts and oversaw the production of this joint report.

4) The experts addressed the following questions:

   a) What do the results in the second Compusoft report indicate about:

      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) 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?

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?

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

5) The experts attempted to reach agreement in relation to the questions set out in clause 4(a).

AREAS OF AGREEMENT

a) 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, the NTHA Panel recommends caution in the way the results are interpreted with respect to the collapse of the CTV Building.

b) 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.

c) It must be emphasised that these are the computational results for the structural analysis model of the CTV building and these computational results must be interpreted back to the actual 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.

Agreement on uncertainties in the computational model and its data.

a) It was agreed that the concrete strength be taken as 1.5 times the design strength for the purposes of the Revised NLTHA. (Note there is to be a future 'hot tub' discussion between concrete experts at the Royal Commission to discuss the various concrete test results and this may provide further guidance on the likely concrete strengths).

b) No allowance for 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 Revised NLTHA.

c) The use of EI_{eff} =1872 kNm^2/m for 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).

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

e) The beam-column joint model does not consider the time variation in axial forces on the joint behaviour (initial gravity loads used) and does not allow for loss of vertical load carrying ability or for loss of shear strength in the adjoining beams.

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

g) A better beam-column joint model is very desirable. However, it should be noted that the CTV joints are different from those discussed predominatly 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 and so parametric analyses of the joint constitutive model would be beneficial in subsequent analyses. Most joint models that are proposed in the literature are for two-dimensional joint models. Whereas some of the joints with high vertical loads are in te interior of the structure and are largely two-dimensional in nature the inelastic effects appesr to be more dominant in the exterior jouint which are three dimensional. Development and calibration of a three-dimensional model would take a considerable amount of time and effort even if experimental data on this type of joint was available.

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

i) 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.

j) It should be noted that the fibre modelling of columns does not explicitly consider bar buckling.

k) 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).

l) 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.

m) Because of computational demands, and time constraints, the analyses utilize small displacement theory (with a "P-Delta adjustment"). Hence, geometric nonlinearities are not explicitly considered, which maybe significant given the high axial loads on vertical load resisting elements.

2) **The Revised NLTHA responses of the CTV Building to the earthquake records at 4.35am on 4 September 2010?.**

a) 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)
b) 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.
c) 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.
d) 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..
e) The walls in the North tower show some inelasticity at the base of the walls.
f) The walls in the South coupled shear wall show some inelasticity at the base of the walls.
g) The coupling beam in the South tower just above level 1 show inelastic behaviour but only for the CCCC record

3) **The Revised NTHA 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 four ground motions early in the analyses.
b) The performance of individual column hinges has been assessed first with respect to the time at which vertical load carrying capability has been compromised. An indicator of this could be taken to be when the compression strain at the column centroid exceeds the ultimate strain capacity of the concrete. Table 44 below presents the time at which the first four column hinges are seen to exceed this criteria, based on ultimate compression strains between 0.006 0.00825. from Figure 6 in the Compusoft report. In Table 1: shows column axial failure times (note L1 denotes level1 and b is base of column)

Table 1: Column axial failure times (note L1 denotes level1 and b is base of column)

CBGS		CBGS (seq)		CCCC		CCCC (seq)		CHHC		REHS	
ID	Time	ID	Time	ID	Time	ID	Time	ID	Time	ID	Time
A2L1b	6.82	C2L1b	6.82	C2L1b	4.06	C2L1b	3.88	C2L1b	5.82	C2L1b	6.54
C2L1b	6.84	D2L1b	6.84	D2L1b	4.06	D2L1b	4.06	D2L1b	5.94	C2L3b	6.72
D2L1b	6.86	B2L1b	6.86	B2L1b	4.24	B2L1b	4.10	C1L1b	6.28	C1L2b	6.74
B2L1b	6.96	E2L1b	7.10	C2L3b	4.30	C1L1b	4.28	C1L4t	6.34	D2L1b	6.88

c) 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.

d) There are quite large variations in column axial forces which will contribute to column yield interaction.

e) There is considerable damage to joints, particularly in the two way joints with more damage being seen in grid-line A than in grid-line F. It appears as if beam column joint strength degradation occurs after some columns hinges are shown to lose their vertical load carrying capacity

f) Post-processing of the potential for beam-pullout suggests that such failure is likely to have occurred. As a result, ideally this failure mechanism would be modelled explicitly in any revised analyses, to allow for effects subsequent to this failure to be considered, and understand whether it is important in the global failure of the CTV structure.

g) The 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.

4) **Why the CTV Building may have failed on 22 February 2011 based on the Revised NTHA 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.

   b) 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.

   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.

   d) The possibility of beam pull out from at least some of the exterior columns cannot be excluded..

   e) The inter-storey drifts on line F without Spandrel Panel interference shown the Revised Compusoft report are greater than the columns can sustain.

5) **The Revised NTHA Collapse scenario appears to be.**

   Variability and uncertainty in physical properties and analysis processes do not allow a particular scenario to be determined with confidence.

   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.

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.

AREAS OF DISAGREEMENT

1) **Column Failure Criteria.**
   a) Dr Hyland considers that the concrete column failure criteria could be given by the cover concrete reaching the ultimate concrete strain $\epsilon_{cu} > 0.004$

2) **Slab Flexural Stiffness.**
   a) Dr Hyland is unsure of how the lower value of flexural stiffness used in the Revised NTHA was justified.

3) **The Revised NTHA response of the CTV Building to the earthquake at 4.35am on 4 September 2010.**

   a) Dr Hyland regards that the damage predicted by the Revised NTHA does not agree with the level of observed damage following the September earthquake. Column damage indicated for the CCCC record should have been observable, particularly at the upper levels. The revised NTHA appears to over-predict the damage for the September earthquakes, although only slightly for the CBGS record.

4) **The Revised NTHA response of the CTV Building to the earthquake at 12.51pm on 22 February 2011.**

   a) Dr Hyland comments that the scenario does not include allowance for interaction with masonry infill on Line A nor interaction with Spandrel Panels on Lines 1, F and 4.

5) **The Revised NTHA Collapse scenario appears to be.**

a) Mr Smith comments that there was failure of the line F columns. Some records predict mid to upper level columns to exceed their ultimate compressive strain prior to axial load carrying capacity being lost in the level 1 columns

Signed: ....

PROFESSOR ATHOL CARR

Date: 26th July 2012 ....

Signed: ....

DR CLARK HYLAND

Date: 26/7/12 ....

Signed: ....

ASHLEY SMITH

Date: 26/7/2012 ....

Signed: ....

DR BARRY DAVIDSON

Date: 26th July 2012 ....

Signed: ....

DEREK BRADLEY

Date: 26/07/2012 ....

Signed: ....

TONY STUART

Date: 26/07/2012

John Mander

Signed: By his authorised agent

PROFESSOR JOHN MANDER

H.B RENNIE

Date:..

Signed: ....

DR BRENDON BRADLEY

Date: 26/07/2012

Signed: ....

ROB JURY

Date: 26/07/2012