Section 3

AN ALTERNATIVE GRAVITY-DOMINATED COLLAPSE SCENARIO

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3.1 Overview of alternative hypotheses

- *The Trigger*
- *The Incipient Failure*
- *The Collapse Mechanism*

Figure 3.1. Beam-to-column connection failure sequence at the west wall; a trigger for the East-West Collapse Mode

Sequence | **Sketches**

Stage 1

The building sways to the west with a large velocity pulse. The E-W beams on column lines 2 and 3 at the West wall are required to form large negative moments that cause the joint core concrete and the beam-soffit cover concrete to crush.

Stage 2

During the next half-pulse the building lurches eastward. The beam along line 2 and 3 pull away from the west wall and their line A column seats to form the alternating positive moment. The crushed cover concrete from the previous reversal spalls off and the beam slumps down a little, with a partial or full loss of seating. Due to the loss of seating at the support line A there is a transfer of the previous gravity load from the tributary area of the beam onto the neighboring columns on line B. This action an axial force increase of up to 40% the columns along line B

Stage 3

As the building attempts to return to an upright condition by moving west, the unseated beams are inhibited from fully returning due to the presence of the west wall.

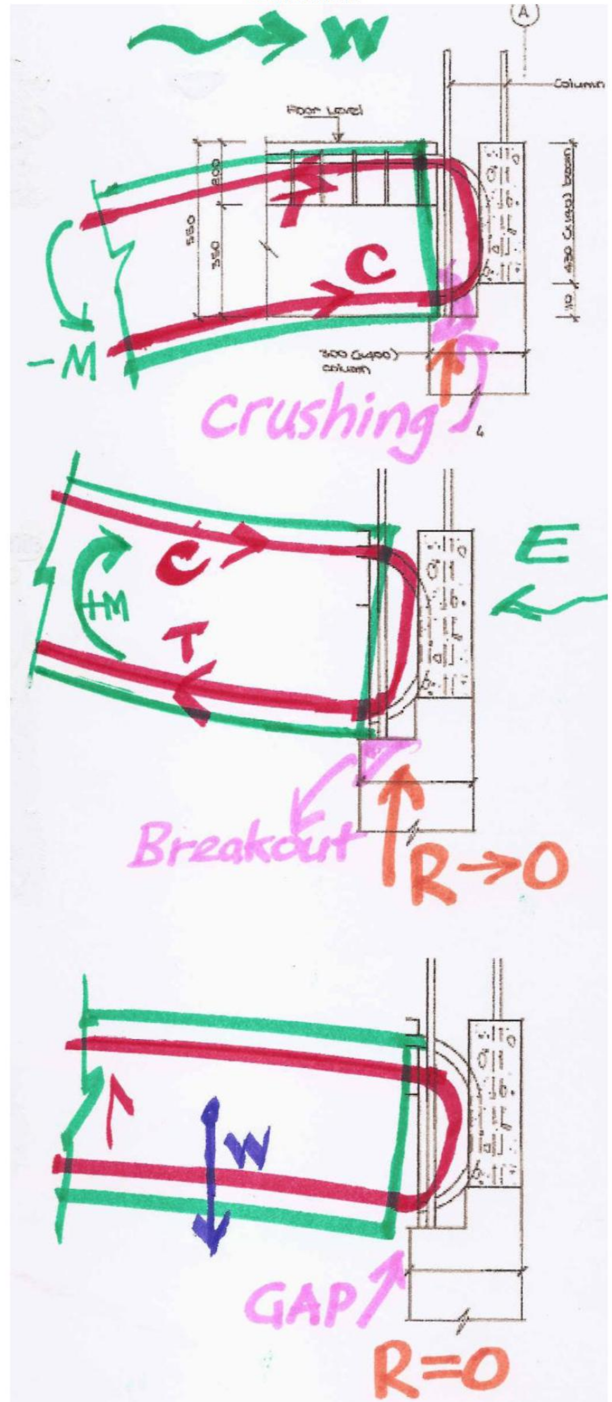


Figure 3.2. Four-story double bending buckling failure starting on column Line B Leading to the East-West Collapse Failure Mode

Stage 4

Permanent differential deformations remain, that inhibit the columns along line be from remaining straight. This sets the columns up for a classic Euler buckling type failure, especially under further axial load derived from vertical accelerations and their consequent vibrations

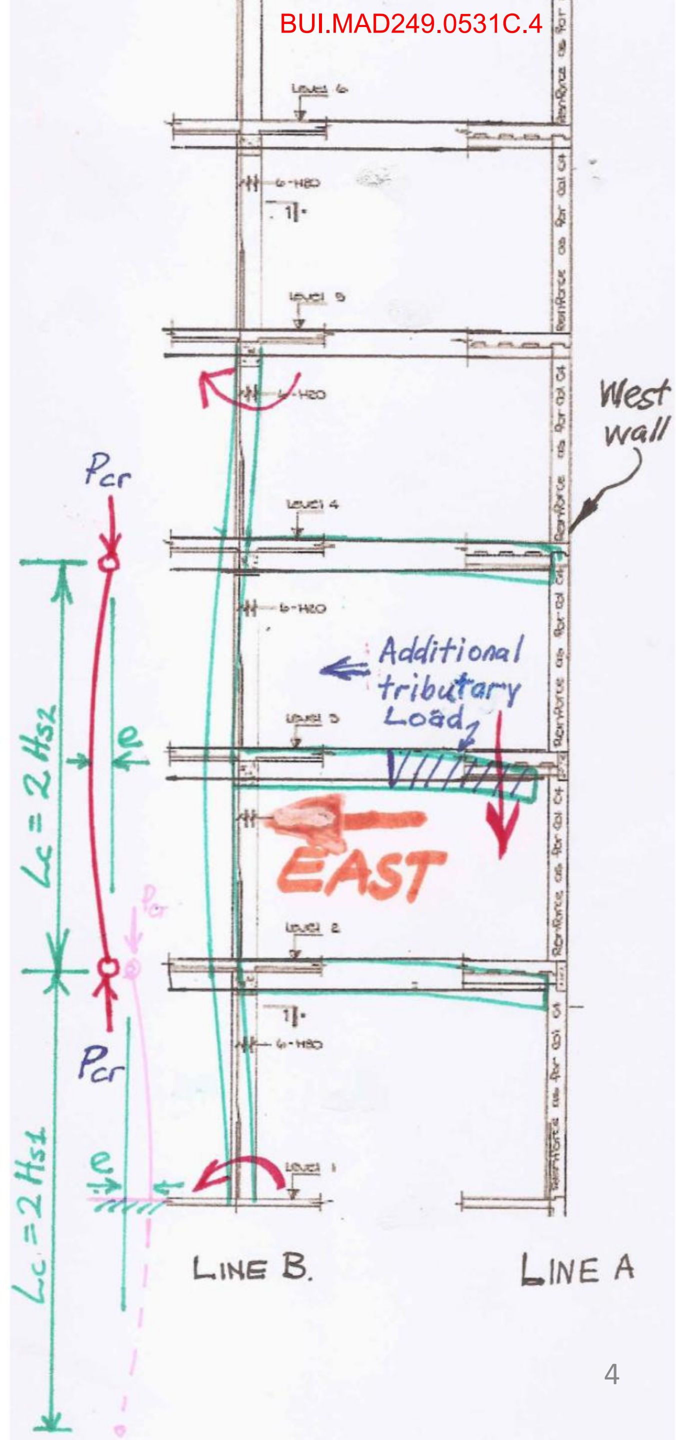


Fig. 3.3. Formation of the E-W Collapse Mechanism

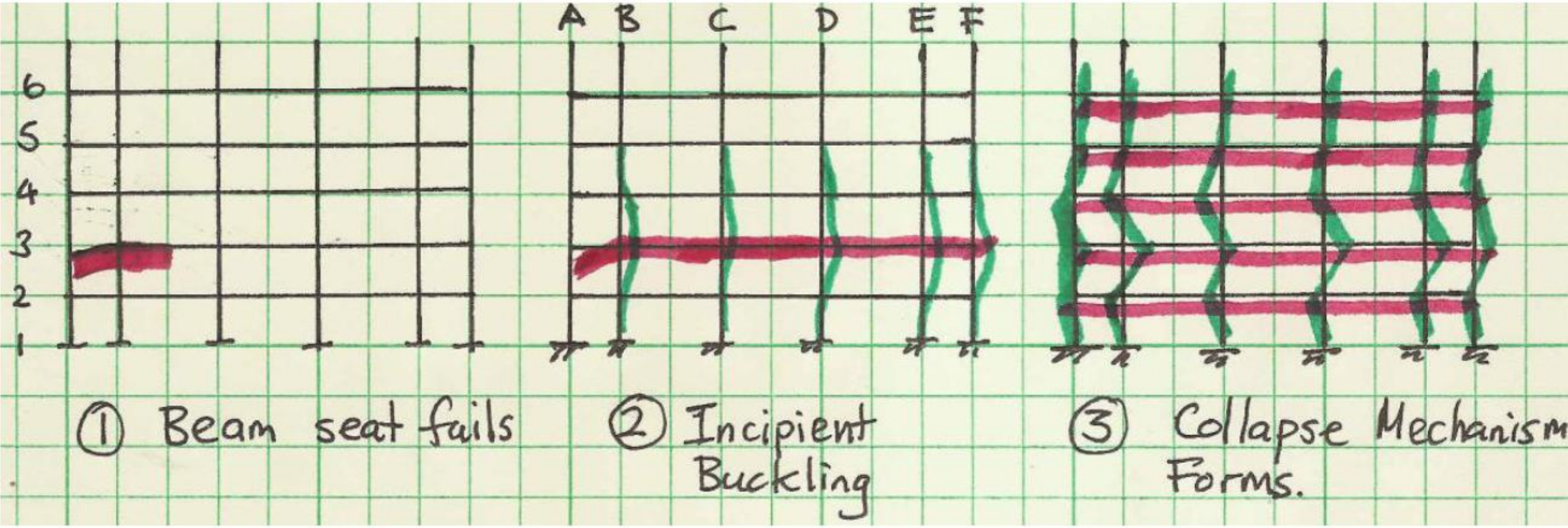
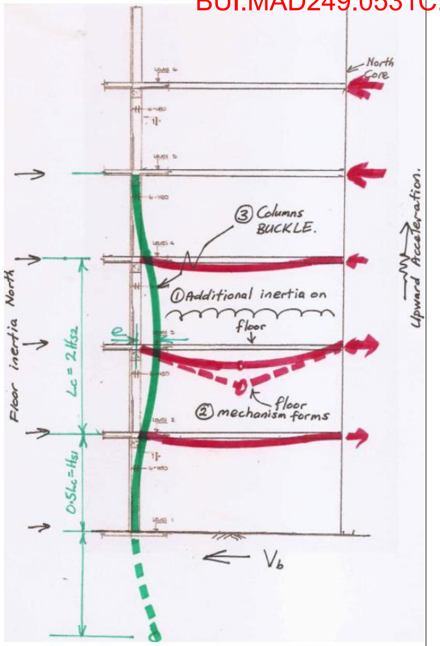
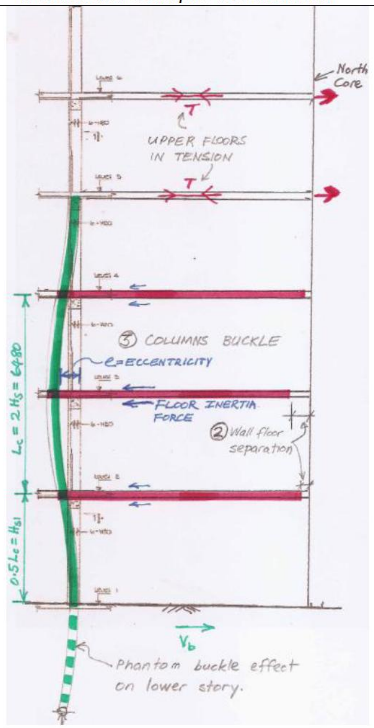


Figure 3.4. North-South Collapse Failure Modes

SEQUENCE	**SKETCHES**
Step 1 Due to the lack of beams in the N-S direction and very high vertical motions, the in-plane stiffness is low. The slab buckles downward due to a combination of upward vertical acceleration, and N-S sideway of the frame. ***Step 2*** Because the slab buckles, and the columns lack lateral support in the N-S direction, a 4-storey, double bending column buckling mechanism forms. ***Step 3*** Column buckles and collapses.	 *Northward Collapse Mechanism*
Step 1 Due to the absence of drag bars in the lower stories, there is a large strain demand placed on the slab steel connecting with the North core. After one or two cycles the bars fracture due to low cycle fatigue. ***Step 2*** The columns on lines 2 and 3 lack lateral restraint from moving independently southward, therefore they move away from the north core. A double-bending column buckling mechanism forms in the lower four stories. ***Step 3*** Several columns buckle and the structure collapses downward.	 *Southward Collapse Mechanism*

SEQUENCE

Step 1

Due to the lack of beams in the N-S direction and very high vertical motions, the in-plane stiffness is low. The slab buckles downward due to a combination of upward vertical acceleration, and N-S sideway of the frame.

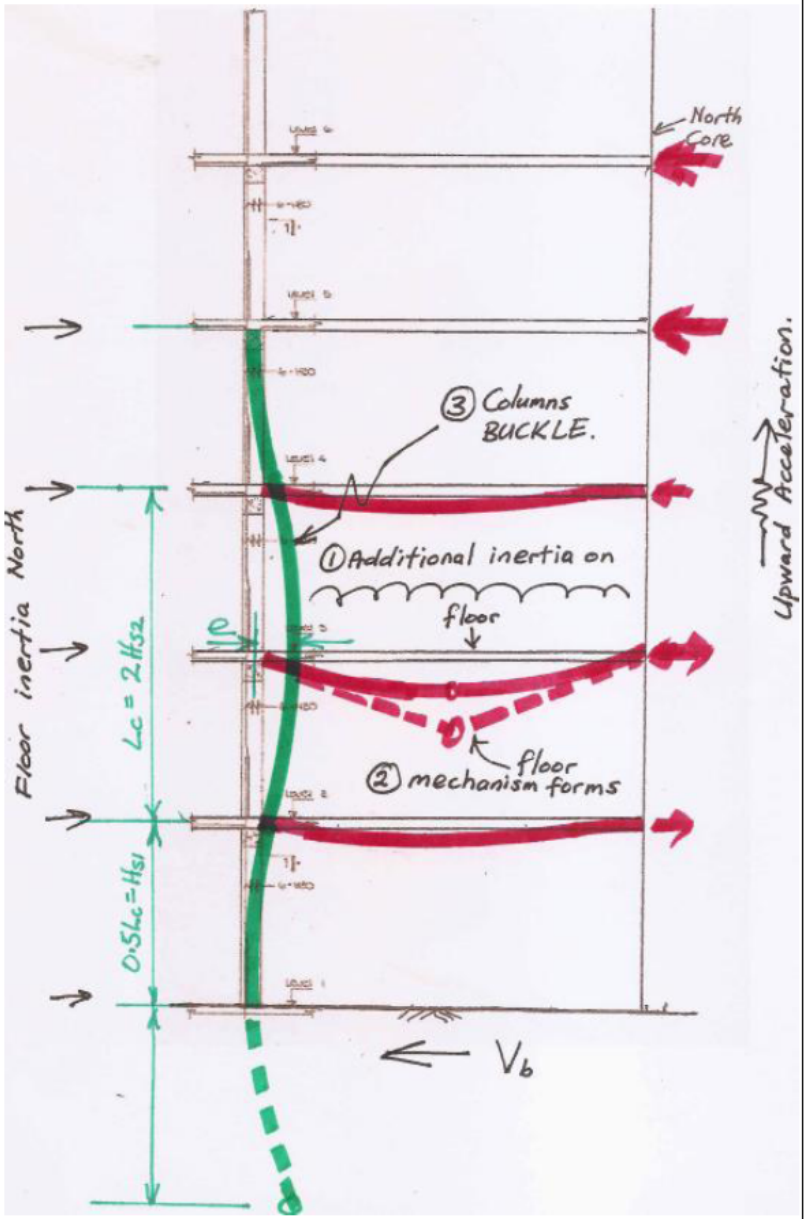
Step 2

Because the slab buckles, and the columns lack lateral support in the N-S direction, a 4-storey, double bending column buckling mechanism forms.

Step 3

Column buckles and collapses.

SKETCHES



Northward Collapse Mechanism

Step 1

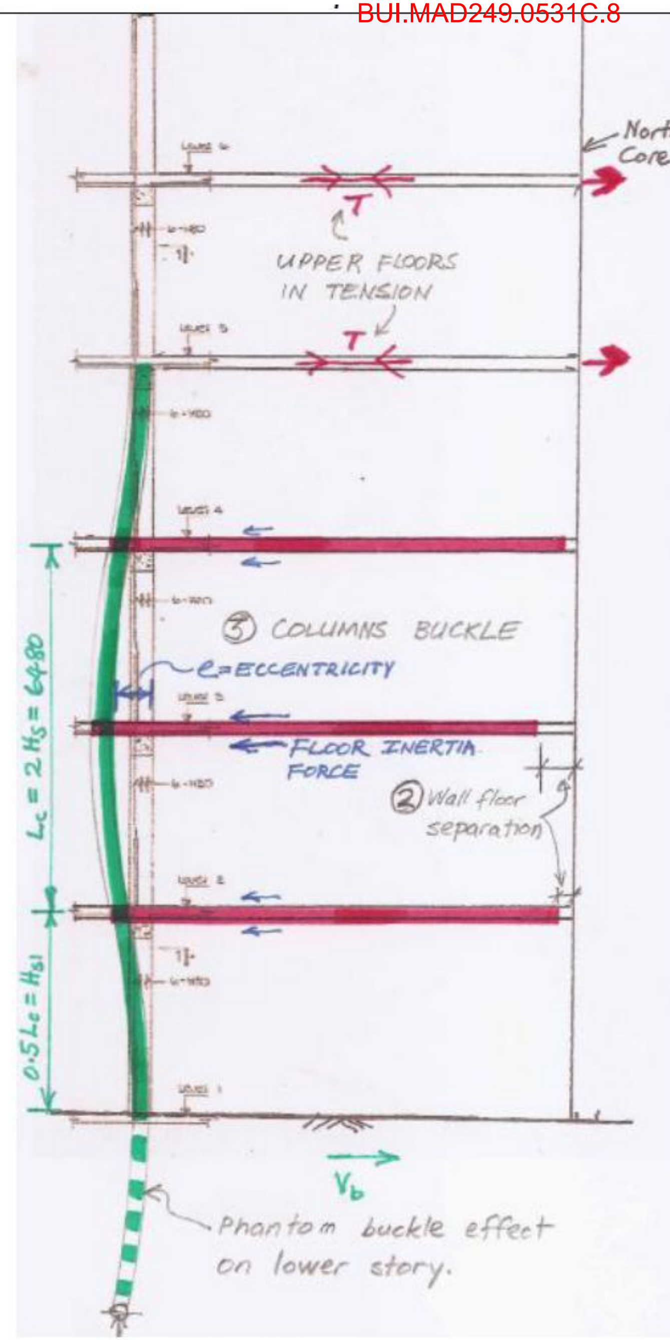
Due to the absence of drag bars in the lower stories, there is a large strain demand placed on the slab steel connecting with the North core. After one or two cycles the bars fracture due to low cycle fatigue.

Step 2

The columns on lines 2 and 3 lack lateral restraint from moving independently southward, therefore they move away from the north core. A double-bending column buckling mechanism forms in the lower four stories.

Step 3

Several columns buckle and the structure collapses downward.



Southward Collapse Mechanism

4. Closure

4.1 The CTV Building was designed and constructed in an innovative fashion. This structure was one of the first in a new generation of multistory buildings in the 1980s that used precast components. Instead of using a ductile moment frame as had been the custom for cast-in-place structures of the day, the CTV Building was designed with a "strong" wall system coupled with an "elastic" frame of columns and beams to support a proprietary type of floor system composed of a lightly reinforced slab cast on galvanized steel metal-rib decking. The Building was designed to the NZS 4203 Loadings Code, and a deflection check was made to ensure the displacements under the code-specified seismic loading were not excessive and that the columns remained within the elastic range.

4.2 When the Darfield Earthquake struck, it imposed ground accelerations that were essentially similar to the design limits for which the structure of the CTV Building had been designed. As a consequence, the structure was damaged; such damage would be expected, by design. The structure did not collapse, and met its design objective of ensuring life-safety.

4.3 In light of the possibility of a large aftershock, and given the fact the engineers knew many structures around Christchurch had either met or exceeded their design expectations, they strictly should have been immediately Red Stickered by fiat; a site inspection was not even necessary to make this decision. Following this period, such buildings should have been both inspected and analyzed for collapse potential in subsequent earthquakes. If necessary, gravity critical structures (such as the CTV Building) should have been shored up to ensure collapse prevention while valuables could have been retrieved and repair/retrofits implemented.

4.4 The CTV Building was inspected after the Darfield Earthquake and damage noted and the building deemed safe to reoccupy. However, the owners/engineers evidently did not pay heed to the many reports from the CTV Building occupants that the building felt uncomfortably lively. Further questions should have been raised regarding the soundness of the structure by the owners and thoroughly investigated by the assigned inspecting engineers.

4.5 The CTV Building tragically collapsed in the Christchurch Earthquake with a significant loss of life. An investigation into the collapse by the DBH led to the H-S Report. This report has been discussed and critiqued in this submission. There are several assumptions and various aspects of the H-S Report that bring into question the veracity of the claims and conclusions. In fact the peer reviewer Holmes, as well as the DBH expert advisor Priestley, are not in agreement with key aspects of the report. It is for this reason further work is essential.

4.6 One of the key areas leading to faulty conclusions in the H-S Report concerns the concrete strength. Testing and analysis commissioned by ACRL, and undertaken by independent experts, demonstrated that the concrete was not deficient as claimed in the H-S Report. In fact the concrete strength is likely to be in the range of 1.5 times the specified design strength.

4.7 Another key area of deficiency in the analysis is the correct modeling of the columns, coupled with the degrading strength of the beam-column joints. Axial load-moment interaction was not correctly considered in the NLTHA. Also, the beam-column joints that had no transverse reinforcement were modeled as rigid end blocks. As such the strength deterioration that occurs when the joint core concrete cracks was not modeled.

4.8 Further nonlinear time history analysis is needed to fully understand the nature and causes of the collapse of the CTV Building. In those analyses it will be essential that all four Geonet motions recorded during the Christchurch Earthquake are included in order to correctly gauge the spread of results that might have conceivably happened at the CTV Building site on February 22, 2011. Moreover, it is essential that the effect of the weakened structure following the Darfield earthquake be captured. This is most easily done via an end-on-end analysis, where the damage done in the Darfield Earthquake is captured. In previous analyses detailed in the H-S Report on the work performed by Compusoft, the program was stopped at the completion of the Darfield Earthquake and then restarted as if the structure was undamaged at the commencement of the Christchurch Earthquake.

4.9 Analyses as part of this submission show that a sway failure is unlikely, and that a classic elastic Euler buckling failure over the lower four stories is possible in either the E-W or the N-S directions. Such a failure does not rely on significant, if any, post-elastic performance. The lower four stories were able to buckle due to the relative movement of the floors with respect to the shear wall system, and the relative movement necessary to achieve this need only be small, in the order of 30 mm. The collapse is primarily caused by the substantial increase in axial loads in the columns due to the exceptionally high vertical accelerations.

Supplementary Comments

- Q: How could the CTV Building collapse be avoided?
- A: Simply use larger columns. More transverse steel is desirable (best-practice), although not essential (circa 1986).
- Bottom slab steel through beam supports would improve connectivity under high vertical motions.