PINE GOULD CORPORATION BUILDING

BECA PROGRESS REPORT, 29 June, 2011

Comments by Nigel Priestley, Updated, 28/7/11

Executive Summary:

- Paragraphs 8 and 9 seem contradictory. (Still not addressed. Also, the statement in para. 8 seems to be contradicted by the final section of p2 of the Exec Summary. Note that columns with heavy axial load are likely only to exhibit hairline cracks after EQ activity, even if ductility has occurred, as the axial load is sufficient to yield the reinforcement in compression, and close the cracks. I suspect this would also apply to the walls, as an axial compression stress of only 0.74MPa would yield all vertical bars and close cracks.)
- Paragraph 10: Failure of the E wall in compression or tension? (This is discussed in more detail subsequently. (Still not adequately addressed)
- Second-to-last para, p1,line 1: change "to not" to "do not".
- **Mode of collapse:** This seems incomplete to me. Why did the East wall fail in compression? If the failure was initiated by fracture of the rebar in the West wall, it might be expected that the wall would then rock, with very high displacement capacity. I suspect that the gravity load system played a part, as discussed subsequently. Note that the vertical reinforcement ratio (about 0.0025) may have been too low to get a spread of plasticity in the wall, resulting in condensation of the plastic hinge length. (i.e. tension strength of the concrete exceeded that of the tensile reinforcement). (Please respond to highlighted section)
- **Commentary:** The 1997 study identified that the structure was well below current standards. Did (should) the study have recommended strengthening beyond the addition of perimeter steel columns? No concerns about interior columns?

Section 4 Building Description. (Greatly improved now)

The description is inadequate, in my view. No mention is made of the flooring system, or the connection between the floors and the shear core. The level of axial force in the columns should be identified, and some comments about the ability of these columns, and their connections to the flooring system, to resist increased axial force and lateral displacement should be made. ~~By contrast, the detail provided to establishing that the ground conditions were not critical seem excessive.~~

Note that the figure numbering needs tidying up (e.g. 2 figure 4.4's). Note also the first fig 4.4 seems to imply that steel props were only placed at the four corners of the building (see note in Fig.4.4)

Did the foundation structure have sufficient mass to avoid the shear core rocking on its foundation? Given the low rebar ratio, I would guess so, but it might be useful to state this particularly in light of the 1997 assessment.

(A minor point: You describe the ground floor as level 0, and the first floor as level 1. The CTV study describes ground floor as level 1. I think the approach should be consistent – Perhaps DCH to arbitrate?)

Section 5.3 1997 Structural Report

This 1997 report identifies the columns as being overstressed significantly in plastic rotation under current design seismicity. The BECA report does not identify columns as being a critical element of the structural response. Does BECA disagree with the 1997 conclusion, or does it believe that the addition of perimeter columns solved the problem? If so, I think this should be stated. It seems that BECA disagree with conclusions in the 1997 report about the core rocking on its foundation under seismic response. If so, I think this needs to be stated to avoid ambiguity in the BECA report. (Please respond)

Section ~~6~~ 4.2 Site Investigations: ~~Ground Conditions~~ (OK now)

~~As noted above the detail provided here is inconsistent with the detail provided in the building description.~~
Rest of Section 4: No comments

Section ~~7: Ground Motions in the 3 Earthquakes~~ 5: EQ Effects on Site and Bldg

As mentioned in my report on the May 30th interim report, I would like to see acceleration and displacement spectra as well as the capacity spectra, if possible.
This is still my preference. The capacity spectrum gives a good general view, but is very difficult to interpret for a specific period.

~~Despite the title of Section 7, spectra are only shown for the Feb 22 event. The spectra for the other two earthquakes should also be shown –particularly for the Sept. 4 2010 event.~~
Note that the paragraph after the acceleration spectra in Section 5.1 is not quite correct. The period lines are not related to the flexible structure (which to me implies PGC, but to specified elastic periods.

Section 5.5.2 (Performance in Sept22: The photos in Fig.5.5 indicate significant response to me. The spalling of rendering (and surface concrete) at the intersection of the 2 diagonal cracks, and along the lower diagonal in the left photo indicates that there was some sliding on the shear cracks, possible as a result of wide flexural cracks at peak response.

Section 6: It seems, given the title for Section 5, that this should be renumbered Section 5.4 (also logical, since 5.2 and 5.3 describe Sept and Dec.

Section 6.1.1 The first paragraph refers to duration of shaking, and implies that this can be seen in Fig. 6.1. This is also implicit in the title of Fig.6.1. In fact, it is the response spectra that are shown.

Section 7: Excellent description of failed condition.

Section 8: Evaluation/Analysis
~~**Section 9.3 Analyses Completed and Results**~~
Section 8.1.3 See comment on Fig.4.4 above

Section 8.1.4: Title is confusing as is seems to relate to the actual EQ performance. Perhaps "Assessed Capacity of the Building" would be better?

Section 9: I think the word "Consultant" is inappropriate in the title (taken from DCH outline suggestion?), and the first sentence should be deleted, for the same reason.

Renumber Fig 8.1 as 9.1. Also change references in text to 9.1.

Section 10:

Section 10.2 Para 2: The two sentences seem contradictory. Perhaps add "However" at the start of the second sentence, and refer to Fig.5.5?

I am still a little uneasy about the compression failure of the east wall: What was the calculated compression stress on this wall before and after fracture of the west wall reinforcing steel? (Please comment)

The two columns on De and Ee (12"x12") are not discussed at all, though these were unpropped, and would have high compression stress. Are these not felt to be important? (Please comment)
~~The report does not describe how the curvature is transformed to plastic rotation (i.e. what was the assumed plastic hinge length). Given the low reinforcement ratio in the walls, it is not clear to me that normal equations for plastic hinge length would be appropriate. Predicted displacement capacity will be extremely sensitive to this assumption.~~ Now covered adequately in Appendix.

APPENDICES

Good, and well presented.

Appendix A4

Generally well described. Just a few comments:

- A4.3.3.2: Mass described as lumped at each floor level. Presumably this was not the case for the Full ITHA model, where distributed mass would be needed for the vertical response. How was vertical acceleration handled: the time step would be crucial for representation.
- Modelling of the columns and beams is not described – some info required
- Did you compare the wall shear strengths with the maximum shear forces from the ITHA? These may be substantially higher than those "consistent with flexural failure" as described in the report. In fact I am uncertain what this means – shears from a reduced modal superposition, from an inverted triangle of inertia forces, or from modified modal superposition (Priestley, Calvi, Kowalsky, 2007)? **Please comment!**
- Displacement capacity of the columns (particularly the 2 interior columns) should be stated, and discussed.
- A5.6: Diaphragm connection forces. How were these determined? If from ITHA, then OK, but if from modal analysis the results will be low unless the higher mode contributions are unreduced (i.e. the elastic values). **Please comment.**

I understand that state-of-the-art equations for determining shear demand (including higher mode effects) and shear capacity were used to justify the statement that shear capacity was OK. If so, this should be stated. If not, the methods for determining shear capacity and demand should be noted.

~~Fig.9.1 compares capacity determined from pushover analysis with the capacity spectrum for the Feb 22 event. A similar graph should be provided for the Sept 4, 2010 event. Given that the capacity is significantly below the current NZS1170 demand, I suspect that the Sept.4, 2010 demand may well exceed, or at least be very close to, the calculated capacity. If so, this will need discussion in the report. Is it possible that the Sept.4 event caused fracture of the reinforcing steel, but that residual cracks were small because of localized cracking? This could explain the increased liveliness of the structure reported by inhabitants. Note that Academy Towers, which had a similar reinforcement ratio to PGC suffered fractured reinforcement in the Feb22 event at the base crack, which remained small after the earthquake as gravity loads were sufficient to close the crack. Crack width alone is not a sure indicator of damage.~~

~~It is stated that capacity was greater than required by NZSS 95. It would be helpful for a graph to include the inferred capacity spectrum for NZSS 95 and compare with capacity. I suggest that such a graph include the Pushover curve, and capacity spectra for NZSS 95, NZ 1170, Sept,4 2010, and Feb.22, 2011.~~

Section 10, Conclusions

Given the very low reinforcement ratio, and its location at the centre of the wall in a single layer, I rather doubt that incipient buckling of the rebar would put sufficient force on the wall to spall it, and make it lose compression capacity, though this is only a suspicion on my part. The alternate hypothesis – that fracture of the west wall rebar caused such rotations that the compression flange failed also seems rather unlikely – if the rebar fractured, the wall is likely to behave as a rocking system, which could protect it from damage. **I'm still worried by this**

I wonder if interaction between the shear core and the flooring system might have played a role in the collapse. ~~No information~~ **Little** is given in the report about the floor system, but it is clear that as the wall displaced laterally, fracturing the west wall rebar, it will have increased the axial force on the adjacent columns to the east, which were apparently already overloaded. This could have been combined with additional compression from vertical response. The combined effects could have been enough to fail the columns in compression, combined compression and bending, or joint failure. If this occurred catenary action from the failing floor system would tend to pull the east wall over, and overload it in compression, leading to a failure very similar to that noted. The separation of the roof slab from the shear core is consistent with this. **Still of concern to me.**

~~Appendix 3 Structural Analyses~~

~~As noted earlier, the means for converting moment-curvature to force-displacement needs to be clarified.~~

~~It seems that both the "stick" model and the "full" model describe only the shear core, and that the flooring system was not included in either model. If this is correct, the reasons for this need to justified.~~

~~The steel stress-strain model of Fig.A3.3.2 doesn't look very realistic – I should have thought that the peak stress should be achieved at about 0.08 rather than 0.02 strain. However, this will not have a great influence on the results.~~

~~I don't understand the wall stiffness used in the stick analyses. My understanding is that the shear core was described by the results of a moment-curvature analysis. If this is the case, are the quoted values (which seem high) used only outside the plastic hinge region? A simple diagram describing the stick model would be useful.~~