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Justice Mark Cooper, Chair
Royal Commission of Inquiry into Building Failure Caused by the Canterbury Earthquakes
PO Box 14053
Christchurch Airport
Christchurch 8544
New Zealand

SUBJECT: REVIEW FOR ROYAL COMMISSION

Structural Performance of Christchurch CBD Buildings in the 22 February Aftershock: Stage 1 Expert Panel Report dated 30 September 2011 (DBH Expert Panel)
Investigation into the Collapse of the Pyne Gould Corporation Building on 22nd February 2011 dated 26th September 2011 (Beca)
Report on the Structural Performance of the Hotel Grand Chancellor in the Earthquake of 22 February 2011 dated 26 September 2011(Dunning Thornton Consultants Ltd)
Investigation into the Collapse of the Forsyth Barr Building Stairs on 22nd February 2011 dated 26th September 2011 (Beca)

Dear Justice Cooper:
In accordance with your request, I have monitored the development of the subject reports and reviewed the final versions. My comments on the process of preparing the reports and the conclusions and recommendations of the reports are contained herein. My comments are intended to be informative to the Commission, but in some cases point out a lack of clarity or identify unanswered questions in the reports and investigations.

More specifically, my review consisted of the following:

- Review of Department of Building and Housing Terms of Reference for their Technical Investigation and their Project Plan dated 18 April 2011;
- Review of four draft individual buildings reports available about 30 June 2011 (CTV building was still included);
- Meeting with David Hopkins at DBH 8 July 2011 to discuss June reports;
- Review of four draft individual building reports available in late July and early August 2011 and transmittal of brief comments to the Commission;

- Review of final reports on 3 buildings (the CTV building report was delayed) and the Expert Panel Report as listed in the Subject, above.

Comments on Process for Development of Reports

The organizational structure for development of the reports, which included an expert panel to provide overview of reports developed by individual consultants, is commended. Input from individual outside panel members during development of an investigative report, even if not always in total agreement, should be expected to improve the thoroughness and thoughtfulness of the investigation. In the end, the panel was charged with "reviewing and approving the engineering consultant reports on each building" (quote from the Terms of Reference for the Expert Panel).

Data used for input into the investigations was also thorough and typically included:

- Original design drawings;
- Any known alterations;
- Site soil and foundation data;
- Material strength data;
- Best estimate ground motions at or near each site;
- Reports from eye witnesses and emergency responders, including interviews and photos;
- Extent of structural damage from 4 September 2010 and Boxing Day earthquakes (based on reports available) and the post earthquake safety stickers issued;
- Detailed descriptions of structural damage and/or collapse from the 22 February 2011 earthquake.

Comments on Reports and Conclusions

In general, the reports are thorough and clear and I have no major disagreements with any findings or conclusions. However, I have many comments and observations generated by the contents of the reports—or in some cases by what was not explained in the reports. I have not performed any significant calculations to check the results of the Consultant's analyses, so my comments are more related to methods used in the investigations as viewed internationally, the logic of conclusions, and other issues that I have judged might be of interest to the Royal Commission.

My comments will be organized around the chapters of the Expert Panel Report (EPR), starting with Chapter 4. The chapters in the Expert Panel Report covering the three individual buildings (Chapter 5, 6, and 7), in general are accurate summaries of the Consultant Reports (CR). Comments on those investigations will identify whether the source of the comment is the EPR or the CR. Comments and concerns noted in earlier reviews, if not resolved in the final reports, will not be specifically identified, but will be included below.

Chapter 4.0 Context

a. Article 4.3.1: I believe the most complete description of the performance intent of the New Zealand seismic design standards is found in the commentary of NZS 1170.5:2004. That commentary on page 9 suggests that although buildings are designed primarily for ground shaking intensities expected to occur, on average not more than once every 500 years, there is a margin against collapse at that design level of "at least 1.5 to 1.8." Utilizing this margin, the commentary suggests that structures designed to the standards have only a low probability of collapse for shaking considerably larger than the 500 year return period, up to intensities with return periods of 2500 years.
   The last sentence of the first paragraph of 4.3.1 states that buildings are designed "to survive" earthquake ground shaking with returns of 500 years. Although the technical details of design are based on that shaking level, the commentary discussed above suggests that buildings are intended to "survive" much larger shaking. Similarly, the second paragraph says, "total collapse…, while is can never be ruled out, is not expected at ground shaking corresponding to the design level." The "design level" is usually taken as the 500 year return, as is implied in the first paragraph of 4.3.1. Again, it would be more consistent with the commentary to the standard to say "total collapse…, while is can never be ruled out, is not expected at ground shaking corresponding to 1.5 to 1.8 times the design level," or "to ground shaking with a return period of 2500 years."

b. Article 4.3.2, 4.3.3: These articles are important to put the ground motion intensity of the various events in context, which is not at all straightforward, except for persons well versed in seismic design concepts. Unfortunately, this article has several editorial errors which make the explanation more difficult to follow.

   Article 4.3.2 is essentially describing Figures 4.4 (a) and (b) but the last paragraph (page 19) is not consistent with these figures:

   "The red lines represent design levels used for Christchurch buildings in 2010, 1984, and 1976. The green line represents the 1965 standard…"

   Figure 4.4 (a) indeed shows design levels for 2010 (actually 2004, but applicable in 2010) and 1984 in red, but the 1965 standard is not in green (the green line is EQ2: CHHC (N01W)). Figure 4.4(b) shows the design levels for 2010 and 1984 in red but also shows the current 2500 year return motion in red. The 1965 standard is not shown in Figure 4.4 (b). The green in this figure is EQ2:CHHC (S89W).

   Article 4.3.3 is essentially describing Figures 4.5 (a) and (b). The second paragraph indicates that the

   "curved red line represents the design level for most modern buildings in

Christchurch—the 2004 standard."

This is true for Figure 4.5(a) but the primary curved red line in Figure 4.5 (b) is the 2500 year motion. The 2004 standard is a faded, hardly visible red line that is not labeled.

Finally, the last paragraph of 4.3.3 on page 22 states:

"The exceptional demands of the 22 February 2011 event are clear from Figure 4.5 (b) where the grey line is well above the prominent red line (representing design for a one in 500-year ground shaking level to the 2004 standard)."

Actually, the grey line is not well above the prominent red line in the figure, but the two fit together fairly well. But, the prominent red line in fact represents the 2500 year motion, not the 500 year ground shaking level from the 2004 standard. This is an unfortunate editorial error because this important point concerning the exceptional demands of 4 February could be lost to the casual reader.

The perception of the differences between the September 4 and February 22 shaking intensities is also minimized by the fact that the comparison figures (Figure 4.4 (a) and (b) and Figures 4.5 (a) and (b)) are drawn to significantly different scales, each with a predominant and similarly shaped red response spectrum curve. However, the two red curves are very different, the one in the (a) figures being the 500 year return spectrum and the one the (b) figures being the much larger 2500 year return spectrum. The figures have been digitally manipulated to approximately the same scale in Figure 1 below and the visual impression is very different.

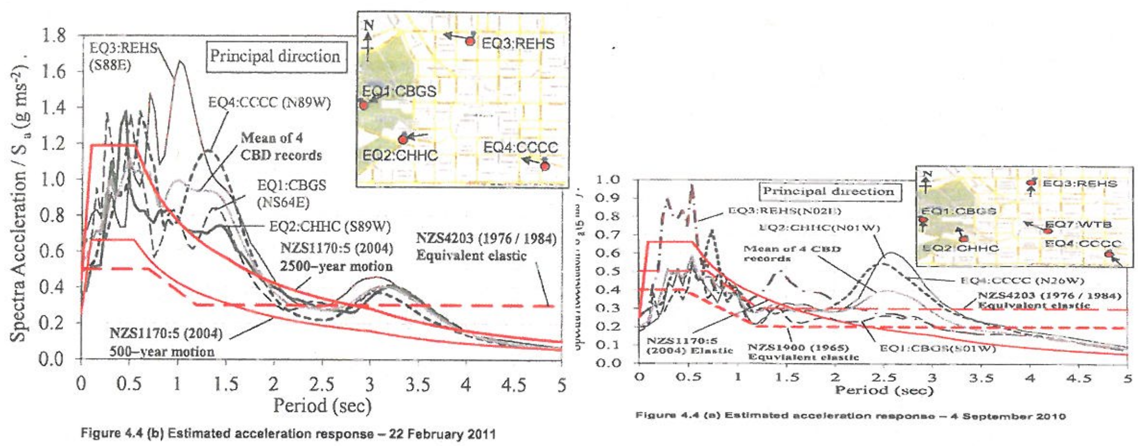

Figure 1: Figure 4.4 (a) and Figure 4.4 (b) drawn to the same scale

Chapter 5.0 Pyne Gould Corporation Building

The CR and the EPR chapter are thorough and clear. As indicated in Article 4.3.5, an inelastic time history analysis was run of the structure, which is a computer simulation of the effect of shaking on the structure over time. Although the exact shaking at the site is unknown, other recordings in the CBD would be representative, and such an analysis is the closest simulation of the earthquakes that can reasonably be obtained. Still, results are subject to assumptions made in modeling the building and could be affected by differences in actual shaking at the site compared to that obtained elsewhere.

There is no question that the central tower failed between level 1 and 2, causing large lateral displacements. The small concrete columns and the steel props added as a partial retrofit could not sustain these lateral displacements and lost their ability to support the floor girders. At some point, many of the floor girders also were pulled away from their support at the tower walls. These losses of vertical support resulted in partial or complete collapse of the floors one upon the other. Given this agreement on the global cause of collapse, I offer the following additional discussion, which does not necessarily challenge any conclusions.

a. The report concludes that the failure of the tower was flexural, with compression crushing in the east wall creating an overturning motion to the east, which is consistent with reports from the field. It is noted, however, that the two northernmost east-west oriented cross walls in the tower had large penetrations, little or no trim reinforcement, and were extremely weak in shear. Preliminary calculations indicate that the capacity of the tower in shear and in flexure between levels 1 and 2 were similar. A shear failure in these walls would produce very similar field conditions. The failure mode in computer simulation could be affected by modeling assumptions and the exact signature of the shaking signal used for analysis. In any case, the *possibility* of a shear failure may only be important to more specifically identify critical structural weaknesses in other older lightly, centrally reinforced shear wall buildings.
b. Floor framing of the building included large floor girders spanning 10.16 m (33'-4") with tributary area of 5.08m (16'-8") supported on the thin 203 mm (8") centrally reinforced core walls. The floor girders were aligned at each floor. There were no pilasters supporting the girders, no confinement ties within the wall and no added reinforcing bars under the girders shown on the drawings. This detail for support of the girders was not highlighted in the report, although the loads from the girders were probably modeled in the analysis. It is interesting to speculate if the presence of some kind of column element at every girder (5 m on center) would have delayed or prevented compression failure of the wall.
   In addition, it was noted from the drawings that the continuity reinforcement connecting the girders to the tower walls or the interior floor slab of the tower was poor, almost non-existent. Given slightly better performance of the tower, could these girders have pulled away from the tower support and collapse the floors?

c. The east wall of the tower was discontinuous at level 1 at the north end of the tower. That is, the wall at the ground level for the northernmost 5 m of the east wall was offset over 1 m to the east of the main wall line above. Therefore, that portion of the wall would have no stiff support for the tower flexural flange forces. Support for such compression forces would have to come from near the ends of that discontinuity at lines b and c. It is possible that the crushing (or buckling) of the east wall started directly over those hard supports (as discussed in Article A5.8 of the CR), most likely first at the corner b-E. The report does not describe how this discontinuity was modeled, if at all. Of course, if a solid wall with no discontinuities failed in compression in the model, the discontinuity, if modeled, would probably make it worse.

d. The installation of perimeter steel props was apparently previously recommended by the evaluating engineer because the rotational capacity was barely at the 1/3 NBS, and at greater rotations, vertical support could be lost (Article 8.1.2). However, the Beca analyses "do not predict failure of the columns until after the shear-core fails." This suggests that there may be room for improvement in standards regarding allowed deformation in gravity columns with various configurations of reinforcement to achieve more consistency in evaluations. In the U.S., allowable drifts in older concrete buildings are often controlled by gravity columns unless the columns are retrofitted (usually by adding shear strength and confinement). It is not clear to me if this is a widespread problem in New Zealand. In any case, the props were not installed with the expectation that they would support the perimeter gravity load after a complete failure of the building lateral system (the tower).

e. Both the CR and the EPR indicate that "the benefits of an active approach to the screening of existing buildings for critical structural weaknesses (CSW) have been highlighted." I totally agree. Although there are lessons from these earthquakes that can be used to improve design provisions for new buildings, larger risk reductions can be obtained by refining evaluation and extent of retrofit of older buildings. It may not be feasible to require full engineering analysis and evaluation of every older building in New Zealand. Therefore, the concept of CSW is a good one, already in use in *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* by the NZSEE. But the effectiveness of CSWs to efficiently identify seismically dangerous buildings is dependent on the degree of specificity that is possible. For example, in the PGC building, what was the CSW?
    1. The mere presence of a lightly, centrally reinforced shear wall?
    2. The support of large concentrated loads on a thin wall without some kind of column elements?
    3. The weakness of the tower in flexure? in shear?

f. The report indicates that the building's capacity was somewhere in the 33% to 50% NBS by normal assessment techniques, apparently as shown in Appendix A4 and A5 (Figure A5.6.2).* I could not find a description of the point where the pushover abruptly ends, although that might be the crushing of the wall. The apparent discrepancy between performance on 4 September (no or little damage) and Figures A4.7.1 and A5.6.2 may be the damping used in the spectral plots (assumed here to be 5%). The Capacity Spectrum approximate method significantly reduces the spectral values for assumed increased damping as damage occurs. The ratio of displacements would be considerably reduced if such reductions were taken. In Figure A4.7.1 the lack of damage perhaps could be more easily explained, and in Figure A5.6.2, the %NBS may be increased to 60% or more.

*The figure numbers in Appendix A4 often do not agree with the text. There at two Figures A4.6.2. I think Figure A5.6.2 is referred to as Figure A5.2.1 in the text.

The reports suggest that the NZSEE assessment guidelines be reviewed so that buildings of the PGC type are identified as potentially poorly-performing in earthquakes. Notwithstanding the tragedy of the collapse in February, did the lack of damage to the building in September indicate that the 50% NBS assessment was not high enough given the similarity of the motion to NBS standards? I recommend that before assessment guidelines are revised, the goal of the program for earthquake prone buildings be clearly identified. One can argue about the details of the characteristics of shaking in February, but it would appear to be greater than 100% NBS. The PGC building survived something close to 100% NBS in September. The brittleness of older concrete buildings and the susceptibility of their performance to suddenly change from "adequate" to "dangerous" with small changes in characteristics of ground motion have already been pointed out in a paper by Pampanin, Kam, Tasligedik, Gallo, and Akguzel. But are buildings in Wellington or other cities going to be assessed for motions greater than 100% NBS? If the earthquake-prone policy is intended to find and mitigate only the "worst of the worst," then poor performance and even collapse of some buildings should be expected in 100%NBS level shaking.

Chapter 6.0 Hotel Grand Chancellor Building

The EPR chapter is clear and generally consistent with the conclusions in the CR, but the CR lacks explanation of what analysis was completed and the sources for both forces and displacements stated in the report and in Appendix F. I am in general agreement with the conclusions, particularly the summary in Article 6.1 of the EPR, but additional description of the analysis portion of the investigation in the CR would have been useful.

a. An important issue that is not emphasized in the CR or the EPR is the bi-directional effects on Wall D5-6 as noted below. Engineers should identify structural components subject to significant seismic actions in both horizontal directions and carefully consider potential consequences.

    1. Shaking in both directions would impose vertical loads on the corner elements of the upper moment frame, eventually supported by Wall D5-6
    2. Shaking in the East West direction would induce story shear related "overturning" loads from the cantilever transfer beams at level 11-12.
    3. Shaking in the North South direction would induce chord force loads in Wall D5-6 from normal lateral load resistance

4. Shaking in the vertical direction could have generated amplified dynamic response from either the upper floor cantilevers, or the cantilever transfer beams. Such vertical dynamic response and its damageability is not well understood, but is probably more problematic at cantilevers (although I believe the current code requirements to consider vertical accelerations on cantilevers is due to lack of redundancy in the cantilever itself, not the concern of supporting amplified reactions).

b. A similar issue that affected the performance of this structure is the potential for serious degradation of the vertical load carrying ability of an element due to seismic damage. According to the report, Wall D5-6 was supporting 100 m^2 from 21 floors and was also an important lateral force resisting element.

c. There is a minor disagreement between the EPR and the CR regarding code compliance. The EPR states,

"the structural design appeared to be compliant with the codes and standards that were applicable when the structure was designed. However, for the failed wall D5-6, it does appear that there were some design assumptions that may have contributed to the failure" (page 34)

While the CR (page 24) states,

"Comparison with NZS 3101:1982 suggests that the D5-6 shear wall exceeded the recommended slenderness ratio, which leads to the deduction that the tendency to buckle may have exacerbated the propensity to failure…"

The difference between a "design assumption" and "exceeding the recommended slenderness ratio" is probably a subtlety, but if the level of compliance with applicable (or current) codes is of interest, it should be noted. It also is interesting to note that a very similar wall, D10-11, did not fail. The building configuration above wall D10-11 was almost identical to that above wall D5-6 except the cantilever transfer beam at line 10 apparently was not full floor depth. However, there were several other important differences. Wall D10-11 was considerably shorter that D5-6 at the ground level--3m as opposed to 5.1m—and had a return wall on line 11. In addition, Wall D9-10, although having very similar loading to D5-6, inexplicably was shown on the drawings with considerably more boundary zone reinforcing at line 10 with 12-D32 plus 6-D20 with confinement stretching over a 1000 mm length as opposed to only 4 D24 in a 200 mm confined length in D5-6. Actual in-situ reinforcing of D9-10, including the splice detail of vertical wall bars, or other possible differences with D5-6 are unknown. Perhaps more insight into the failure of D5-6 could be gained from study of D10-11.

d. Both the CR and the EPR suggest that the precast stair failure would not have occurred without the major displacement caused by the failure of wall D5-6. However, the clearances in the stair support detail (given the probability that only one end would slip) and the projected building drifts were very similar to those projected for Forsyth Barr, which, given potential compression shortening and uncertainty in calculating displacements, was judged sufficient to cause loss of end support. It appears that drifts were estimated in the Grand Chancellor using the average of the displacement spectra for the 4 motions recorded in the CBD and the first mode shape obtained from ETABS. Using this method to estimate drifts on any one floor, especially for a structure with a major vertical irregularity and higher mode participation, would seem highly uncertain. In my opinion, the stairs may well have lost end support without the main failure of wall D5-6. This is only important because it further emphasizes the importance of providing adequate drift allowance for this type of stair both for new buildings and for checking existing buildings. This issue has already been covered by a DBH Design Advisory, but I recommend that the support detail used in the Grand Chancellor not be assumed to be adequate.

e. The discussion in Article 5.1, *Response Spectra*, particularly the tables on page 16 and Figure 7, are apparently based on consideration of the first mode displaced shape and spectral displacements demands. In my opinion, first mode assumptions for a building this tall are not appropriate, particularly one with a major vertical irregularity.

   The apparent lack of damage in September, as discussed on page 15, may have been due to the fact that the strong demand was only at a narrow range of periods about equal to the building's first period. Any softening would move the structure off the peak and significantly reduce demand. On the other hand, the higher peak demand in February is at a period longer than the building's and the structure may have softened and moved into higher demand.

f. The analysis of Wall D5-6 in Article F.1 also requires additional explanation. The introduction states that the structure has been modeled in ETABS to determine periods and displacements under code loadings. This may have been useful, but how the results were used is not described. The derivation of Axial Actions in Table F1.1 is not described. For example,

   1. The source of the entries for Seismic Over-strength at 6D and 5D is not given, but it is unclear why both are not plus-minus numbers to be combined with gravity loads in the first row. Use of axial loads derived from the full limit state of 14 floors of moment frames in both directions would appear to be overly conservative.

   2. The moment in article F1.2 is an in-plane code moment with an over-strength factor. Use of a single over-strength factor in such a complex load path is questionable. In addition, it appears that the moment from unbalanced vertical loads on the wall would not be included in that analysis, which even for D+L gravity loads are large ((8500kN-6300kN)*4.9m/2 = 5390 kNm). Unbalanced loading from other sources—e.g. seismic over-strength beam shears—would increase this moment.

It is difficult to judge the capacity comparisons in Articles F.1.5 without a better understanding of the sources of the magnitudes of loadings used.

g. I have several comments on the bulleted list on page 24 and 25. The bullets are not numbered, but I will identify them counting from the top.

    1. Third bullet: I don't understand: all earthquake shaking is in 3D and would have components aligned with the major axis of the structure regardless of its orientation.
    2. Tenth bullet: The report does not document any field evidence of tension yielding. A reasonable combination of loads to cause such yielding is not given in Article F.1.

h. The Recommendations in the CR are identical to those in the EPR. I have several comments related to the recommendations.

    1. As discussed in comment a, an important lesson from the response of this building is the importance of bidirectional loading—in some structures. I suggest adding a recommendation to improve design methodology for this loading case.
    2. Second bullet: Design Rigour for Flexural Shear Walls. As noted in comment b above, particular attention also should be paid to walls that are subject to high axial gravity loads.
    3. Fifth bullet: I agree but I am not sure how this is a lesson from this building. Does this refer to the wall-beams?
    4. Sixth bullet: Is this referring to ratcheting in the vertical direction of a cantilever due to the cyclic overturning action of the supported moment frame? Or the effect of frame elongation on the cantilever support elements? Although these may theoretically be issues, it is not clear how they came into play in this building.

Chapter 7.0 Forsyth Barr Building

This report is thorough and clear. I have no comments on the report. Parenthetically, I do not know why Figures 5.5 and 5.9, the Vertical Acceleration-vs-Displacement Response Spectra plots are included in the report. Such plots for vertical acceleration are rarely shown and the figures are not referenced in the text (at least that I could find).

I also note that a Design Advisory on the stair issue has already been issued by DBH.

Chapter 8.0 Principal Findings and Recommendations
My comments on this chapter are by article as noted below.

a. Article 8.2 Findings

1. Item 2. *Vertical accelerations*: High vertical accelerations have been mentioned in several reports as a possible contributor to damage. I agree that the extent of contribution is difficult to quantify, but perhaps additional research should be undertaken to improve quantification and design methodology.
2. Item 3. *Duration of Shaking*: It is my understanding that a larger event with a greater duration would also be sourced at distance so that shaking intensities similar to 22 February would likely not be coupled with a longer duration.

b. Article 8.3 Priority Recommendations

1. Item 2 *Walls and Columns:* High vertical accelerations are again mentioned. It is unclear how the vertical acceleration spectrum currently specified in the standards is to be used for design. I recommend that additional design guidance be developed.

   Concerning item b, it is not clear to me how a wall can be *designated* as not part of the earthquake-resisting structure. Either a wall is isolated from the structure (a nonstructural wall) or connected, in which case it will resist lateral loads.
2. Item 4. *Lightly-reinforced shear walls*: As pointed out in my comments on the PGC building, the core tower had several deficiencies. I agree with the last paragraph of this item that suggests guidance be given as to how the specific vulnerabilities of these walls can be worked in to current assessment guidelines.

c. Article 8.4 Other Recommendations

1. As discussed in item 6 a above, I recommend development of additional guidance for design of elements subject to bidirectional seismic loading.
2. Item 4 *Cantilevers and vertical accelerations*: As noted in 6.a.4 above, previous concern about vertical accelerations applied to cantilevers may be related to lack of redundancy in a cantilever for vertical load support (the same could be said for elements supporting gravity load in tension—which this building also has). There was no mention in the report of damage to the cantilevers mechanism itself, although the tension hanger mechanism on line 8 had an apparently lap slippage, and the cantilever on line 6 had cracking at the wall-slab connection. This damage was not discussed in detail but it was suggested to be related to the excessive vertical displacement of D5-6, rather than from amplified vertical loads

from vertical acceleration.
Additional design guidance is probably also needed to properly comply with current requirements for cantilevers.

3. Item 5 Stair supports: This item seems to be a duplicate of 8.3.1.
4. Item 6 *Structural integrity*: I recommend that this item be changed to a *Priority Recommendation*.
5. Item 7 *Earthquake prone buildings*: In my opinion, there are potentially far more lessons to be learned from the Canterbury swarm of earthquakes about existing buildings that about the design of new ones. However, the placement of complete discussion of this subject in Item 7 under Article 8.4 *Other recommendations*, with a reference to this article in Item 3 of Article 8.3 *Priority Recommendations* makes the Panel's recommended priority for research and improvements in this area unclear. More specific comments on Item 7 follow.

    i. Is the first sentence intended to say "that the PGC Building could have been classified as <u>not</u> earthquake-prone…"? It is not clear how a finding that the building <u>was earthquake-prone</u> would cause the Panel to consider changes in the legislation on earthquake-prone buildings.

    ii. I totally agree with items a., b., and c. of this discussion.

    iii. Item d: There has been much discussion of the use of 33% NBS as a trigger for becoming earthquake prone, not limited to the potential misinterpretations discussed in item a of Article 8.4. There are clearly buildings that could be scored over 33% NBS that could collapse in strong ground shaking. The problem, and it is an international problem, is how to efficiently identify collapse hazard buildings in the overall building stock. Few national or local governments can succeed in requiring costly evaluations of all older buildings. It is recognized that 33% NBS was set to capture, with reasonable probability, the worst of the worst. But, despite a tremendous effort by knowledgeable engineers and academics, the NZSEE assessment guidelines may not be adequate. Certainly the US evaluation standard, ASCE 31, is not adequate, because it is far too conservative and fails practically all older buildings. Such a standard leads policy makers to believe that 70%-80% of all older buildings need retrofit—an unrealistic goal for an active mitigation program. Such improvement may be achieved over a long period of time due to normal replacement of the building stock and/or a policy or triggered retrofits as suggested in item 8.4.7.f.

    The "adequacy" of an assessment methodology is also largely a matter of policy. What risk is acceptable? For a brittle buildings, the concept of checking safety for a lower intensity ground motion that is more likely to occur has the flaw, as pointed out by Pampanin, Kam, et al., that acceptable performance could change to partial or complete collapse for a slightly more intense motion. Perhaps the best system would integrate a building fragility over the local hazard curve, which would encompass the probably of collapse for all potential ground motions.

In the U. S., the Federal Emergency Management Agency (FEMA) and the National Institute of Science and Technology (NIST) are pursuing development of methods to more efficiently identify collapse prone concrete buildings, which are considered to be a major threat to life safety in an earthquake. Although interim improvements to evaluation techniques may become available, a complete new evaluation technique for these buildings is not expected for 5-10 years.

New Zealand could pursue the expansion of the concept of Critical Structural Weakness (CSW) in the assessment guidelines, but, in my opinion, a list of CSWs would be needed that is greatly expanded over the current Table 3.4.

iv. Item e: The national interest and inertia towards mitigation of seismic risk generated by the tragedy in Christchurch should not be lost and public education programs, including development of regional seismic scenarios and drills should be promulgated.

I hope the Royal Commission will find these comments useful. I will be happy to provide clarification on certain points, or to discuss items by telephone or email.

Respectfully Submitted

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