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Royal Commission of Inquiry
into Building Failure Caused by the Canterbury Earthquake
Level 1
Unit 15 Barry Hogan Place
Addington
Christchurch

2 March 2012

Transmitted by email to: canterbury@royalcommission.govt.nz

Attention: Commissioners

Dear Sirs

Submission concerning SESOC Practice Note "Design of Conventional Structural Systems following the Canterbury Earthquakes"

Introductory comments

This submission is made by Beca Carter Hollings & Ferner Ltd, on behalf of the Beca Group of companies.

In their email of Friday 24 February, SESOC advised the above document has been uploaded to the Royal Commission website and that comment is sought, with submissions closing on the 2nd of March. This constitutes a very short period of time within which to review the document and furnish comments upon it. We have endeavoured to do so, but wish to state that the very short timeframe available has prevented in-depth analysis and considered comment concerning the potentially far-reaching implications of this document. Therefore, we have restricted our comments to only a few key issues at this stage. We have commenced, but not completed, our evaluation of the specific issues and recommendations contained in the Practice Note. A progress copy of this evaluation, which has not yet been verified internally, is attached and marked “draft”.

The Royal Commission and industry participants such as: the Department of Building & Housing (DBH), the Engineer’s Advisory Group, and SESOC are to be complemented for their endeavours in communicating key lessons from the Canterbury earthquakes in an expeditious manner. The timely publication of information appraising design professionals of the key issues observed and guidance concerning recommended practice (eg: DBH Advisory concerning improvements to stair detailing), are certainly appreciated by the engineering fraternity. However, the publication of **interim Design Standards** obviously has significant implications for the wider construction industry and therefore warrants rigorous evaluation, peer review and industry debate. Accordingly, we have elected to make a number of comments concerning the process followed in the generation and subsequent public release of the Practice Note. A number of these comments pertain to the actions of the SESOC Management Committee, and we have passed a copy of this letter to them for their consideration.

Background to the SESOC Practice Note

We understand the main intent of the Practice Note is to introduce some interim measures in an effort to mitigate the risk of new building designs being deemed non-compliant in anticipation of code changes following the outcomes of the Royal Commission's investigation. Thus, the Practice Note effectively seeks to both anticipate the likely findings of the Royal Commission and how these findings may ultimately manifest in revisions to the design standards.

Based on feedback from Beca staff serving on the SESOC Management Committee, we understand the origin of the document was one of the larger New Zealand structural practices. It is our understanding that following a request from the Royal Commission to prepare such a guideline, it was offered to the SESOC Management Committee for consideration. The originating firm are to be complemented for the (no doubt) considerable effort expended in generating the initial draft and implementing subsequent revisions to incorporate feedback from members of the SESOC Management Committee and international peer reviewer (we also understand, that the Royal Commission recommended external peer review of the original draft, and that this was provided by Mr Bill Holmes). Thus, it appears the genesis of the document is largely confined to the observations and subsequent conclusions drawn by a small number of contributors, and has been subject to only limited review. It is also not clear whether those tasked with reviewing the earlier drafts of the document have reviewed and approved the final version.

Status of SESOC Practice Note

We note the document is <u>not</u> currently marked "draft", and that it is annotated "First Public Release – 21 December, 2011". Thus, it could be inferred the Practice Note has been operative since that time. However, it appears the bulk of the profession have only recently become aware of its existence.

In our view, the status of the Practice Note is not entirely clear. The footer states "interim Design Standards" suggesting the document is intended to supercede aspects of the current New Zealand design standards. Furthermore, it states the SESOC requirements "should be considered mandatory to achieve the level of performance that the NZBC requires".

Section 1.4 of the Practice Note states "This practice note has been prepared by SESOC for general distribution, for the guidance and assistance of structural engineers involved in particular in the preparation of designs for the Canterbury area, although the observations herein are equally applicable to the whole country". Hence, there is a strong inference the recommendations should be followed throughout New Zealand.

SESOC's email of 24 February clarified the document is not intended for the assessment of existing buildings. However, this is not readily apparent when reading the document itself.

In light of the above, we consider there is an urgent need for the status of the document to be clarified. It is our view the document should be clearly denoted "Draft for discussion".

Process

It appears the process that lead to this document being published as a SESOC Practice Note has been accelerated and potentially subject to less rigour than usual. It is our view the document should be subject to a greater level of review and debate within the profession before public release.

As noted in our introductory comments, the document has potentially far-reaching implications for property owners, Territorial Authorities, the DBH, the NZ Standards organisation, developers, designers, product suppliers, constructors and the general public – particularly as it purports to establish new design standards. In light of this, we consider it important the appropriate technical debate by a properly constituted group of experts takes place. Furthermore, those parties likely to be impacted by the proposed changes should be provided with appropriate opportunity to understand and evaluate the implications of the proposed changes to the design standards, and participate in the discussion.

In essence an appropriate phase of industry consultation is required prior to public release of the document. Given the implications of the changes proposed, the process followed should be similar to a revision to a design standard.

Whilst acknowledging the driver to issue guidance in a timely fashion, we consider it vital that an appropriate process is put in place. For example, SESOC could adopt a process including the following:

- Request the Royal Commission withdraw the document from its website.
- Denote the document “Draft for discussion”.
- Circulate the draft to SESOC members and related technical groups for comment (it is likely several revisions will need to be circulated).
- Provide accompanying explanatory notes as required.
- Agree a process for ratification, publication and distribution of the final version, along with the status of the document.

We anticipate wide consensus may be reached relatively quickly concerning certain aspects of the Practice Note (eg: stair detailing recommendations), whilst other issues will engender considerable debate. Thus, it may ultimately prove necessary to adopt a staged approach to the release of the recommendations.

Comment upon SESOC Practice Note content

We have organised our comments concerning particular aspects of the SESOC Practice Note into broad categories of related issues. The section numbers referenced relate to the SESOC Practice Note. As noted above, time constraints have precluded us commenting on all sections in the Practice Note.

Building Code Philosophy

It appears some of the requirements may change the intent and philosophy of the Building Code and relevant structural Standards, particularly pertaining to damage limitation and to a collapse (Maximum Credible Earthquake) limit states.

The impacts in Christchurch structures are considered by some to be beyond societal expectations given the rarity and size of the event on the 22 February. However, it is not really up to just engineers to make changes to code performance objectives (e.g. in limiting damage or achieving higher performance) without inputs from policy makers, Department Building and Housing, economists and the wider industry and community.

Specific Examples:

- We observe that requirements 2.1 and 2.3 effectively introduce a Collapse limit state by introducing a "sufficient resilience" test for building design, failing which a significantly higher design force and displacement will be required. No quantitative or qualitative parameters are defined for "sufficient resilience". The current New Zealand Loading Standard NZS1170.5 has implicit requirement for "no loss of structural integrity in either structure or part" and margin of protection against collapse. However, NZS1170.5 recognises the difficulty in predicting resilience or collapse reliably.
- We observe that some of the requirements (eg 3.2 to 3.7) and recommendations (eg 4.1) appear to be intended to limit damage to the building or elements of the building under the ULS. This is a significant change in design philosophy which we consider should be debated widely and the implications of such a change investigated, including cost implications for example.
- Capacity design requirement for certain categories of structures (in Requirement 2.1 and 4.2) is an ideal requirement. The DBH Expert Panel report made similar recommendations. However, the cost and practical implications of implementing capacity design principles for all structures may not be justifiable.

Cost implications

In our opinion, many of the requirements and recommendations are likely to have significant implications on the overall construction cost and value of existing buildings. Without careful consideration and research of the effectiveness of the proposed requirements, these 'new' regulations may ultimately prove expensive and inefficient. The economic impact on devaluing existing buildings by the virtue of non-compliance to these new requirements is possibly not thoroughly analysed.

Some specific examples:

- We question whether some of the recommendations outlined in the document will be achievable for "little extra cost" as stated in Section 1.2. For example: Recommendations 2.2, 4.1, 5.1, and 6.1.
- Requirements 2.1 and 2.3: Without clear interpretation of the intent of these requirements, these requirements may effectively increase the design loading by 10% to 40%.

- Requirement 2.5: The cost of compliance of the proprietary system to an arbitrary testing requirement may out-weight the benefits gained. We consider a thorough and consistent testing protocol for propriety system could be developed for structural products.
- Requirement 3.4 inherently increases the design loading for reinforced concrete walls without confining and anti-buckling reinforcing (e.g. current nominally ductile detailing) by 150%.
- Requirement 3.6: Fracture of boundary reinforcement in lightly-reinforced concrete walls can be due to a range of technical reasons: minimum reinforcing, near-fault impulse loading, excessive flexural demand, etc. Requirement 3.6 effectively increases the minimum steel reinforcing by 60% to the NZS3101:2006 requirement (Clause 11.3.11.3). Further research is required to quantify the improvement in seismic performance due to Requirement 3.5.

International benchmarking

We consider any provisions which depart significantly from the current design standards should be carefully evaluated against international best practice. Some specific examples that appear to depart significantly from current international best practice are:

- Recommendations 4.1 and 5.1 in limiting the achievable design ductility for moment-resisting frames appear to contradict most if not all international seismic codes.
- Various requirements which impose a higher bound limit of 1.5 and $1.5/S_p$ times the force and displacement demand on critical elements are also appear to be arbitrary when compared to international practice. Resilience or performance-based design perhaps can be achieved with other more direct means.

Inconsistent observations and lessons

There are several instances where the observations and lessons stated as the drivers of the requirements and recommendations are inconsistent with our experience in Christchurch and other published literature. Some examples:

- Recommendation 2.2: Stiff lateral load resisting systems have not been shown to limit non-structural damage in Christchurch. Available damage statistics (e.g. Baird et al, 2011) and anecdotal observations generally indicate consistent distribution of non-structural damage across structural form and storeys. Any structural form, properly designed and detailed, can generally be designed to achieve a inter-storey drift limit, which in turn limits non-structural damage envisioned in Recommendation 2.2.
- Recommendation 3.2, 3.4 and 3.5: The web buckling of modern ductile walls as shown in Figure 4 is a unique case observed in the Christchurch earthquake. It should be noted that the buckled web wall shown in Figure 4 is part of a V-shaped flange wall; as such the observed web buckling of the wall may be due to other reasons those identified (Elwood, 2012).
- Requirement 3.7: Walls with lumped reinforcing at the end of the walls will yield earlier and may have lower ultimate displacement / deformation capacity when compared to walls with uniformly distributed bars (for the same moment capacity, reinforcing content and ductile detailing) Lumped reinforcing wall will have lower extreme fibre compressive strain, but lower serviceability and damage avoidance limit (in terms of earlier bar yielding and concrete cracking).
- Section 16 : “for multi-storey buildings, there were no observed cases of complete loss of panels” is incorrect as there have been several cases of collapsed panels (e.g. former Physical

Science Library at the University of Canterbury, Anderson building on Lichfield St, MedLab building on Kilmore St) (Kam et al, 2011).

Reasons for building demolition

Our observations are that the reasons for buildings being demolished are many and varied. A large number of buildings are demolished not necessarily because they were not repairable, but rather because of insurance terms, non-structural repair costs, and other commercial drivers (e.g. loss of tenancy, etc). For instance, insurance policy wording such *as "return to same condition as new using current methods and materials and complying with current building regulations"* have prompted many building owners to look for a new building to replace an existing one rather than repairing an existing damaged building. Our observation is that Christchurch is unique in the level of demolition following a damaging earthquake compared with other countries such as the USA following the Loma Prieta and Northridge earthquakes.

The uncertainty surrounding the reparability of ductile plastic hinges (e.g. in ductile concrete beams of moment-resisting frames) is an unresolved issue. However, overseas experience suggests that ductile plastic hinges, designed to modern standard, are generally repairable.

Justifying the changes to design requirements due to the level of demolition in Christchurch appears questionable. Furthermore, while a number of items in the document will arguably improve seismic performance, most will not likely entirely eliminate the effect of "*return to same condition as new*" insurance policy clauses.

Beca observations of Christchurch building performance

Our observations following inspections of many buildings and facilities in the Christchurch area identified a number of issues which we note are not addressed in the Practice Note. Examples include:

Determination of ULS drift and displacement

The Practice Note rightly raises the importance of assessing and determining the Ultimate Limit State (ULS) inelastic drift demand. The ULS drift is a crucial parameter to many of the proposed requirements (eg: Requirements 2.3, 2.5, 3.10, 4.3, 5.3, 8.1-8.5, 9.3, etc.). However the current method of assessing ULS drift in building design relies on elastic analysis and modification factors. The inherent inaccuracy in how the ULS drift is calculated means any design requirements that hinges on this parameter will be equally inaccurate.

An important lesson from Christchurch earthquake is the importance of deformation (curvature, rotation and displacement) and ductility demands on critical structural elements as an earthquake damage parameter. This is inherently built into the NZ Material Standards (eg NZS3101 and NZS3404) but not directly considered in the NZ Loadings Standard (NZS1170). Displacement-based design and non-linear analysis-based design approaches reflect this idea, but neither design approach is commonly used in New Zealand.

Further guidance concerning displacement-based or performance-based seismic design methods appears warranted. We note various existing procedures such as direct-displacement-based (eg: Priestley et al 2007) and non-linear analysis-based (eg: Eurocode 3 and ASCE-41) design approaches are available and are being used in other seismically-active countries.

Structural redundancy for resilience

Requirements 2.1, 2.3, 2.4, 3.10 and 4.2 aim at improving the resilience and robustness of structures which are required to retain their structural integrity at seismic loads beyond their design load.

In American and Japanese standards, provisions for building redundancy are in place to ensure the failure of one or two critical lateral bracing elements would not result in catastrophic collapse of the building. For example, dual frame-wall system will require a minimum portion of the storey shear to be carried by the moment-resisting frames. American Loading Standard, ASCE-7, for example, has a redundancy factor requirement for braced or moment frame and shear wall systems for buildings in regions with high seismicity. Such provision for structural redundancy can perhaps be adapted to New Zealand conditions.

Other key lessons which might require immediate change to design practice

From our experience, as well as from the various technical reports on building performance in Christchurch (eg: DBH Expert Panel report (2012)), there are other key lessons which might require immediate change to design practice. We have not developed any substantial recommendations on these issues:

- Bi-directional loading and inelastic demands: The current Standards rely on specific material standards to account for the effects of bi-directional loadings on individual columns, connections and walls in ductile system. However, existing code provisions may not have accounted for significant bi-direction inelastic demands on ductile systems due to significant aftershocks.
- An upper limit on the axial load ratio on reinforced concrete elements should be imposed to limit compressive/crushing and buckling (local and global) damage. This is in view of typical design practice of encouraging higher axial load contribution on shear and flexural capacities of reinforced concrete elements.
- Earthquake-induced axial loads (from overturning moment and vertical acceleration) are not well defined for wall structures.

We consider that examples of observations of earthquake damage should be widely sought from the profession. These must be prioritised and evaluated in terms of risk and benefit before considering the need to implement any changes in the design requirements. We note this approach was adopted by the California Seismic Safety Commission following the Northridge earthquake resulting in a wide survey of issues observed by many different consultants' academics and associated industry members.

Clarity and Robustness of Recommendations

Some of the requirements and recommendations are unclear in their intent and requirements.

Specific Examples:

- Requirement 2.5 for the acceptance of propriety systems requires the whole system to be tested to a specific displacement demand. The definition of propriety systems and the testing requirement is unclear. Are "ductile" mesh reinforcing, steel bracing or composite deck floors propriety products?

- Requirement 2.3 is also unclear. For IL3 or IL4 buildings, the R=1.3 and 1.8 are to be maintained. Is this on top of the 1.5 or 1.5/S_p multiplier? If the intent of Requirements 2.1 & 2.3 or “sufficient resilience” test is to introduce a “collapse limit state”, it is unclear how such requirements are intended to be met.
- Code Requirement 4.2: Why are only reinforced concrete moment frames to be designed for “resilience” irrespective of its ductility assumptions or design loads?

Conclusions Drawn from Observations

In our opinion, some of the recommendations and requirements put forth in the Practice Note are premature, as there is limited technical and research data to support them. As noted previously, the stated lessons are still inconclusive and some of the recommendations/requirements appear to be inconsistent with international best practice. Considering the significant implications of these requirements, further research and technical review is warranted.

Some specific examples:

- **Section 3:** The various reinforcing detailing recommendations appear to be consistent with the observed concrete wall building damage in Christchurch. Without robust research and experimental evidence, some of the proposed requirements for reinforced concrete walls may prove to be uneconomical and unnecessary.
- **Section 4:** The Practice Note appears to be very “strict” on concrete moment resisting frames, despite noting that the modern RC frames have performed as expected. The recommendations and requirements for RC moment frames appear to be more focussed on damage-mitigation instead of life-safety performance objectives.

  Recommendation 4.1 limiting the design ductility to 1.25 for conventional reinforced concrete moment frames appears to be unduly conservative if assessed against a life-safety performance objective. Anecdotally, most modern-designed ductile moment-resisting concrete frames have performed as expected in Christchurch. To our knowledge, no modern (current code) concrete moment frame buildings have suffered “collapse” or structural failure which resulted in fatalities in the 22 February earthquake, with the exception of a precast concrete spandrel collapse, thought to be the result of construction/design error.
- **Section 5:** Considering the presence of only limited numbers of steel moment resisting frames and steel braced frames in Christchurch, (by comparison to other construction materials) it may be premature to draw any significant conclusions on their seismic performance.

  Sections 4 to 9: The heavy focus on frame elongation and precast concrete floors appears to be inconsistent with the number of observed damage buildings with such issues. Brittle mesh reinforcing, used for precast floor topping diaphragms warrants more relative emphasis.
- Requirement 9.2 appears to discourage, if not prohibit the use of ductile mesh reinforcing for suspended floor diaphragms. However, the seismic performance of “ductile” wire mesh with some level of elongation strain capacity is unclear from the observations in Christchurch, as these are recently introduced product (since DBH Practice Advisory 3 (2006)).

We thank the Royal Commission for this opportunity to proffer feedback concerning the SESOC Practice Note, and would welcome the opportunity to both comment more comprehensively upon the current document given adequate time to do so, and participate in the further development of guidelines to assist the profession and wider industry.

Yours faithfully
Mark Spencer
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on behalf of
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Copy
SESOC Management Committee

Encl.

[Draft] Technical notes for the submission to the Royal Commission on SESOC Practice Note “Design of Conventional Structural Systems following the Canterbury Earthquakes”

File Note

Technical Notes on SESOC Practice Note - Design of Conventional Structural Systems Following the Canterbury Earthquakes

1 Beca Observations

Our observations following inspections of many buildings and facilities in the Christchurch area identified a number of issues which we note are not included in this document.

1.1 Determination of ULS drift and displacement

The Practice Note rightly raises the importance of assessing and determining the Ultimate Limit State (ULS) inelastic drift demand. The ULS drift is crucial parameter to many of the proposed requirements (eg Requirements 2.3, 2.5, 3.10, 4.3, 5.3 etc.). However the current method of assessing ULS drift in building design relies on elastic analysis and modification factors. The inaccuracy in relation to how we calculate the ULS drift means any design requirements that hinge on this parameter will be equally inaccurate.

An important lesson from Christchurch earthquake is the importance of deformation (curvature, rotation and displacement) and ductility demands on critical structural elements as earthquake damage parameter. This is inherently built within the NZ Material Standards (eg NZS3101 and NZS3404) but not directly considered during in the NZ Loadings Standard (NZS1170). Displacement-based design and non-linear analysis-based design reflect this idea, but neither design approaches are commonly used in New Zealand.

Further guidance of the state-of-the-art of displacement-based or performance-based seismic design should be given to the practitioner, with immediate plan for adaptation of such methods in the design standards. Various existing procedures such as direct-displacement-based (eg Priestley et al 2007) and non-linear analysis-based (eg Eurocode 3 and ASCE-41) design approaches are available and are being used in other seismically-active countries.

1.2 Structural redundancy for resilience

Requirements 2.1, 2.3, 2.4, 3.10 and 4.2 aim at improving the resilience and robustness of structures which are required to retain its structural integrity at seismic loading beyond its design load.

In American and Japanese standards, provisions for building redundancy are in place to ensure the failure of one or two critical lateral bracing elements would not result in catastrophic collapse of the building. For example, dual frame-wall system will require a minimum portion of the storey shear to be carried by the moment-resisting frames. American Loading Standard, ASCE-7, for example, has redundancy factor requirement for braced or moment frame and shear wall systems for buildings at regions with high seismicity. Such provision for structural redundancy can be adapted to New Zealand conditions.

1.3 Structural Design Feature Report and Structural Drawings Repository

The availability of construction drawings and structural design feature report in which the intent of the original structural designer is outlined is critical for any pre- and post- earthquake seismic assessment. The DBH Expert Panel report (2012) has recommended a two-staged design feature report as part of the design documentation: building consent and completion of construction. Similarly, Beca has been providing design feature report as part of our design documentation for some years now.

There is existing guidance on design feature report published by the SESOC and the Construction Industry Council. As such, structural design feature report should be made a requirement for non-residential structures, and this can be part of the Practice Note recommendation.

1.4 Other key lessons which might require immediate change to design practice

From our experience as well as from the various technical reports on building performance in Christchurch (eg DBH Expert Panel report (2012)), there are other key lessons which might require immediate change to design practice. We have not developed any substantial recommendations on these issues.

- Bi-directional loading and inelastic demands: The current Standards rely on specific material standards to account for the effects of bi-directional loadings on individual columns, connections and walls in ductile system. However, existing code provisions may not have accounted for significant bi-direction inelastic demands on ductile systems due to significant aftershocks.
- An upper limit on the axial load ratio on reinforced concrete elements should be imposed to limit compressive/crushing and buckling (local and global) damage. This is in view of typical design practice of encouraging higher axial load contribution on shear and flexural capacities of reinforced concrete elements.
- Earthquake-induced axial loads (from overturning moment and vertical acceleration) are not well defined for wall structures.

[List to be developed following further inputs]

2 Loading and Design Philosophy

Key issues:

- Life safety versus damage-control design objectives for ULS.
- Introduction of “Collapse” or “MCE” loading Limit State and the test of “sufficient resilience”.
- Further debate between policy makers, DBH, engineers, and industry is required to refine the performance objectives of new building seismic design.

Requirement 2.1: Capacity design requirement for all structures is indeed an “ideal” requirement. The DBH Expert Panel report (DBH, 2012) on the technical investigation of the collapsed buildings in the Christchurch earthquakes also highlighted the benefits of requirement capacity design approach to the whole building, even in regions of low seismicity.

However, the cost and practicality of implementing capacity-design principles for all structures may not be justifiable. Whether the profession and society is ready to embrace the capacity design requirement for all structures, especially those in areas of low seismicity (e.g. Auckland), is a debatable question. Some structural systems are inherently brittle (e.g. squat walls, space trusses) and capacity design may not be practical. Furthermore, a default of S_p factor of 1 may not achieve the objective of resilience as brittle / nominally ductile can still be designed by 8% strength increase (S_p=1 as compared to S_p=0.925).

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"Sufficient resilience" is set as a condition to meet the requirement. However, no quantitative or qualitative parameters are defined for "sufficient resilience". The current New Zealand Loading Standard NZS1170.5 has requirement for "no loss of structural integrity in either structure or part" and implied margin of protection against collapse. However, NZS1170.5 recognises the difficulty in predicting resilience or collapse reliably.

Recommendation 2.2: Stiff lateral load resisting systems have not been shown to limit non-structural damage in Christchurch. Available damage statistics (e.g. Baird et al, 2011) and anecdotal observation generally indicate consistent distribution of non-structural damage across structural form and storeys.

Any structural form, properly designed and detailed, can be designed to achieve a inter-storey drift limit, which in turns limit non-structural damage envisioned in Recommendation 2.2. One recommendation can be to adopt a displacement-based (either direct or non-linear analysis) type of design approach for significant buildings (e.g. above 6 storeys or IL3/IL4). These approachs however, also require further design guidelines and modification of the existing standards.

Requirement 2.3: "Sufficient resilience" is again defined as the trigger for 1.5 or 1.5/S_p multipliers for design force and displacement respectively. Is "insufficient resilience" refer to brittle or nominally ductile behaviour, or critical primary/secondary structural elements with no redundancy or overall building collapsing at 1.5x design loading or 1.5/S_p design displacement (collapse limit state)?

Requirement 2.3 is also unclear. For IL3 or IL4 buildings, the R=1.3 and 1.8 are to be maintained. Is this on top of the 1.5 or 1.5/S_p multiplier?

If the intent of Requirements 2.1 & 2.3 or "sufficient resilience" test is to introduce "collapse limit state", it is unclear how such requirement can be meet in actual design.

NZS1170.5 Commentary on Clause 2.1 clearly states the basis for Ultimate Limit State (ULS) design – considering "current state of knowledge of the variables and the inherent uncertainties involved in reliably predicting when a structure will collapse." To require building designer to do so – require a paradigm shift towards the "performance-based seismic design", which will require significant changes to design practice and standards.

American codes are gradually moving towards codifying a performance-based design framework which includes damage control and collapse limit states. However, acknowledging the lack of knowledge to achieve reliably damage and collapse – multi-year multi-millions research programme (e.g. ATC-58 and FEMA445) have been in place to develop the technology and guidelines to reliably predict "damage and collapse" limit states.

Requirement 2.4: The consideration of centre of rigidity/stiffness for ductile design/response of inelastic structure appears to be inconsistent with the literature (e.g. Paulay, 2001 and p.336 of Priestley et al, 2007). Research (Priestley et al, 2007) indicates torsionally-restrained building as per Figure 1, generally performs well even with 0.5-2.0x variation of stiffness/strength. Similarly, the scenario in Figure 1 has only been observed in a limited number of buildings (possibly in Clarendon Tower) and requires further research before any firm conclusions can be derived.

Requirement 2.5: The definition of propriety systems and the testing requirement is unclear. Are "ductile" mesh reinforcing, steel bracing or composite deck floors propriety products? Testing propriety systems to 1.5/S_p times the inelastic drift demand imposed by its use and configuration within the structure may not be achievable for every use and configuration.

3 Reinforced Concrete Walls

Key issues:

- The various reinforcing detailing appears to be consistent with the observed concrete wall building damage in Christchurch. It is recommended that some of the requirements to be meet in new building design.
- However, without a robust research and experimental evidence, some of the proposed requirements for most reinforced concrete walls may be uneconomical and unnecessary.

Requirement 3.1, 3.3: Agree.

Requirement 3.2, 3.4 and 3.5: Mostly agreeable. However some of the proposed changes are very different from current New Zealand and international practice. Further research and testing are required.

The web buckling of modern ductile wall as shown in Figure 4 is a unique case observed in the Christchurch earthquake. It should be noted that the buckled web wall shown in Figure 4 is part of a V-shaped flange wall; as such the observed web buckling of the wall may be due to other reasons those identified (Elwood, 2012).

Requirement 3.4 inherently increases the design loading for reinforced concrete wall without confining and anti-buckling reinforcing (e.g. current nominally ductile detail) by 150%.

Requirement 3.6: Fracture of boundary reinforcement in lightly-reinforced concrete wall can be due to a range of technical reasons: minimum reinforcing, near-fault impulse loading, excessive flexural demand. Requirement 3.5 effectively increases the minimum steel reinforcing by 60% to the NZS3101:2006 requirement (Clause 11.3.11.3). Furthermore, the benefit to limit the upper and lower bound of concrete strengths can be debatable. Without further research, Requirement 3.6 can be costly without any significant increase in seismic performance.

Boundary reinforcing fracture has been observed in 2-3 concrete wall buildings designed and constructed recently (post-2000). Considering the extreme seismic load imposed on these nominally-ductile walls in Christchurch earthquake and there have not been any catastrophic collapse, whether the design has performed well or not is a debatable question.

Requirement 3.7: Walls with lumped reinforcing at the end of the walls will yield earlier and may have lower ultimate displacement / deformation capacity when compared (walls with uniformly distributed bars (for the same moment capacity, reinforcing content and ductile detailing). Lumped reinforcing wall will have lower extreme fibre compressive strain, but lower serviceability and damage avoidance limit (in terms of earlier bar yielding and concrete cracking). Priestley et al, 2007).

Requirement 3.8: Introduction of confinement stirrups around Drossbach duct might lead to poor concrete casting around the ducts. Is the confinement steel required for nominally-ductile precast walls?

Requirement 3.9.: No comment.

Requirement 3.10: It is generally agreeable that gravity frame and wall elements should be designed to accommodate $1.5/S_p$ times the drift and ductility demands.

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4 Reinforced Concrete Moment Frames

Key issues:

- The Practice Note appears to be very strict on concrete moment resisting frames despite noting that the modern RC frames have performed as expected. The recommendations and requirements for RC moment frames are more focussed towards damage-mitigation instead of life-safety performance objectives.

Recommendation 4.1: Limiting the design ductility to 1.25 for conventional reinforced concrete moment frame appears to be conservative for life-safety performance objective. Most modern-designed ductile moment-resisting concrete frames have performed as expected in Christchurch.

(Also for recommendation 4.2) It should be noted that no modern concrete frame moment frame buildings have suffered "collapse" which result in fatality in the 22 February earthquake. Partial floor diaphragm failure in Clarendon Tower and Westpac Tower, and precast panels failure at Anderson Building, Lichfield St (1x fatality) are the notable modern reinforced concrete frame buildings with significant damage. However, these buildings have other critical structure weaknesses that lead to the failure of the critical components.

The uncertainty surrounding the reparability of beam plastic hinges is an unresolved issue. Testing literature has shown that ductile beam plastic hinges can be repaired without wholesale replacement of beam reinforcing. It is noted that in Christchurch, a large number of buildings are demolished not because it is not repairable, but rather because of insurance terms, non-structural repair costs, economics drivers e.g. loss of tenancy etc.

Code Requirement 4.2 (and also 5.2 for steel frame): Generally agreeable. However, why are only reinforced concrete moment frames to be designed for "resilience" irrespective of its ductility assumptions or design loads?

Any structure with a lack of redundancy in terms of its lateral load resisting system OR gravity load resisting systems should be designed/detailed for resilience. This includes steel braced system, steel frames, single concrete core-wall system etc. Structural redundancy and/or ductile inelastic mechanism can be introduced

Code Requirement 4.3: Floor diaphragms are required to be detailed to accommodate significant frame elongation for ductile reinforced concrete moment frames. A maximum elongation of 4% of the beam depth for ductile frame and a geometric elongation of 0.5% (of the beam depth) for elastic moment resisting frames have been stated.

The maximum elongation of 4% of the beam depth appears to be higher than the previously published maximum elongation values (range from 2 to 3.7% of the beam depth for 95%characteristic value – Fenwick et al, 2010).

With the exception of Clarendon Tower, no 'collapse' of precast floor or cast-in-situ floors due to frame elongation has been observed in "modern" reinforced concrete moment frame. Westpac Tower and Farmers Carpark are two multi-storey frame buildings with notable damage to the hollow core diaphragm floors due to the use of brittle mesh reinforcing.

5 Steel Moment Resisting Frames and Braced Frames

Key issues:

- Considering only limited numbers of steel moment resisting frames and braced frames in Christchurch, by comparison to other construction material, it may be premature to draw any significant conclusions on the seismic performance.

Recommendation 5.1: Limiting design ductility to Category 2 (ductility = 3) for conventional steel moment frame appears to be conservative for life-safety performance objective.

Requirement 5.2: No comment. Is NZS3404 Amendment No 2 as stringent for steel moment frames as Requirement 4.2 is for concrete moment frames in terms of delivering similar seismic performance?

Requirement 5.3: There is no clear observation from Christchurch earthquakes of frame inelastic elongation or precast floor diaphragms in steel structure. This may arise due to a numbers of reasons, including insufficient data set to make any conclusive recommendations. As such, any 'requirement' may be premature conclusion and unnecessary.

Recommendation / Requirement 5.4: As with other requirements made for steel structure, limited damage observations have been made on the precast floors on composite steel beams. (None published in literature / industry).

Recommendation 6.1: Agree in principle with the damage avoidance detailing for EBFs. However, replaceable active links for EBFs is a relatively new detail. Thus, we seek additional information regarding testing and verification procedure, as well as cost impacts, before widespread recommendation and adoption this detail

Requirement 6.2: Agree.

Requirement 7.1: No comment.

6 Precast Flooring Systems and Floor and Roof Diaphragms

Key issues:

- The heavy focus on frame elongation and precast concrete floors appear to be inconsistent with the number of observed damage buildings with such issues. Brittle mesh reinforcing used for precast floor topping diaphragm, which is a more widespread problem, needs more relative emphasis.
- The prohibition or discouragement of "ductile" mesh reinforcing for suspended floor diaphragm might be premature conclusion as no significant damage of these mesh have been observed in Christchurch.

Requirement 8.1: Agree as part SESOC guidelines on double-tee hanger support.

Requirement 8.2: No comment. It may be worthwhile to recommend changes to the hollow-core flooring production to include top prestressing strands, web reinforcing, mechanical collectors, edge reinforcing etc.

File Note

Requirement 8.3: Rib and timber infill floor have performed very well in the Christchurch earthquakes, in relative to other precast flooring, steel deck and cast-insitu floors. This is not reflected in the text. Requirement 8.3 is also generally satisfied in current practice.

Requirement 8.4: Agree. It should be the current practice to adopt the typical seating details shown.

Recommendation 8.5: No comment.

Requirement 9.1: Agree in principle. The failure or lack of of collector elements (drag beams/bars) has been shown to be one of the contributing causes of the lack of robustness of the CTV building (DBH Expert Panel report, 2012).

In addition to enforcing an upper limit for the strength capacity of the collector elements, it may be prudent to recommend a robust load-path for the diaphragm forces (eg avoid having shear walls outside the building regular plan where a direct load path between the diaphragm and wall is difficult to be established.

Requirement 9.2: Generally agree. Brittle hard-drawn wire mesh should NOT be used as floor diaphragm reinforcing.

Requirement 9.2 appears to discourage if not prohibit the use of ductile mesh reinforcing for suspended floor diaphragms. However, the seismic performance of “ductile” wire mesh with some level of elongation strain capacity is unclear from the observation in Christchurch, as these are recently introduced product (since DBH Practice Advisory 3 (2006).:

Requirement 9.3: Agree.

Requirement 9.4: Agree.

7 Transfer Structure and Seismic Joints

Key issues: Not reviewed in detail.

Requirement 10.1: Agree in principle. Transfer structure for a significant portion of the gravity load should be avoided if possible. It is uncertain whether 1.5 times ULS force or 1.5/S_p times ULS displacement is practical, especially if the transfer structure will result in significant over strength actions on all connected lateral load resisting elements. Providing ‘displacement’ capacity to transfer structure (eg outrigger beam, ground beams) may not be feasible.

Recommendation 11.1: Assessment of ULS drift may be difficult for building design without a computer model analysis. A direct / quick approach to estimate ULS drift / displacement may be useful (e.g. a direct-displacement based design approach). This however requires further guidance/guidelines for widespread application.

It is also noted that structural damage/collapse wall-to-wall pounding and pounding in between modern buildings have not been observed in Christchurch earthquake (Cole et al, 2011). This may arise due to a numbers of reasons, including lack of multi-storey modern buildings built without adequate separation.

Recommendation 11.2: No comment.

File Note

8 Foundation

Key issues: Not reviewed in detail.

Requirement 12.1: Agree. It is not clear what scale of project that geotechnical advice is compulsory / required.

Recommendation 12.2: The recommendation wording is unclear.

Recommendation 12.3: Agree.

Requirement 12.4: Agree in principle.

9 Shallow Foundation

Key issues: Not reviewed in detail.

10 Deep Foundation

Key issues: Not reviewed in detail.

11 Stairs

Requirement 15.1: Agree as per DBH Practice Advisory 13 (DBH, 2011).

Requirement 15.2: Agree in general. The requirement is unclear. The stair structure would be required to be designed to overcome the amount of horizontal force induced by the friction of the stairs' landing bearing.

Requirement 15.3: Agree as per DBH Practice Advisory 13 (DBH, 2011). Typo – Figure 16 instead of Figure 15.

12 Precast Cladding Panels

Key issues: In addition to precast cladding panels, the focus can be on non-structural heavy or glazed façade systems that may result in loss of life in the event of collapse/failure. There have been many observed cases of shattered and complete out-of-plane collapse of glazed facades

Requirement 16.1 & 16.2: Agree. The text "for multi-storey buildings, there were no observed cases of complete loss of panels" is incorrect as there have been several cases of collapsed panels (e.g. former Physical Science Library at the University of Canterbury, Anderson building on Lichfield St, MedLab building on Kilmore St) (Kam et al, 2011).

File Note

13 References:

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