STRUCTURAL ENGINEERING SOCIETY
NEW ZEALAND

PRELIMINARY OBSERVATIONS FROM CHCH EARTHQUAKES

The following is a preliminary list of considerations from the earthquakes, of matters requiring attention. Some of these items may be simply observations of performance that may merit consideration in respect of future Building Code amendments. Others may have more detailed recommendations, possibly including a need for future research:

1. **Underpinning design philosophies**

1.1. Design Basis Earthquake/Maximum Considered Earthquake performance

1.1.1. Use of this terminology (DBE/MCE) has to date been avoided in NZ codes. There is some ambiguity in what MCE actually means and what its relevance is to building design, but if this terminology is to be adopted, it will need clarification.

In particular, it needs resolution with respect to the IL3 and IL4 buildings, where the ULS event is raised (R=1.3 and 1.8 respectively). It is unclear to what extent the added performance is required over and above the ULS in these cases, where the ULS event is now perceived to be close to the MCE (if it is considered that the MCE is fixed regardless of building importance level). Assuming that the same margins of performance are required, this ought to be explicitly stated, noting that most definitions of MCE are not linked to the building importance classification.

1.1.2. Adequate performance in the MCE (or simply larger event than the design earthquake) has been assumed as a consequence of good detailing imposed by the material standards, but there are demonstrably a number of cases where that is not the case. This is specific to form and material. In some cases, it may be necessary to explicitly consider the full MCE drifts, for example in elastic responding structures in non-ductile materials, or possibly in low-damage design structures unless or until these are codified. Deeper consideration may need to be given as to how this may be better expressed in the Building Code or in AS/NZS1170.

1.1.3. In consideration of the higher levels of displacement imposed by earthquakes significantly greater than the DBE, it is clear that although the primary structural elements may be dealt with by the material strain limits in the individual materials standards, there is no explicit requirement for designers to verify that other building elements may achieve the required performance. For example, the following may be considered:

1.1.3.1. Stairs – as an important part of the egress path, these should be considered explicitly for the increased drifts of the MCE event

1.1.3.2. Transfer structure. Given the criticality of transfer structures, they should be explicitly checked for increased actions (including enhanced vertical seismic actions) and displacements.

1.1.3.3. Heavy cladding or non-structural elements on the egress paths. It may not be feasible to ensure that these maintain their full integrity, but it may be possible to at least require some form of fail-safe detailing. For example, to ensure that panel connections have a failure mode that will not result in the panel falling. That could comprise failure of the embedded anchorage in such a way that some reinforcement remains attached within the panel, but not shearing of a bolt or breaking of a weld.

1.1.3.4. Cantilever elements may need to be considered for higher vertical accelerations and displacements, particularly if they are non-ductile parts.

1.2. Capacity design.

1.2.1. Generally, capacity designed buildings have behaved as expected by engineers, demonstrating that where the approach has been followed, it has worked. But it is apparent that societal expectations were not met and this points to a disconnect between what engineers understand as "earthquake-resistant" and what clients and tenants perceive as "earthquake-resistant". This disconnect was clear from the Northridge earthquake in 1994 and has been clear from the Christchurch earthquake series.

1.2.2. However, no measures are in place for subsequent review of remaining life of buildings. Low cycle fatigue in particular is a critical issue for buildings that may have been through a number of cycles of yield, and yet there is no accepted method for its evaluation. This is a very significant research requirement, taking into account the observations of performance in the earthquake and subsequent testing that has been performed in some cases.

1.3. Serviceability performance

1.3.1. Overall levels of (generally non-structural) damage in smaller events have been disappointing, notably in the September earthquake, which caused significant non-structural damage in what may otherwise have been considered a serviceability level event, at least in the CBD.

2. Performance/Design Objectives

2.1. As noted above, there is some indication that SLS limits may be set too low; however in Christchurch city any indications of this from the September 4th and December 26th 2010 earthquakes were destroyed in the February 22nd 2011 earthquake so it is unclear what conclusions can be drawn. The SLS has been raised slightly (from R=0.25 to R=0.33) in the DBH interim advice of 19 May and that decision (plus the raising of the ULS Z factor for Christchurch) should be critically reviewed before being adopted into the Loadings Standard. It should be noted that this may affect designers' choice of ductility. It is recommended that if this is carried forward, it is not at the expense of capacity design or ductile detailing.

2.2. ULS – is it set at right level? For example, if we compare NZS1170 and NZS3101 limits to ASCE41-06, it is found that the definitions of Life Safety (LS) objectives in NZS1170 are more comparable to ASCE Collapse Prevention (CP) limits (although it is understood that ASCE is reviewing this). Note that comparison of different Building Codes may require a significant research effort.

2.3. There is a requirement through detailing to consider building toughness in an MCE event and the criteria are given in the NZS 1170.5 commentary. How much of this should be brought into the standard warrants further discussion

2.4. The question as to whether the buildings have performed as expected (as noted in 1.2.1, but not specific to capacity design) could be alleviated in part by a re-statement of what "designed to the Building Code" means. The achievement of 100% compliance is regarded as the epitome of good practice by many lay people, but it is in fact the absolute minimum acceptable standard. This could be informally addressed by engineers discussing alternatives with their clients, but in the past this has met with resistance, mostly over cost perceptions. However a better client education process, possibly with more explanation in the Building Code, could help. This is an area where industry organisations such as SESOC and IPENZ have a significant role to play.

3. Design Practice– a number of industry standard practices may be in question:

3.1. Analysis/loading standard issues

3.1.1. In general, the question as to whether the design standards need wholesale review or change hinge on the question of how much we wish to pay for increased performance in rare events. While feelings may be running high at present, in time, this may dissipate. However a related question is whether we will be able to insure buildings as easily as in the past, and indeed, whether we should.

3.1.2. Questions may be asked as to whether the Probabilistic Hazard analysis is serving us as well as it should. It is understood that minimum code levels have been set on the basis of a M6 earthquake at 20km radius. However damage observations tend to show a significant increase in damage levels within a 10km radius. It is possible that this is related to specific geological conditions in Christchurch, but this may merit further research and consideration - should a lesser source distance be used to set a minimum design load?

This would probably be opposed on the basis of cost, but it is clear from standard pricing guides that there is negligible cost impact for new buildings. With reference to Rawlinsons New Zealand Construction Handbook, the quoted rates for multi-storey office buildings, for example, are marginally more expensive in Auckland than Wellington and Christchurch, but all are within 2% over a range of different building types. This suggests that regional material and labour cost factors more than compensate for any cost difference as a consequence of seismic loading.

3.1.3. Use of nominally ductile procedures ($m=1.25$, $S_p=0.9/0.925$) without verification of adequate member ductility or safe building mechanisms – a common practice, particularly in areas of lower seismicity (Auckland for example) This has been seen in particular in the cases of buildings that have multiple walls, leading to thin, lightly reinforced walls. However, the risks of this are specific to materials and structural form and limits, if any, should be in the materials standards not the loadings standard.

3.1.4. In many cases the effect of the deep alluvial soils appears to have been underestimated. Although the subsoils clearly complied with the description of Class D in AS/NZS1170.5, there has been significant amplification of the high period response in some cases. It is not known if this is unique to Christchurch, but needs further research. These effects may mean that the increase in ULS and SLS factors made for Christchurch should be considered for the east coast of the South Island south of Christchurch which has similar geology and tectonic setting.

3.1.5. Low damage design is being touted as the way to go forward, but this is still a nascent technology (apart from base isolation). In particular it is important that we do not inadvertently swap the mistakes of the past for new mistakes in our eagerness to move forward. Concern should be expressed about PRESSS systems in any material which do not address beam elongation issues that are potentially just as severe as in ductile moment frames.

3.1.6. Base isolation appears to be a technology that has been proven (notwithstanding that there was only one base isolated building in Christchurch; but it has done well in other countries). However it is still not regularly used in NZ apart from in monumental structures, primarily because of cost considerations. This appears to contradict overseas practice, where it is often used. This is possibly a misconception, possibly a regulatory issue and possibly a market scale issue. The impact of Christchurch deep soils also needs careful consideration with the potential increased accelerations in the long period range.

3.1.7. Vertical accelerations have been touted as a significant factor in many failures, but there should be caution exercised in this regard, until further research has been done. Vertical accelerations were very high, but these are very short period transient effects, so before changes are made to the way we view them, research into their actual impact needs to be conducted. There will however be some aspects of design which may need further consideration, for example in the case of transfer structure where increased vertical load may result in disproportionate impact.

3.2. General Building Configuration.

3.2.1. There are a number of aspects to this that may merit consideration, but this is a very complex area. It has been generally observed though, that more regular buildings with greater levels of structural redundancy have performed better, all else being equal.

3.2.2. Plan irregularity. There has been discussion of the need for stronger regularity provisions, but this is problematic. For example an 18 storey tower of rectangular floor plan and a complete perimeter frame has behaved torsionally. The reason appears to be that the more flexible direction frame does not have the torsional stiffness to regularise the motion, once one end hinged and softened ahead of the other. Conversely, there are cases where the irregularity has been recognised and addressed in the design, and this approach has worked successfully.

3.2.3. Vertical irregularity. There are also a number of examples of where vertical irregularity has caused problems, mainly in shear wall structures. It is not known whether this was in error, or whether there are other factors such as higher mode effects that have been under-estimated by the Building Code.

3.2.4. With reference to the plan irregularity comment above, it appears that redundancy is something that could be addressed in the Building Code. In respect of torsional behaviour, an interim recommendation is that if there is a substantial difference in the contribution of the orthogonal lateral systems to resisting torsional response, that there should be at least three lateral load resisting elements of similar stiffness, in the more rigid direction.

3.2.5. The earthquakes have emphasised the need to approach diaphragm design much more rigorously. For example effects of elongation, sub-diaphragms, collector elements and load transfer to lateral load-bearing elements are all factors that could potentially be addressed in more detail. One of the weaknesses of diaphragm design is that it currently does not really fall well into any one of the materials standards. Requirements to use deformed bars (as opposed to mesh) must be emphasised.

3.2.6. A nationally accepted well based method of diaphragm design is an urgent requirement. This should address both demand and capacity issues, i.e. analysis as well as design.

3.3. Materials - concrete

3.3.1. Use of precast flooring systems in conjunction with ductile frames where elongation occurs is a practice that needs serious review. In future, precast flooring systems may need to be restricted to non-elongating systems, or at the least, to be explicitly considered in the design of the main structure. Some matters for consideration include:

3.3.1.1. Calculation of crack widths due to elongation,

3.3.1.2. maximum crack width that can be sustained without fracture,

3.3.1.3. where diaphragm forces can be transmitted to lateral force resisting elements,

3.3.1.4. influence of reinforcement on delamination and how this needs to be allowed for in design,

3.3.1.5. The reduction in remaining useful life associated with crack widths at supports etc. Adequacy of floor seatings has been called into question in recent years, but recent practice is still mixed.

3.3.2. Grouted ducted splices for precast connections. There have been a number of instances where ducts were not grouted, that should have been. But also, ducts may limit yield penetration by providing an over-confined region at a critical plastic hinge location. This may result in necking of the bar, and considerably lower available ductility than is expected.

3.3.3. Also on ducted splices – confinement of the splices is generally inadequate. Consideration should be given to either cross-ties to horizontals outside the ducts, or alternating zig-zag bars (probably the former is preferable).

3.3.4. Confinement of steel in walls. A number of walls have developed buckling failures of either (or both) vertical and horizontal steel. Future requirements may include providing confinement to all bars in the potential plastic hinge zone of the wall (ie for a height not less than the width of the wall or at least one storey zone (not just the outer part of compression block). This may extend to ensuring that cross-links tie to the outer layer of steel in all cases – frequently horizontal bars were unconfined, and have failed, although it is not clear whether that may have been initiated by buckling of the vertical steel, without which the horizontal steel would have remained intact. An adverse implication of tying the horizontals is that this would force the vertical steel inwards, and reducing the confined core areas at the wall ends.

3.3.5. Use of T- and L-shaped wall systems, resulting in compression failures in the outstanding leg. Many of these walls have behaved poorly, although this may be related to secondary vertical actions as noted below. The need to consider these walls about their principal axes may also be considered.

3.3.6. The whole issue of compression force in concrete shear walls needs review. Christchurch walls showed failures due to much higher than expected compression loads and these can come from the following sources in addition to the considered flexural overstrength:

1 Additional vertical loading as the inelastically responding wall uplifts the typically rigidly connected floor system. The recent PhD by Richard Henry at UofA showed this can increase the compression force on the shear wall by 25% minimum
2 Vertical accelerations from the earthquake combining with flexural actions on the wall from lateral loading (although vertical accelerations are generally in the very short period range and may be transient, therefore not a major contributor).
3 Compression failure at the toe of the wall from 1 and 2 causing the plastic neutral axis to migrate further into the wall for a given direction of loading than expected
It is clear that many of these walls were in compression over most of their length during the cycling back and forth of the building leading to failure in the middle regions that were detailed to be in tension only and so the two layers of reinforcement were not cross tied. T and L shaped walls would make this worse as the compression region currently considered is that much smaller.

Alternatively, it should be considered that although a full section analysis will generally indicate that the steel in the central portion of the wall yields in tension in both directions, this does not consider that for walls under appreciable gravity load (as well as the added vertical loads noted above), any tension cracks in the centre of a wall must close as the wall goes through loading reversals – hence the reinforcement is required to yield in compression even though by analysis it is always in tension.

3.3.7. There are some aspects of wall behaviour that may be addressed by redistributing the steel in the walls. A trend that has developed along with using ducted splices is to distribute the vertical steel evenly to make construction easier. However it may be better to concentrate the steel at the ends in order to provide better crack distribution.

3.4. Materials – unreinforced masonry – see below under strengthening.

3.5. Materials – steel

3.5.1. While there were less large steel framed buildings in Christchurch than concrete frames, those include a mix of eccentrically braced, concentrically braced framed and moment resisting systems ranging in height from 2 to 22 storeys. This is a sufficient data set to state general conclusions on performance.

3.5.2. Most of the recent steel framed buildings performed well and delivered strength and stiffness in excess of design expectation, although there were some exceptions.

3.5.3. One EBF structure and one CBF structure showed examples of poor performance due to inadequate detailing. In the case of the EBF, the braces did not line up with the stiffeners, leading to severe overloading and ductile fracture. In the case of the CBF, the connections were not adequate to develop the tension capacity of the brace as design procedures require. These examples show that poor design or detailing will result in poor performance.

3.5.4. There was brittle failure of some proprietary tension bracing connectors which are cast steel that show the need to ensure cast components are considered for brittle fracture suppression in the same way as fabricated components must be from NZS 3404. (Cast components fall outside these provisions at present)

3.5.5. Composite steel concrete floors, comprising insitu concrete on steel deck supported on steel beams, exhibited excellent performance, with neither diaphragm damage nor more than hairline cracking observed, even immediately over the active link regions of eccentrically braced frames. This however raises potential concerns with respect to capacity design if these elements contribute to the link strength, so there is a need to consider this in the design of the other frame elements. Effects of slabs and their contribution to element and building performance are currently being undertaken at the Universities of Auckland and Canterbury.

3.5.6. Two significant older steel frame with infill buildings behaved poorly – the BNZ tower shear walls failed in one case, with reasonably large embedded structural steel sections failing in tension. It is very important that the exact detail of these steel members and where they failed in tension needs to be investigated. If possible, the failed regions should be cut out and metallurgically examined

3.5.7. Light steel framed housing with or without brick veneer performed very well with minor non structural cracking the only issue reported.

3.6. Materials – timber – timber structures, predominantly low rise, performed well and in many instances provided vital support to URM elements. There are few modern timber non-residential structures, so these cannot be commented on.

4. Review Practice

4.1. It is understood that until relatively recently, most Building Consents in Chch were awarded on a Design Certificate (precursor) or Producer Statement. There was random review of projects, rather than systematic review. This probably changed in the late-90s or early 2000s.

4.2. Full peer review was rare, with most buildings being reviewed internally (to the CCC).

4.3. Notwithstanding the above, it is debatable whether the overall standard of design was any lower as a consequence.

4.4. It is unclear whether the CCC has the ability to review large numbers of consents or technically challenging designs. However they are able to use external peer review, as has been done by other BCAs.

4.5. A common practice noted elsewhere in the country where peer review is used extensively has been for building developers to engage their own peer reviewer in order to supply PS2s contemporaneously with the designers' PS1s. It has been suggested that there is a lack of independence in this process, and this practice may be subject to review. Note however that this is not relevant to any observations from the Christchurch earthquakes.

5. Construction Monitoring

5.1. Involvement of engineers through the construction process may be an issue. This is generally a contractual matter, whereby the engineer is engaged to provide a pre-determined level of construction monitoring, generally on a fixed fee basis. Although the obligation is related to the work activities, it is noted that the fee seldom varies regardless of the contractor, and regardless of their QC systems. Hence there may be pressure on engineers to soak up cost if the contractor has particularly poor QC systems in place.

It should be further noted that the level of construction monitoring followed is generally considerably less than full supervision and engineers are frequently not able to complete full inspections of all work. The level of sign-off given in the PS4 statements is therefore generic, and reliant on the contractors own statements in support.

5.2. It is noted that there were some generic details that have obviously not been universally well followed through. Examples include:

5.2.1. Ducts that were not grouted

5.2.2. Block walls not grouted properly

5.2.3. Poor positioning of confining or anti-buckling steel

5.2.4. Poor execution of tie-down details or other connections in timber structures

5.2.5. Introduced eccentricities in concentric bracing elements through poor alignment

5.2.6. Take-up of movement allowances as construction tolerances.

5.3. A possible solution for some of the above could be to follow a similar practice to that followed in the US codes. There, critical structural review elements are specified in the code and require special inspection. A Special Inspector is a completely independent reviewer who is engaged specifically to ensure that all identified elements are implemented exactly as per the engineers' drawings, no exceptions.

6. Performance observations (noting that SESOC is to put in place web-based information gathering

6.1. Reinforced concrete frames:

6.1.1. Damage to frames is not so easily repairable, with concerns about the impacts of:

6.1.1.1. Frame elongation, with large floor cracks and reduction of available floor seating. This is especially important with precast flooring systems. Putting aside earthquake, there is often pre-existing shrinkage cracking at these locations, which with creep will result in cracks greater in width than any rotation from imposed loads can close. Considered with earthquake, it is critical that designers always design precast flooring for the simply supported load case, with no continuity, regardless of the presence of saddle bars.

6.1.1.2. Low cycle fatigue. Determination of the strain in reinforcement within plastic hinge zones, where the cracking pattern does not match that seen in the lab of large numbers of closely spaced narrow cracks. Instead we observe small numbers of wide cracks with no cracking in between, indicating that yielding in the reinforcement has concentrated into one or two locations. It is likely that the reinforcement strain in these cracks is much higher than expected from lab based assessment procedures. Charles Clifton's tentative recommendation is to base the yielded length for reinforcement strain determination in these cracks on the crack width + 1bar diameter, not crack width + 6 bar diameters.

Until reinforcement is extracted from these joints and tested to determine the strain demands that have occurred we need to be very cautious on this topic. It is of utmost importance that this is done and that representative joints are built in the laboratory and tested under seismic-dynamic conditions to determine if the cracking patterns shown in the field can be replicated in the laboratory. Only when this is done and strain studies on reinforcement from actual joints are undertaken will we have a good understanding of how these joints have performed in practice under dynamic seismic loading instead of under static loading in the laboratory.

It is noted that some testing has been done in the field, breaking out concrete at crack locations and using Leeb hardness testing of the steel. This has indicated in some cases that the strain hardening has occurred shorter lengths either side of the crack as noted above.

6.1.1.3. The impact of settlement and tilt – often not technically an issue, but owners are claiming that buildings must be fully restored to pre-earthquake condition, including being completely straight and level.

6.1.2. Combination with precast flooring is not good – in addition to the points noted above, it is noted that additional floor liveliness was anecdotally recorded for some precast floors post-September, although this may also be attributable to increased occupant sensitivity. (If occupants expect a floor to be lively they are much more perceptive of any liveliness).

6.1.3. Hinge formation did not conform to expectations from prior testing – fewer larger cracks as noted above. There may be a number of possible reasons for this including:

6.1.3.1. Concrete age-related strength and material properties. It is recognised that tensile and compressive strengths of concrete change with age, and each change at a different rate. Further, concrete suppliers have often supplied significantly higher strength concrete, either to guard against missing the target strength, or to satisfy other (usually contractor led) criteria such as early lifting of panels. In a recent example, use of self-compacting concrete resulted in a 7-day strength of 90MPa, where 40MPa was the nominated 28-day strength.

It is assumed that if an element is pushing the lower bound reinforcement concrete (such as in concrete panels where there are excess wall elements) then if the concrete strength is too high, the reinforcement will not be able to force crack distribution.

It is noted also that lab specimens are often tested at 28 days, regardless of the concrete strength. At this age, the ratio of tensile strength to compressive strength will be very different.

6.1.3.2. Dynamic bond strength of concrete together with a low rate of strain-hardening of the steel may influence bond in plastic hinge regions

6.1.3.3. The load history of the real earthquakes is very different from that typically used in laboratory situations, where a cyclic loading regime using multiple repeats of gradually increasing displacements. In reality the major displacements noted were very early in the cycle, and hence the major crack may have fully widened in the first load cycle.

This may mean that use of the pseudo-static testing process should be reviewed, presumably in favour of dynamic testing, although such facilities are not available in NZ (at an appropriate scale).

6.1.3.4. Boundary conditions. These need to be representative too as they can add compression or tension to beams as elongation occurs. This effect should be considered.

6.1.4. Most taller buildings are slightly older, that is they pre-date 1995 code changes, therefore there is not so much observation possible of more recent building design performance.

6.1.5. Older buildings may have had more structural redundancy, as early computer systems only had sufficient capacity to process the specifically designated later load resisting structure. However, the gravity structure may have contributed considerably to the overall resistance, albeit often with reduced ductility capacity, due to reduced detailing being used.

6.2. RC walls:

6.2.1. Difference between expectation and actual performance – few large cracks. This is potentially the same indication of a fundamentally different nature of bond development between concrete and reinforcement than what has been observed pseudo-statically in the labs.

6.2.2. A number of toe crushing failures – lack of adequate confinement

6.2.3. Poor performance of taller singly reinforced walls without boundary elements. This may include precast tilt panel structures, but these may need to be considered in two categories – the first being the industrial structures where the panels do not carry significant axial load other than self weight; and the second in cases where the panels do carry axial load, often from floors where this method of construction has been used in residential multi-storey construction.

6.2.4. Grouted ducts not performing well – broken out of walls in a number of locations

6.2.5. There were a significant number of locations where horizontal steel has broken out of walls.

6.2.6. It is debateable whether the current practice of ignoring out-of-plane movement of concrete walls may be unconservative. There may be cases where the combination of in-plane and out-of-plane movements has caused failure of walls, including possibly the Grand Chancellor shear wall.

6.3. Foundations

6.3.1. Mixed solutions have performed poorly. In particular it is noted that where tension piles have been used to provide tie down actions to walls in parallel with pad or raft foundations, the outcome has been mixed

6.3.2. Although many buildings that have been damaged due to liquefaction were designed prior to a good understanding of the nature of the problem, some more recent buildings that have used a risk-management approach may not have performed acceptably. In these cases, often pads with reduced bearing pressures were used. This has not worked well in some cases, with the displacements causing other compatibility issues.

6.3.3. Conversely, in some cases where hardfill rafts have been used to provide some additional robustness, this appears to have worked, at least for low-height structures, even though it may be difficult to verify by analysis.

6.4. Performance of adjacent buildings has been a problem in many cases. There are many cases where buildings have been closed because of structural issues on the adjacent buildings. This is a most critical issue in considering IL3 and IL4 buildings, which may be required to be operational post-earthquake and which may otherwise be suitable.

7. Existing Structures/Strengthening

7.1. The wording of the Act is not clear – it refers to collapse, which is readily definable in the context of outcome, but not practical for engineers to analyse sensibly. There are (or have been) engineers suggesting that a collapse limit state of 33% of code equates to a calculated capacity of approximately 25% at ULS, i.e. the threshold gets dropped even lower, or is lower than it appears. The

Act should refer to ultimate limit state capacity, which is at least something that can be verified by engineers' calculations.

7.2. The linking of the EPB definition to the Dangerous Building definition (as in the September 2010 Order in Council) is a problem in that it implies that all EPBs are dangerous buildings. This is problematic in that it would therefore appear that all EPBs are dangerous even if undamaged (potentially happening away form the epicentre, eg in Rangiora). This makes it questionable whether these buildings can or should be legally occupied prior to strengthening, notwithstanding the acceptable timeframes for strengthening.

7.3. A disappointingly low number of buildings were strengthened prior to the earthquake.

7.3.1. Successive passive policies resulted in relatively few buildings being strengthened, unless undergoing change of use.

7.3.2. The Christchurch EPB policy still allows too long a period for compliance – 30 years for IL2 buildings at the time of this review, although that is being reassessed currently.

7.4. Load levels

7.4.1. Secured buildings (generally .05-.1g) worked well through Sept earthquake

7.4.2. But not so well in February, which is not surprising given the 2.5 to 3 times higher PGAs.

7.4.3. Buildings strengthened to the minimum standard (new systems, up to 33%) achieved life safety performance, but will still be demolished in many cases.

7.4.4. Buildings strengthened to a higher standard (new systems, 67% or more) performed better – many survived with moderate to low damage.

7.4.5. From this, we would conclude that 33% of code is still too low in most cases.

7.4.6. We question whether there should be more of a deterministic approach taken to ERBs and EPBs in particular. If it is considered that approx 50% of code loading for buildings in Chch was about the minimum level that provided some degree of security in these earthquakes (assuming good founding), this corresponds to Z=0.11 relative to NZ1170.5. By comparison, 100% of code in Auckland is Z=0.13, ie it would be necessary to go to 85% of code in the lowest seismicity areas to achieve a similar performance in a similar sized earthquake. At the least, consideration should be given to a minimum that is higher than 33% of code in these lower seismic zones, with 67% being an obvious target.

7.4.7. It is <u>not</u> acceptable that IL3 and IL4 buildings in particular, are required to conform to the same standard (relative to code) as IL2 buildings, ie 33%. In the case of buildings required to be operational following an emergency (of which earthquake is one of our most significant hazards) no relaxation from 100% (of IL4) should be permitted, at least from the SLS2 provision. Furthermore, allowing a lengthy timeframe to achieve compliance is equally unacceptable. If a building is not fit to be designated IL4, its use should be downgraded. The only relaxation that may be acceptable is in cases such as Christchurch currently, where the seismicity has been increased reflecting the recent earthquakes, and the existing facilities have performed well (only Chch Womens hospital and the Control Tower at the airport probably fit into this category). This may require verification of performance by methods such as inelastic time history analysis, which will give a better evaluation of likely performance.

7.5. Policy setting

7.5.1. It seems incongruous that EPB policy is not covered by central government policy, although most other building related matters are. Given the age of the overall NZ building stock, this represents a significant hazard, and it seems more appropriate that this is addressed at central government level.

7.6. Strengthening methods.

7.6.1. It was apparent that at the lowest level, any work was better than none, and even some very rudimentary strengthening could be surprisingly effective.

7.6.2. Saying that, there were many cases, where the lack of a complete load-path was exposed for strengthening projects just as badly as for new buildings.

7.6.3. Out-of-plane failure of masonry parapets and walls was a significant issue. It is difficult after the fact to determine whether this was as a consequence of failure of ties, or failure of the walls due to excessive slenderness.

7.6.3.1. Failure of ties could be attributed to overload (generally spaced too widely for full loads, even though they may have been adequate for securing to lower standard), to installation faults (a surprising number of suspiciously clean ties were observed post-earthquake), to poor detailing (horizontal drilling of epoxied ties engages only one brick, compared to a 22.5° slope as standard US practice).

7.6.3.2. Failure of the walls may have been in case where the h/t ratios were higher than acceptable, but this is not easy to verify. If h/t ratios have been set according to lower levels of resistance, how should these be varied according to overload?

7.6.3.3. Notwithstanding the notes above about epoxied ties, through ties with rosehead washers appear to offer significantly superior performance, provided they have blocking on the inside to the adjacent framing.

7.6.4.

8. Non-structural systems

8.1. Mixed performance

8.1.1. Eg Gib in CCC new office is now on its fourth repair...

8.1.2. There may be too much elaboration?

8.1.3. Lift guide rail systems need a fundamental redesign to make them more tolerant of permanent drift following earthquakes. They are currently the “Achilles heel” of low damage construction being the most sensitive element to building residual drift

9. Procedural matters:

9.1. Immediate recovery and assessment

9.1.1. USAR training and mobilisation very successful

9.1.2. L1/L2 assessments need fine-tuning (although it is noted that there are reviews underway that will provide considerably deeper consideration than this)

9.1.2.1. In particular with training of personnel – too many mistakes were made by well-intentioned engineers with limited training and/or experience. The engagement of graduates in particular for this sort of work was questionable. However it is apparent that there were many mistakes made.

9.1.2.2. Record keeping was mixed – unprepared, data not available when needed, too much reliant on hand-written forms and translation by people unfamiliar with jargon. Better systems need to be considered.

9.1.2.3. There is a serious lack of understanding (still) of the significance and meaning of the placards. A lot of education is needed both for building owners and the general public.

9.1.3. Detailed evaluation

9.1.3.1. No systems or legislation in place prior to the earthquakes, resulting in a scramble to put measures in place now.

9.1.3.2. Lack of legislative support for post-earthquake assessment, resulting in this being addressed in the CERA legislation – it needs a permanent home.

9.1.3.3. It is notable that many people have confused the L1/L2 assessments for detailed evaluations. The public (and possibly some owners) have a misguided view of what is required and how long it will take to complete adequately; and what the risk is to building users. This is a much broader issue than can be covered purely in engineering terms. In particular it is noted that the speed of the recovery is almost diametrically opposed to the desire for safety.

9.1.4. On-going safety

9.1.4.1. Placard life is an issue, both in the practical sense that they do not remain colourfast, and in the sense that they expire after the Civil Defence state of emergency is lifted, even though the hazards may remain.

9.1.4.2. Complication of process – placards should simply remain live until repairs completed. This may require tidying up of the legislation – because initial placarding is done under CDEM Act, they expire when state of emergency is lifted, and can only be used if state of emergency is actually declared.

9.1.4.3. A time-limit for the detailed engineering evaluation of buildings once the state of emergency is lifted needs to be imposed. The size of the task is immense, if all potentially affected buildings are to be reviewed at some level. As noted above, this is critical to the recovery but involves more than just technical considerations.

10. Future Code/legislation changes (speculative):

10.1. Seismic Loading

10.1.1. Probabilistic hazard analysis (PHA) has shortcomings –

10.1.2. Although statistically correct, PHA appears to underestimate real earthquakes.

10.1.3. Understand M6-6.5 earthquake possible anywhere in country.

10.1.4. Current iso-seismals may be too complicated, give impression of greater accuracy than actually exists.

10.1.5. Seem inconsistent with NZ geology – narrow country along a plate boundary.

10.1.6. Deterministic may be better, or combination of the two, eg PHA with deterministic minimum level.

10.1.6.1. Suggest therefore a two zone approach – Zone A central region along main faultlines, Zone B outerlying secondary fault region, for the rest of the country.

10.2. General design philosophy and building configuration

10.2.1. Introduce explicit review of key building elements for the impact of MCE drifts

10.2.2. Increase SLS1 load level, but maintain overall ductility/capacity design approach

10.2.3. Possibly add SLS2 limit state recommendations to IL3 structures?

10.2.4. Review and amend regularity provisions, including:

10.2.4.1. Consider redundancy provisions as noted above, including torsional response.

10.2.4.2. Review other existing provisions to ensure they are set at the right level

10.2.5. If low damage design is going to have widespread use, it is important that some controls or requirements are put in place to ensure that all of the performance issues are addressed, particularly diaphragm design.

10.2.6. Introduce better consideration or guidelines for diaphragm design, both flexible and rigid.

10.3. Soils and foundations

10.3.1. Identification of regions of highly liquefiable geology, with greater consideration to be given to where we can build and what forms of foundation may be suitable

10.3.2. More interaction between geotech engineers and structural engineers may be required, although this may be a practice issue rather than a code issue. But it may be beneficial to consider requiring sign-off from geotechnical engineers in parallel with structural engineers PS1s, at least in critical locations.

10.4. Materials standards

10.4.1. As noted generally in the sections above, with in particular:

10.4.2. Concrete:

10.4.2.1. Consider restrictions on use of precast flooring with elongating frame systems

10.4.2.2. Review and change design and detailing requirements for concrete walls, addressing:

10.4.2.2.1. Confinement/anti-buckling steel for potential hinge zones needs to be reviewed.

10.4.3. Wall configuration issues need to be addressed (no more T- and L-shaped walls?

10.4.4. Pattern of strain development in rebar in plastic hinge zones needs urgent investigation to understand what has been observed in the field and to provide robust guidance for post-earthquake peak strain evaluation

10.5. Existing Buildings:

10.5.1. Consider central policy regarding EPBs.

10.5.1.1. Raise both threshold load and strengthening requirements.

10.5.1.2. Look at timeframes for strengthening, taking into account the use of the building. Those which are greater than IL2 should either have the use changed in the interim to lower the importance level, or should be immediately dealt with.

11. Training and Competence of Structural Engineers

11.1. Initial training of structural engineers is completed at the Universities (Canterbury and Auckland) in NZ but is the level adequate? Should there be post graduate professional study and examination (similar to the IStruct E examination in the UK, noting that this is specific to structural engineering as opposed to the more general ICE qualification) prior to being eligible for CPEng and LBP?

11.2. Training of overseas engineers is a significant issue, particularly for those from non-seismic areas. Civil and structural engineering are listed as long-term skill shortages, noting that the qualifications are:

Bachelor Degree (Level 7) qualification and registered on the International Professional Engineers Register or Asia Pacific Economic Co-operation Engineers Register or membership of the Society of Petroleum Engineers OR a Washington Accord accredited engineering degree.

While it is acknowledged that these qualifications are of a high standard, they do not necessarily cover seismic design to the level necessary to practice in New Zealand.

11.3. The adequacy of current CPEng procedures for initial and continuing assessment needs to be considered. As implied in 11.1 above, there is no specific examination to achieve CPEng, although there is a robust interview process for most first time candidates.

11.4. Should there be a higher level qualification for certain types of structures i.e. multi-storey? IL4? Etc.

11.5. Adequacy of Continuing Professional Development training-how can we ensure that those that need this get the appropriate training?

12. Geotechnical Studies

12.1. Should site specific studies including liquefaction potential be mandatory prior to issuing building consent?

12.2. Should a PS 4 from a geotechnical engineer be required prior to issue of the Code Compliance Certificate?

13. Consenting Issues

13.1. If PS1 is accepted by BCA should there be some mandatory random auditing carried out on issuer of PS1 or PS2?

13.2. What level (if any) of Peer review should be carried out? Should this be independent?

13.3. Should peer reviews be mandatory for particular types of buildings i.e. multi storey, IL3 & IL4?

13.4. Construction Monitoring should be mandatory for any project that a PS1 is issued for. Should the BCA review and ensure that an adequate level of CM is carried out and that this be a condition of the consent?

14. Standards and Codes

14.1. The present system of developing standards in NZ is slow and under resourced. For example, the timber design standard, NZS3603:1993, has had four amendments in order to address some of the known issues with timber strength and to bring it into line with the loadings standard. However, it has been in need of rewriting for some time.

14.2. Resourcing for the speedy development of updated standards should be made by DBH or public agency in the interests of "public Good" to incorporate lessons from research and major events such as earthquakes.