Mitigation of seismic hazard in Australia by improving the robustness of buildings

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ABSTRACT: Robustness is defined in the commentary to the Australian Standard AS1170.0-2002 as follows: “A structure should be designed and constructed in such a way that it will not be damaged by events like fire, explosion, impact or consequences of human errors to an extent disproportionate to the original cause.” The statement appears to be relevant to earthquake-resistant design but is vague and difficult to interpret. In most cases the engineer is likely to simply follow the minimum standards and assume that their building will be sufficiently robust as a result. These standards do consider earthquakes, but for most buildings it is only necessary to design for an earthquake event with a return period of 500 or 1000 years for ultimate strength design. These are small return periods compared with the recurrence intervals of large earthquakes in Australia, and this approach does not appear to take into account the fact that, unlike with wind loading, the ground motions corresponding to a particular return period increase indefinitely at a steep rate with increasing return periods. This paper contains practical suggestions for design that illustrate the importance of following basic principles and using appropriate detailing. Engineers are encouraged to go beyond the minimum standards to ensure that their building is sufficiently robust when considering very rare events. There is also a section that illustrates research efforts being made to determine the level of robustness of the current building stock in Australia, with the eventual aim being to suggest changes to Australian Standards.

1 INTRODUCTION

The twenty-fifth anniversary of the most damaging earthquake in Australia’s history passed at the end of 2014. The magnitude, M_w 5.6 earthquake that struck Newcastle on December 27, 1989 caused 4 billion dollars of damage in today’s terms and claimed the lives of 13 people. This was a small earthquake compared to the largest historical earthquakes that have occurred in Australia (up to M_w 7.2), and at present it is difficult to rule out the possibility of events this large or even larger (up to M_w 7.5) occurring close to Australia’s major cities. Geological studies (Clark et al., 2010, 2011) indicate that the slip rates on these faults are very low (e.g. 50 metres per million years) compared with those in areas of high seismicity (eg. 50 km per million years), so the average recurrence interval of events greater than M_w 7.0 on any particular large fault in Australia is likely to be of the order of tens of thousands of years.

While it’s important not to overstate the seismic hazard, it is also important not to understate it. The Canterbury earthquake sequence in New Zealand which occurred from 2010 to 2012 demonstrates that the damage caused by very rare events striking close to the CBD of a major city is likely to be extensive and costly (Goldsworthy,2012). It is important to note that the most damaging earthquake in the Canterbury sequence, the February 22nd 2011 Christchurch earthquake, a magnitude M_w 6.1 event, had a reverse faulting mechanism similar to that on most faults in Australia and was shallow (less than 20 km from the surface). The damage from such an event in Australia is likely to be catastrophic given the vulnerability of many existing structures, even ones designed in accordance with the current

Australian design standards.

Robustness is defined in the commentary to the Australian Standard AS1170.0-2002 as follows:*"A structure should be designed and constructed in such a way that it will not be damaged by events like fire, explosion, impact or consequences of human errors to an extent disproportionate to the original cause."* This is an admirable objective and stems from the Ronan Point disaster in England in which the south-east corner of a 22 storey block of apartments suffered a progressive collapse when a resident on the 18^{th} floor went to light her stove and a gas leak caused an explosion that blew out the load-bearing flank walls (Griffiths et al, 1968). The damage was definitely disproportionate to the original cause in that case. The statement appears to be relevant to earthquake resistant design but it is vague and difficult to interpret. It is left to the individual engineer to decide what sort of damage would be considered "disproportionate to the cause" in the case of an earthquake event. In most cases the engineer is likely to simply follow the minimum standards and assume that their building will be sufficiently robust as a result. These standards do consider earthquakes, but for most buildings it is only necessary to design in accordance with The Building Code of Australia (Australian Building Codes Board, 2011) for a ground motion with a return period of 500 years (Importance Level 2 (which corresponds approximately to a 10% probability of exceedance (PE) in 50 years) or 1000 years (Importance Level 3) for ultimate strength design. These are small return periods compared with the recurrence intervals of large earthquakes in Australia, and this approach does not appear to take into account the fact that, unlike with wind loading, the ground motions corresponding to a particular return period increase indefinitely at a steep rate with increasing return periods (Somerville et al., 2013). As a result, if a building is designed for the same probability of exceedance for both wind and earthquake, the residual risk, defined here as the exposure to loss remaining after the required design standards have been adhered to, will be higher in the case of earthquakes.

This paper contains some practical suggestions for seismic resistant design that illustrate the importance of following basic principles and using appropriate detailing. These are largely based on the second author's experiences after the Christchurch earthquake, his structural design of the new Royal Adelaide Hospital in Adelaide (M^cBean, 2015) in which seismic-resistant design was carefully considered, and his co-authoring of the new guidelines to RC design (Munter et al., 2015). There is also a section that illustrates some of the research that has been and is currently being conducted by the first author together with her PhD students and co-supervisors to determine the level of robustness of the current building stock in Australia, with the eventual aim to suggest changes to Australian standards, both in terms of the philosophy behind the designs as well as improvements in the standard of detailing required in the material standards.

2 PRACTICAL SUGGESTIONS TO IMPROVE THE ROBUSTNESS OF BUILDINGS

Whenever a large earthquake strikes close to a built-up area, buildings and their non-structural components and contents fail in ways that have been observed time and time again. Sometimes there are new lessons, but if some basic design principles have been adhered to, and there has been sufficient attention paid to the details, it has been observed that buildings have a much better chance of survival. This was also borne out in the aftermath of the Canterbury sequence of earthquake in New Zealand from September 2010 onwards. Much has been written about the performance of building structures in Christchurch as a result of these earthquakes and this will not be repeated here. The most sobering of these is, of course, the findings of the NZ Royal Commission conducted into the reasons for the high loss of life and extensive economic loss suffered by Christchurch during those events (Canterbury Earthquakes Royal Commission, 2012).

2.1 Appropriate Material Selection

The designer should choose materials that are inherently ductile rather than brittle. The total loss of almost all unreinforced masonry structures in Christchurch illustrates the vulnerability of this type of construction in particular.

2.2 **Regular Configuration**

Structures configured with layouts that are regular in plan, elevation, mass distribution and lateral resistance have been shown to perform well during real earthquakes. Often, complicated planforms can be rationalised or simplified with the introduction of seismic joints which subdivide a complex layout into a number of more regular structures. The complex form of the new Royal Adelaide Hospital was subdivided in this way to create 18 regular, laterally independent, but contiguous structures (McBean, 2015).

2.3 **Redundancy**

Redundant structures have the capacity to redistribute force internally. Designing structures with redundant load paths gives them the ability to absorb extensive damage, redistribute actions internally and continue to function.

2.4 **Direct Load Paths**

Load paths resisting both gravity and lateral design actions should be simple, well established and direct. Non redundant load paths such as transfer structures should be avoided where possible, as failure of these individual elements leads to collapse.

2.5 **Punching Shear Failures**

Collapse due to punching shear failure at slab-column connections can be all but eliminated through the addition of a very modest amount of additional bottom face reinforcement passing through the joint. Reinforcement of this type is currently not required by AS3600-2009, however it is specified in ACI 318-14 where it is referred to as “integrity reinforcement”. An example of this type of failure from Christchurch is given in Figure 1.

Figure 1: Punching shear failure in Grand Chancellor building, Christchurch

2.6 **Column Design**

Columns are arguably the most important element of any building structure. Generally, if columns fail, the structure collapses. It is for this reason that particular attention must be paid to ensuring their survival during earthquakes. With the advent of very high strength concretes, the trend in Australia has been towards smaller columns requiring extensive confinement to simply work under gravity loads. Unfortunately this can come at the expense of ductility. For columns with ordinary strength concrete the provision of closely spaced well-configured ties, particularly in the column end regions, is good design practice. It is important to note that analyses and testing by (Goldsworthy, 2007) and (Wilson, 2014) have shown that heavily loaded columns have reduced drift capacities, particularly as the axial load ratio approaches or exceeds the balance point on the column interaction diagram. Figure 2 (Paulay, 1988) illustrates the reasons behind this, i.e. simply that the compression strain required to achieve a given ultimate curvature ductility is much higher for columns that are heavily loaded.

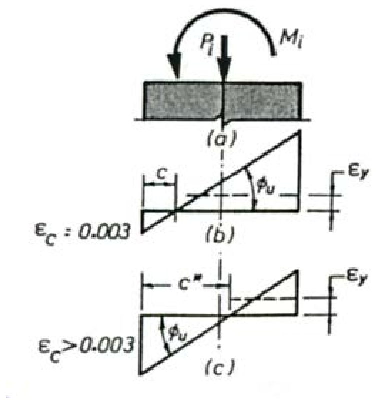

Figure 2: Effect of axial load level on the compressive strain required for a given ultimate curvature (b) Low axial load (c) High axial load (from (Paulay, 1988))

2.7 Thin Loadbearing Precast Concrete Panels

The increased use of high strength concrete in loadbearing precast concrete panels has resulted in a proliferation of thin, highly loaded and often lightly reinforced panels. Such walls are frequently used in loadbearing applications for apartment construction, where they also provide the lateral stability from their in plane strength. In Christchurch it was observed (Sritharan et al 2014) that concrete walls with low levels of vertical reinforcement tend to form relatively few flexural cracks in hinge zones, whereas walls with higher quantities of vertical steel consistently form multiple cracks distributed throughout the hinge. The large single hinge zone crack associated with lightly reinforced walls concentrates the plastic strain and that was observed to sometimes cause fracture of longitudinal wall reinforcement. Lightly reinforced walls relying on a single layer of central reinforcement exhibit no useful level of ductility, and certainly not the value of μ=2.0 currently attributed to such walls by AS3600. Analytical work on this topic has been completed at the University of Melbourne as outlined in the next section, and testing of this wall type is currently underway at Swinburne University of Technology to verify this behaviour. It is the authors' opinion that wall design based on the simplified method in AS3600 should be used with extreme caution in seismic applications. Also, thin structural walls were observed to buckle and fail prematurely in Christchurch (Sritharan et al 2014). It is suggested that minimum wall thickness to height ratios should be established to prevent this type of failure.

2.8 Drift and Displacement Compatibility

The California State University carpark collapsed during the 1994 Northridge earthquake (Moehle et al, 1994). The structure incorporated a well detailed ductile perimeter moment frame (which was assumed to resist lateral design actions) and an interior gravity frame with simple detailing. As the structure responded inelastically to the ground motion, the drift required to mobilise the ductile perimeter frame exceeded the drift capacity of the interior columns which failed, leading to collapse. The lesson is that everything connected by the floor diaphragms moves together. Failure to understand and acknowledge this during design may lead to collapse. For example, for construction efficiency many CBD office buildings incorporate a reinforced concrete lift and stair "core" together with loadbearing reinforced concrete precast boundary walls which support the floorplates. A common mistake made by practicing engineers in this situation is to assume that the core resists all lateral design actions and to design the precast panels for gravity loads only. During an earthquake, the drift required to mobilise the inelastic response of the core assumed during design will greatly exceed the drift capacity of the perimeter walls leading to their failure and collapse. This situation is made worse if the designer has assumed a ductility of μ = 3.0 for the core and detailed it in accordance with Appendix C of AS3600.

2.9 Services Coordination

The design process is iterative and buildings are designed by teams of collaborating individuals. It is important however for structural engineers to understand how the building services will influence their structural design at the earliest possible stage. Of particular importance are penetrations in the floorplate which are adjacent to columns and the large risers often required for services on the face of cores and structural walls.

2.10 Unintended Interaction with Non-structural Components

Poor detailing can easily result in non-structural components such as precast cladding and infill masonry having an adverse effect on overall structural behaviour. A common example of this type is the "short column" shear failure induced by interaction between infill partitions and a reinforced concrete frame (see Figure 3). Such column failures are frequently observed during real earthquakes. Detailing of non-structural components is substantially improved when designers properly consider the inter-storey drift associated with the overall seismic structural response.

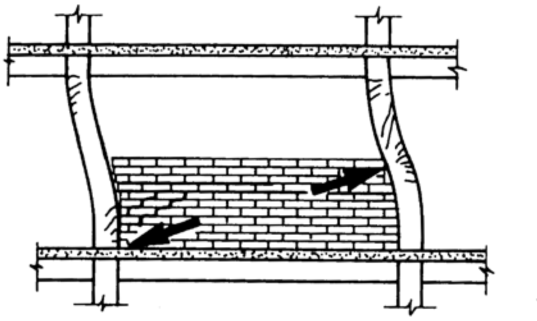

Figure 3: Effect of partial-height infill (from Paulay and Priestley, 1992)

2.11 Tie Everything Together

Designers are often under pressure to minimise the quantity of reinforcement used in their designs. However, it must be remembered that a chain is only as good as its weakest link and that a properly designed and detailed lift or stair core will be unable to laterally restrain a building if the floor diaphragms are inadequately anchored into it. The strategic inclusion of relatively small quantities of additional reinforcing steel to ensure load paths between all structural elements are reliably maintained greatly enhances robustness. The largely undamaged core that remained standing after debris was cleared from the CTV site in Christchurch, as shown in Figure 4 illustrates the importance of proper anchorage between structural elements.

Figure 4: Failure of connection between diaphragm and core, CTV building, Christchurch

2.12 Diaphragm Integrity

Floor diaphragms are often overlooked or ignored during design, yet their function is critical to the overall seismic performance. Many diaphragm failures were observed in Christchurch, frequently associated with beam elongation in hinge zones leading to fracture of low ductility mesh reinforcement used as secondary reinforcement in floor systems. In addition, design actions within diaphragms can be seriously underestimated when the interaction of structural walls and frame action is ignored.

3 RESEARCH TO EVALUATE AND IMPROVE ROBUSTNESS

In the New Zealand material codes it has been common practice since the 1980s to use capacity design principles in design (Park, 1992 and Paulay, 1985). In the very rare earthquake event experienced in Christchurch on February 22nd 2011, it was this design approach that saved many lives since, although many post-1980s buildings were severely damaged in the regions of the structure that were designed to go into the plastic range, the buildings did not collapse catastrophically.

Australian material codes do not enforce capacity design principles and hence it is possible that buildings designed in Australia would undergo a brittle and catastrophic failure in a very rare event. As mentioned above, the Building Code of Australia (Australian Building Codes Board, 2011) requires engineers to carry out an ultimate strength design typically for an event with either a 1:500 or a 1:1000 probability of exceedance. The possibility of collapse under a higher level event such as a 2500 year return period event is not considered, even though this is the state of the art for design in other countries such as the United States and New Zealand (Nordensen and Bell, 2000).

In Australian research into the robustness of older buildings, and also of buildings designed in accordance with current codes, researchers have attempted to identify those structures that are likely to be most vulnerable to earthquakes. Post-Christchurch there is a greater sense of urgency associated with this work and a greater tendency to consider the ability of buildings to withstand a higher level event such as a 2500 year return period or higher event. The following sections include examples of some of the research that the first author has been involved in over the last 20 years. Given that the damage incurred as a result of the Christchurch earthquake in February 2011 caused a shift in the philosophy of seismic design amongst many researchers in New Zealand and Australia, the research has been divided into that done before and after the Christchurch earthquake, with a section sandwiched in between on proposed performance objectives for design and assessment. The first author is currently working together with other researchers in Australia in the earthquake mitigation component of the CRC for Bushfires and Natural Hazards that commenced in 2015 and will continue over eight years.

4 EXAMPLES OF PRE-CHRISTCHURCH RESEARCH

A research project on reinforced concrete band beam frames designed in accordance with Australian standards as the primary lateral force-resisting system, as is commonly done in car parks around Australia, revealed that frames of two to eight storeys would be expected to remain elastic in a 500 year return period event, even though they had been designed with a response modification factor (R_f) of 4 in accordance with AS1170.4- 1993. This result was obtained using both a displacement-based assessment approach (Goldsworthy and Abdouka, 2012), which included a consideration of interior joints (Stehle et al, 2001), and in non-linear time-history analyses (Stehle et al, 2000). Using the displacement based assessment and a displacement spectra from (Faccioli et al, 2004) for a 2500 year return period earthquake in regions of low to moderate seismicity it was determined that band beam frames on rock sites and those with better detailing on intermediate sites would be expected to perform adequately, provided that a strong-column weak beam hierarchy was achieved in the frame. Such frames were not recommended for use on very soft soils.

It may have surprised some engineers to discover that the frames are unlikely to yield under the design level earthquake given that there is a ductility factor used in the design. By exploring the reason for this, it was possible to obtain some insight into the actual response of a building frame to an earthquake as opposed to the response expected on face value when using the equivalent load approach in AS1170.4 – 1993, the standard that was applicable at the time of this research. One reason for the structural overstrength is that in calculating the critical design moments at the ends of beams and columns, the gravity load combination (1.2G + 1.5Q) is often greater than the earthquake combination of (G + 0.3Q +E); also strength reduction factors used in design can create an even greater flexural overstrength. If the gravity load domination and the ensuing flexural overstrength were entirely responsible for causing the frame to remain in the elastic range then the situation would be as shown in Figure 5, and the frame would experience a base shear four times that which it had been designed for in accordance with AS1170.4-1993 (assuming that the band Beam Frame (BBF) behaves in accordance with the principle of equal displacements). If designers had checked the shear in interior columns and beam-column joints based on the earthquake forces that had been reduced by the response modification factor, they would have been on the low side by a factor of 4.

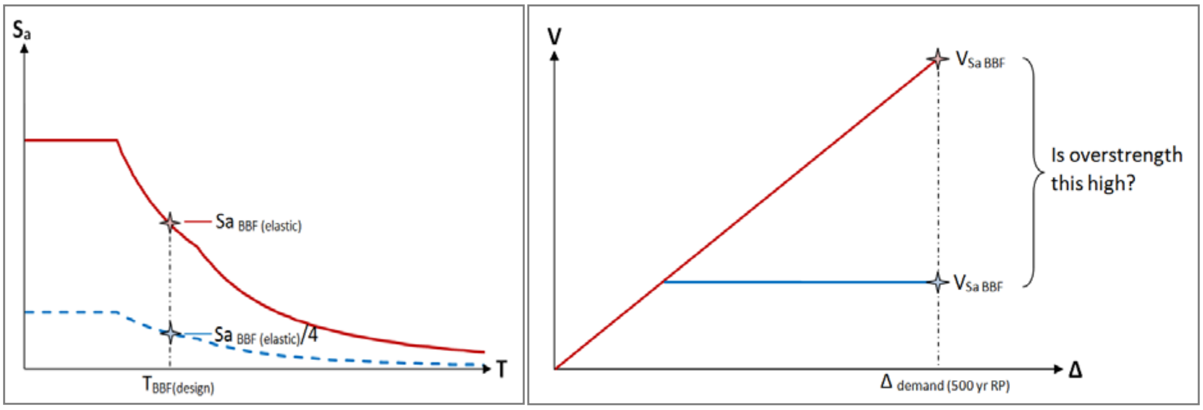

Figure 5: Overstrength of BBF assuming elastic response and period as in AS1170.4

What saves the situation somewhat is that the period of the bare frame estimated by the standard (see section 6 of AS1170.4-1993) is often considerably less that the first mode period obtained in a modal analysis, and the actual acceleration response of the bare frame is reduced as a result of this increased period.

Another point is that the reason the period is lower in the Standard is because of the assumed stiffening effect provided by non-structural components. The decrease in the period is warranted (although sometimes difficult to quantify) and the resulting increase in response acceleration should be taken into account in the design of diaphragms, connections between non-structural components and the building, connections between the diaphragm and lateral force-resisting structural elements, the foundations, and the building contents. In the actual earthquake the behaviour would be more complicated. The initial high stiffness would attract higher forces which would be taken by the combined resistance of the frame and the so-called “non-structural elements”, but as the non-structural components such as masonry infill and plaster walls cracked, and the stiffness of the overall structure softened, the response to the remaining ground motion would be closer to that of the bare frame (Mohyeddin et al, 2013).

Research has also been conducted on steel concentrically braced frames (CBFs) designed in accordance with AS4100 and used as the lateral force-resisting system (Wallace et al, 2002). In the Australian research (Wallace et al, 2002) test results revealed that if bracing connections with fillet welds were attached to braces with a member strength exceeding the connection capacity, little ductility could be expected. If capacity design principles had been used the brittle failure of a connection would have been prohibited by designing the end connections to be stronger than the brace element (taking brace overstrength into account); but that is not required in the Australian Standards. Non-linear time-history analyses of the case study frame (Wallace et al., 2002) revealed that design in accordance with AS4100 and AS1170.4-1993 was unconservative even for a 500 year return period event.

5 ESTABLISHMENT OF PERFORMANCE OBJECTIVES FOR DESIGN AND ASSESSMENT

5.1 Design of new buildings

After the Christchurch earthquake, engineers in both Australia and New Zealand have suggested changes to the current seismic design philosophy (Goldsworthy (2012), Buchanan et al., 2011)). The following performance objectives are proposed here by the first author for new designs:

For structures within Importance Levels 2 and 3 as specified by the BCA **(**Australian Building Codes Board, 2011**):**

CBD areas

- Damage control under a 2500 year RP event
- Collapse prevention under a 5000 year RP event

Non-CBD areas

- Damage control under a 500 year RP event
- Collapse prevention under a 2500 year RP event

For structures within Importance Level 4:

- Operational under a 1000 year RP event
- Damage control under a 5000 year RP event

In order to satisfy operational and damage control performance limits the strains in the structure would need to be limited (to limit damage to the structure itself), and there would need to be limits to the drifts in the building (to limit non-structural damage). In some situations floor accelerations would also need to be limited, for example when there is sensitive equipment that could be damaged.

The philosophy behind the proposed performance objectives is that within 100 years or so, when the more vulnerable buildings have been demolished, even a 2500 year return period earthquake close to the CBD of one of Australia's major cities would not result in the CBD being closed off for a long period. The experience in Christchurch, which was subjected to a very rare event for that city, was that the business disruption caused by shutting down the business district for a long period led to large economic losses (Goldsworthy, 2012). A reserve displacement capacity is also needed and this would be fostered by the adoption of the performance objective of collapse prevention under an even rarer event, for example a 5000 year RP event. This, and the previously specified performance objectives for new designs, are in line with the thoughts expressed by others (Walker and Musulin, 2015), although they are possibly less conservative in some instances.

5.2 **Assessment of existing buildings**

The following performance objective is proposed here by the first author for the assessment of all existing buildings:

- Collapse prevention under a 2500 year RP event

If this is not met, serious consideration should be given to adopting simple retrofitting strategies. It would also be desirable, but not essential, for the following objective to be met:

- Damage control under a 500 year RP event

6 **EXAMPLES OF POST-CHRISTCHURCH RESEARCH**

6.1 **RC Walls and Frames**

It is expected that many of the existing buildings that do not satisfy the assessment criteria given above will be those that were designed before the introduction of the earthquake loading standard, AS1170.4, in 1993. Nevertheless even those buildings designed in accordance with the various versions of the earthquake loading standard, even the latest edition AS1170.4-2007, may not, in some cases, satisfy the proposed assessment criteria. Also, as the performance criteria proposed above for new design is even more stringent, the current standards need to be investigated thoroughly to ensure that all new buildings meet these criteria. It is recognised by many earthquake engineers that the current force-based design methods such as those specified in AS1170.4 are inadequate for the reasons outlined in (Priestley et al, 2007). Nevertheless these methods are still the backbone of seismic design in many countries, including Australia, and, even if they are flawed, the question is whether the results produced are adequate.

The research on walls focuses on the performance of walls in buildings of 15 storeys or less with detailing commonly used in Australia, and more specifically on C-shaped core walls. There is a paucity of research on non-rectangular RC walls, and in particular on non-ductile RC walls in which the concrete is unconfined and strength hierarchies are not considered explicitly.

Design in accordance with AS1170.4-2007 assumes that there is a certain level of ductility associated with buildings designed, for example, in accordance with the reinforced concrete structures standard, AS3600, and the steel structures standard, AS4100. According to table 6.5(A) in AS1170.4-2007 a ductility factor of 2 and a structural performance factor, S_p equal to 0.77 can be assumed for “limited ductile shear walls”. Hence, the forces derived from the elastic acceleration response spectrum for a 500 year or 1000 year return period event that are used in design are able to be reduced by dividing by this ductility factor, μ. They are also reduced further by multiplying by S_p, which accounts for overstrength. However recent research (Henry, 2013) reveals that reinforced concrete walls with light reinforcement are not likely to form a series of well-distributed cracks at the base of the wall when subjected to in-plane bending. Instead the strain concentrates in a single crack at which the longitudinal reinforcement is likely to break as mentioned in the previous section of this paper. This phenomenon is the subject of a current PhD study at the University of Melbourne; it is a problem which is compounded by the strength of the concrete in the wall tending to be significantly greater than the characteristic compressive strength specified in design, and would also be exacerbated if low ductility reinforcing steel were used in the walls. The situation when the percentage of reinforcement is such that the cracking moment is greater than the ultimate moment capacity has been recognised previously as a problem (Goldsworthy and Gibson, 2012), but in the latest research it has been determined that the problem exists even for walls with a higher percentage of reinforcement.

The lack of consideration given to the consideration of strength hierarchies can lead to situations in which designers think they are being conservative, but, in fact, a brittle failure mode might be made more likely. An example of this is given here. In Australia, C-shaped core walls are often used. Moments about the minor axis are likely to control the amount of longitudinal reinforcement that is required in design, and hence the flexural capacity about the major axis will be considerably higher than that required in accordance with AS1170.4. The designer will usually have calculated lateral forces causing moments about the major axis due to earthquakes using the equivalent load method in AS1170.4 and hence using a μ/S_p factor of 2.6. However, given the large flexural overstrength, the wall is likely to remain elastic and hence a reduction in the maximum acceleration response due to ductility is not warranted. Flexure is not the problem here, but shear potentially is, also the capacity of the connection between the wall and the foundation and the foundations themselves may be inadequate. The shear force that needs to be checked is that due to the elastic response on the design response spectrum, with no reduction for ductility. Designers in New Zealand are well aware of strength hierarchies and the checks that need to be made to ensure that brittle failures do not precede ductile ones; this approach is part of the capacity design method that has been used in New Zealand since the 1980s and it is this approach to design that meant that many buildings in Christchurch were able to withstand the large displacements even though there may have been considerable damage in the regions that were designed to behave in a ductile manner.

Another problem that could arise in Australia is that walls designed for a 500 year return period event about the minor axis may have insufficient ductility to withstand the 2500 year return period earthquake (let alone the 5000 year return period earthquake which is proposed above as the earthquake level for which the collapse should be prevented for new buildings in the CBDs). When the walls are bent so that compression occurs in the two legs of the C-shaped section, there is a large tension capacity in the flange and compression failure could occur at the ends of the legs even before the steel yields in the flange. Alternatively there might be limited yielding but the ductility is likely to be low since the concrete is typically unconfined and hence the maximum compression strain is 0.003.

The vulnerability of walls with a range of key parameters such as different wall sections, reinforcement ratios, wall heights, and site soil conditions is being investigated in a project at the University of Melbourne in which fragility curves are being developed.

The research on frames focuses on the performance of ordinary RC frames detailed in accordance with the main body of the Concrete Structures standard pertinent to the particular time period in which the buildings were designed, and their ability to move with the walls when the overall building is subjected to earthquake ground motions of various levels. This research ties in with the issues on drift and displacement compatibility discussed in the previous section. It focuses on low to medium rise buildings and assumes that the assumption made in design would have been that the walls provided the lateral force-resisting system and the frames were designed to resist gravity-loading only. Prior to the advent of the Earthquake Loading Standard, AS1170.4 in 1993, it was not mandatory to design for earthquakes in most of Australia, so in some cases the lateral force considered would have been that due to wind only. In addition, the detailing used in the Australian Concrete Standards is non-ductile in that strength hierarchies are not considered, beam-column joints contain no or very few ties, the tie spacing in the beams and columns is high and the column bars are spliced a short distance above the floor in an area where the moments are likely to be high. Of particular concern are older buildings with highly eccentric cores, since the frames at the free edge may be subject to displacements considerably higher than those that would occur if the core were concentric.

6.2 **Composite frames with concrete-filled steel hollow sections as columns**

There is also an ongoing research project into a type of lateral force-resisting system for low to medium rise buildings that naturally performs well under current Australian seismic design conditions. This consists of a regular building with moment-resisting composite (steel/concrete) frames around the perimeter of the building which can also be supplemented by moment-resisting frames in one direction within the building. In this case concrete-filled square hollow sections are used as the columns, with steel beams that are composite with a slab that consists of metal decking with concrete on top. In order to provide a semi-rigid moment-resisting connection between the beams and columns, specially designed double T-stub connections that are anchored into the infill concrete within the column using blind bolts with headed anchors, in addition to through-bolts, are used. Provided that fixity against rotation can be achieved at the base these frames will be adequately stiff to satisfy the serviceability criterion for wind and also the drift requirements for the design level earthquake.

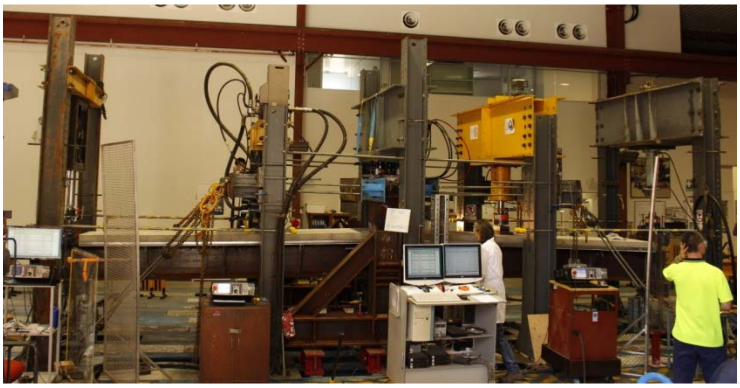

Figure 6: Large sub-assemblage test for composite frame with blind bolts

The period of the building in each direction is high due to the flexibility of the frames and the high seismic mass that they must cater for. It is likely to exceed the corner period on the displacement spectra of 1.5 seconds and a single degree of freedom equivalent structure would hence have a maximum response displacement equal to that on the plateau of the displacement response spectra. The current design level (500 year return period) peak displacement response in Melbourne is of the order of 60 mm for Site Class D and this can easily be tolerated by the flexible frames. At present these frames have been designed in accordance with the current Australian standards as well as European standards for composite structures where necessary, but these designs could be revised to meet the more demanding performance criteria that have been proposed above. At higher return periods the frames would have to tolerate even more displacement and the approach would be to design an element that yields within the connection, for example the flange of the T-stubs, so that the blind bolts and their anchorage into the concrete infill would be protected. The requirement would be that the connection between the T-stubs remained strong and stiff and the T-stub to beam connections would eventually yield to dissipate energy under a very rare event. Extensive testing has already been performed on components of this system, including a large subassemblage test as shown in Figure 6, and the results are promising.

7 CONCLUSIONS

There are faults that run through many Australian capital cities that have the potential to generate an earthquake of magnitude M_w 6 or higher. Even though the probability of a large earthquake striking close to the CBD of one of the capital cities is low, the damage from such an event is likely to be catastrophic, given the vulnerability of many existing structures, even ones designed in accordance with the current Australian design standards. Something that has been observed time and time again in past earthquakes is that if some basic seismic design principles have been adhered to, and there has been sufficient attention paid to the structural details, buildings generally have a much better chance of survival. Practical suggestions have been made here that sometimes exceed the requirements of the current Australian Standards, but are, nevertheless, important, if adequate robustness is to be achieved. In Australian research into the robustness of older buildings, and also of buildings designed in accordance with current codes, researchers have attempted to identify those structures that are likely to be the most vulnerable to earthquakes. Some research by the first author is presented here as well as proposals for performance objectives for design and assessment purposes. By applying these objectives researchers will determine to what extent changes are required to the Australian Loading and Materials Standards.

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