Moving towards displacement-based assessment in Australia

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ABSTRACT: The current Australian Earthquake Loading Standard AS1170.4-2007 is primarily a force-based design code. There is little attention paid to the displacement-based approach to design or assessment. It is possible to derive the displacement spectra from the acceleration spectra defined in this Standard, but these displacement spectra are significantly different (and often less conservative) to those proposed by others. The lack of a consensus on the displacement spectra that should be used, particularly for a very rare event, is a major obstacle to the use of displacement-based assessment of the lateral load-resisting systems in new or existing buildings. Another impediment is that seismic design philosophy in Australia does not give consideration to very rare events; this is not in line with the performance matrix type approach that has been adopted in New Zealand and elsewhere. However, it is the performance requirement of collapse prevention under a very rare event that is likely to govern seismic designs in regions of low seismicity. Given that the design philosophy is the foundation of the design edifice, the latest approaches to this are discussed in this paper. The author advocates the use of simple displacement-based assessments of the vulnerability of buildings to collapse, and presents some of the impediments to this which are illustrated by reference to previous case studies.

1 INTRODUCTION

In Australia, the present minimum performance objective for seismic design of most buildings (those with an importance level of 2 in accordance with the Building Code of Australia (ABCB 2011)) is that the building remains "life safe" for an ultimate design level event with a 1 in 500 annual probability of exceedance. This would usually be said to be a "rare" event and corresponds approximately to a 10% chance of exceedance in 50 years (approximately a 500 year return period) and a higher probability of exceedance if the building's expected life is higher than 50 years, which is often the case. There is no requirement in the building code, or in the general seismic design philosophy in Australia, that requires a very rare event to be considered (except for buildings that are deemed to have a higher importance). This philosophy allows designers to continue to design non-ductile buildings that may be capable of withstanding a low magnitude 500 year return period event, but may not have the reserve displacement capacity to withstand a very rare event.

After the Canterbury earthquake sequence and the resulting devastation some New Zealand academics and practitioners (Buchanan et al 2011) have proposed that the New Zealand seismic design philosophy be radically changed such that an "operational" performance level is the objective for most buildings in the event of a rare, or even a very rare, earthquake. The Christchurch earthquake of February 22nd, 2011 was a very rare earthquake event for that city. Most of the 182 fatalities occurred in two reinforced concrete buildings, the Pyne Gould and the CTV. In contrast to these catastrophic failures, many other reinforced concrete buildings did not collapse; most of these had been designed in accordance with capacity design principles and detailing methodologies, as required in Christchurch after the mid-1970s, and performed in a ductile manner as expected, with substantial plastic deformation experienced in certain regions of the structures. However, plastic deformation equates to damage, and this damage has been sufficient in many cases to justify the demolition of the buildings. It is because of these economic imperatives that some engineers are advocating that more demanding performance objectives be adopted. This approach, which relies on the use of new "damage-resistant" technologies and/or the imposition of a higher design loading, would be expected to lead to a better performance in terms of the overall community even though it is based on objectives for individual buildings. If this new design philosophy is adopted in Christchurch, the massive disruption to services and business activities that occurred in the aftermath of the February 22nd 2011 Christchurch earthquake, mainly due to closing off of the CBD for an extended period, would be much less likely in the event of another very rare earthquake. If applied in New Zealand more generally, in the long term it would yield a similar benefit.

If a very rare event were to strike in one of the capital cities in Australia, the effect on the community could be as, or more, devastating than that in Christchurch, due to the vulnerable nature of many buildings (Goldsworthy 2012). Risk assessments are required to assess whether the Australian design philosophy needs to be changed; for example to include a performance objective of collapse prevention under a very rare event. Within these proposed risk assessments, building vulnerability studies are needed to determine the level of damage expected for a given earthquake level. This is usually done on a probabilistic basis using fragility curves. As a preliminary approach simple displacement-based assessments are a useful tool to determine which types of construction currently in use are likely to be resilient. These displacement-based assessments would also be useful in determining whether a newly designed building is likely to collapse in a very rare earthquake. The eventual aim is to guide designers away from non-resilient types of construction. However, there are impediments to the use of displacement-based assessment in Australia that need to be overcome before this method can be used with confidence.

2 AUSTRALIAN SEISMIC HAZARD

Although Australia is located in a region of low to moderate seismicity, it is one of the most active intraplate regions in the world due to strains created by the Indo-Australian plate colliding with the Eurasian and Pacific plates. GPS data shows the Australian Plate moving very fast to the north/ northeast, at about 70 mm/year. Tectonic stress measurements show that almost all of Australia is experiencing a high level of horizontal compression and most faults show reverse faulting. A consequence of high compressive stress is that earthquakes can occur to very shallow depths. Most of the Australian capital cities have known faults in their vicinity that are capable of generating damaging shallow earthquakes; historical earthquakes of magnitude 6 or higher in Australia have been caused by ruptures on shallow reverse faults. The Canterbury earthquake sequence has considerable similarities with the continental earthquakes experienced in Australia in that the Canterbury earthquakes were caused by high compressive stress, occurred at very shallow depths, and produced many aftershocks lasting for a prolonged period.

3 AUSTRALIAN SEISMIC DESIGN PHILOSOPHY

As outlined in the introduction, in Australia the current performance objective for most buildings is to achieve “life safe” performance or better under a rare earthquake event. The inter-storey lateral drift under this earthquake level is limited to 1.5% in accordance with the Australian Earthquake Loading Code AS1170.4 (Standards Australia 2007) and this is intended to limit damage to both structural and non-structural building components. The design is typically “force-based” with the design level earthquake "forces" usually determined by carrying out an equivalent static analysis in accordance with Chapter 6 in AS1170.4 (Standards Australia 2007). The design Peak Ground Acceleration (PGA) in Australia's capital cities for a rock site is between 0.08g and 0.1g, compared with a design PGA for Auckland of 0.13g and for Christchurch of 0.22g. It is important to note that PGA values of 0.5g to 0.7g were actually realised in the Christchurch CBD on February 22nd 2011.

In Australia, for most buildings, there is no explicit provision made for a higher level event, i.e. a "very rare" earthquake event as opposed to the "rare" event that is considered in the ultimate limit state approach. However, as happened in Christchurch, it is the very rare event that could cause major damage, potentially rendering the CBD unusable for a long period. This is exacerbated in Australia by the fact that material design standards such as the Steel Structures code (Standards Australia 1998) and the Concrete Structures code (Standards Australia 2009) do not require designers to use capacity design principles in their design; yet the implementation of these design principles in New Zealand (since the 1980s) saved many lives in the Christchurch earthquake. Due to the lack of thought given in Australia to strength hierarchies within a building, and the failure to incorporate weak ductile zones that allow the building to safely deform into the plastic range, the performance of some buildings is likely to be poor in a very rare earthquake event. The robustness clause in AS1170.0 (Standards Australia 2002) is intended to ensure that the damage caused by an event is not disproportionate to the magnitude of that event. The question here is what type of damage would the structural engineering community view as appropriate for a very rare earthquake event. It is the author's opinion that "collapse prevention" is the minimum performance objective that should be considered.

4 RISK-BASED ASSESSMENTS AND CHANGES TO THE DESIGN PHILOSOPHY

Recent U.S. practice (FEMA450-2 2003) attempts to create a uniform margin against collapse at the design ground motion for all regions across the U.S. It has been recognised for some time that there is a larger ratio of the level of ground motions experienced in a 2500 year return period event to those in a 500 year return period event in regions of low to moderate seismicity, as compared with regions of high seismicity. To account for this, the ground motion hazards in the U.S. have been defined in terms of maximum considered (MCE) ground motions. A lower bound estimate of the margin against collapse in structures designed to the seismic provisions in the U.S. standard has been taken as 1.5 in terms of ground accelerations. Hence, the design earthquake ground motion was selected at a ground shaking level that is 1/1.5 (or 2/3rds) of the MCE ground motion. For most regions in the U.S. the MCE ground motion has been defined with a uniform probability of exceedance of 2 per cent in 50 years (return period of about 2500 years). There is a later modification (American Society of Civil Engineers 2011) in which the MCE terminology has been replaced by "Risk-targeted MCE". In this case the mapped ground motions were developed on the basis of risk of collapse, however, the values themselves have only changed slightly.

There is no explanation given in the U.S. approach of the basis on which the probability of exceedance of 2% in 50 years was chosen; a 0.5%, 1% or 5% probability could equally well have been chosen. A more rational approach to determination of the appropriate return periods has been advocated (Walker et al 2011 and Walker et al 2012). They recognise the non-linear relationship between the impact of disasters on society and the magnitude of the disaster. Given that risks that are acceptable to individual building owners might not be acceptable to the community (as was the experience in Christchurch), they consider design philosophies based on individual building performance objectives to be inadequate. Instead they propose to regard return periods for design as variables that depend on the expected population within the design event area over the expected building life. In general, buildings in cities with large concentrations of people/infrastructure may need to be designed for higher return period events and there would also be likely to be higher standards in these areas for the retrofitting of existing buildings. This would be determined by conducting community-based cost/benefit risk assessments for a variety of scenarios.

A similar approach is advocated by the author and two of her colleagues, Gary Gibson and Paul Somerville, for the assessment of the costs and benefits associated with proposed changes to seismic design philosophy in Australia. Their approach is to focus on the CBD of Australian capital cities and to use a Catastrophe Loss Risk Assessment tool from the insurance industry to assess the expected overall damage to each community on an average annual basis for a variety of earthquake hazard levels (in terms of structural and non-structural damage, downtime, and deaths). The revised situation due to the hypothetical implementation of proposed design philosophy changes would be assessed on a probabilistic basis using the risk assessment tool, and the probable cost of these changes would be estimated on an average annual basis, to determine if there would be likely to be a net benefit to the community over a chosen number of years. The types of changes to the design philosophy that would be investigated would include that of implementing a performance objective of collapse prevention under a very rare event, or requiring retrofitting to a certain level for existing buildings that would be likely to perform poorly under a rare event. Future likely development scenarios in the CBD areas, maintenance costs and insurance implications would need to be considered. Given that a human life is equally important in a small country town as in a large city, the design return period corresponding to the collapse prevention performance level is expected to be the same throughout Australia, however, because of the scale of the possible economic and social disaster in the CBD areas, the design return period corresponding to operational or damage-control type performance levels would be likely to be higher in the CBD areas.

The end result may be to maintain the status quo, however until these assessments are carried out using robust vulnerability models of a range of structural systems it is not possible to say.

5 DISPLACEMENT-BASED ASSESSMENT OF THE VULNERABILITY OF BUILDINGS TO COLLAPSE

5.1 Necessary Information

In order to conduct displacement-based assessments of the viability of different types of structures in Australia under a very rare event the following are needed:

1) Displacement spectra representing a very rare event that can be used to estimate the displacement demand.
2) A reasonable representation of the lateral load-resisting system
3) An assessment of the displacement capacity of the system

These points will be discussed here in relation to case studies conducted on non-ductile reinforced concrete walls, non-ductile reinforced concrete band beam frames, and on a newly proposed composite framing system to illustrate some of the difficulties encountered when using this approach.

5.2 Case Studies

5.2.1 *Description*

In Australia, low to medium rise reinforced concrete buildings typically rely on a core that consists of two walls in each direction to withstand the loads due to wind and earthquake. The core is usually placed centrally in the building. Reinforced concrete frames in buildings with substantial core walls, although moment resisting, are typically considered to be more flexible than the core walls, and are not usually designed for lateral loads. The type of detailing used in these reinforced concrete buildings for both the walls and the frames has limited ductility (it would be classified as “non-ductile in New Zealand). In the case study on walls (Goldsworthy and Gibson 2012), displacement-based assessments were used to determine the vulnerability to collapse of low to medium rise reinforced concrete buildings that use non-ductile reinforced concrete walls as the lateral load-resisting system.

In a previous case study (Goldsworthy and Abdouka 2012), the vulnerability to collapse of reinforced concrete band beam frames consisting of reinforced concrete columns and very wide and shallow beams (band beams) with detailing in accordance with the usual Australian practice (and also with suggested improved detailing) was also assessed using a displacement-based approach.

In a recent case study (Agheshlui et al 2012) a 5-storey office building located on a soft soil site in Melbourne has been designed in accordance with Australian Standards and then assessed using a displacement-based approach to examine its performance under a 2500 year return period earthquake. It uses a lateral load-resisting system consisting of moment-resisting perimeter frames with steel beams connected to concrete-filled steel tubular columns using semi-rigid blind-bolted connections.

5.2.2 *Displacement Spectra*

In the displacement-based code in the appendix of (Priestley et al 2007) the authors suggest that the seismic design for buildings in regions of low seismicity are likely to be governed by the very rare earthquake event. In that case they define this as the 2500 year return period event. Despite the comments made above about the possible need to consider an even higher level event for built-up areas or those with a high population density, further studies are needed to provide this information, so the 2500 year return period event will be considered here to represent the "very rare" earthquake.

The displacement spectra used in the wall case study were derived by Gary Gibson (senior research fellow in Earth Sciences at the University of Melbourne) using the latest techniques available

(Goldsworthy and Gibson 2012). The vulnerability was assessed for the 475 year and 2475 year return period level of hazard in the Adelaide CBD. Adelaide, the capital city of South Australia, was chosen for this case study since it has one of the highest degrees of seismic hazard of the capital cities in Australia and also has unfavourable deep and/or soft soil at many sites in the CBD.

There is insufficient strong motion data to allow development of a ground motion model specifically for the Adelaide area. A range of ground motion models, determined using data from elsewhere, were used to select the most relevant for Adelaide. Out of the ground motion models developed using Australian data the Somerville 2009 Cratonic model was the most logical choice for the reasons given in (Goldsworthy and Gibson 2012). Prior to the development of the so-called Next Generation Attenuation (NGA) ground motion models, the response spectral models developed using internationally available data gave acceleration response only. NGA ground motion models included many improvements, one of which was to improve displacement motion estimates. Results for the three NGA functions that are considered appropriate in this study (Abrahamson and Silva 2008, Campbell and Bozorgnia 2008, and Chiou and Youngs 2008) have been computed. The four ground motion models (including the Somerville model discussed above) have each been applied to all relevant source zones and active faults, so results can be compared. The resulting displacement spectra for the 2475 year return period level hazard on deep and/or soft soil sites (V_{s30} = 257 m/s) are shown in Figure 1. It is clear that there is considerable variation between the values obtained using these different ground motion models. In the wall case study the Somerville 2009 non-cratonic model was used since it mostly provided an upper bound to these results.

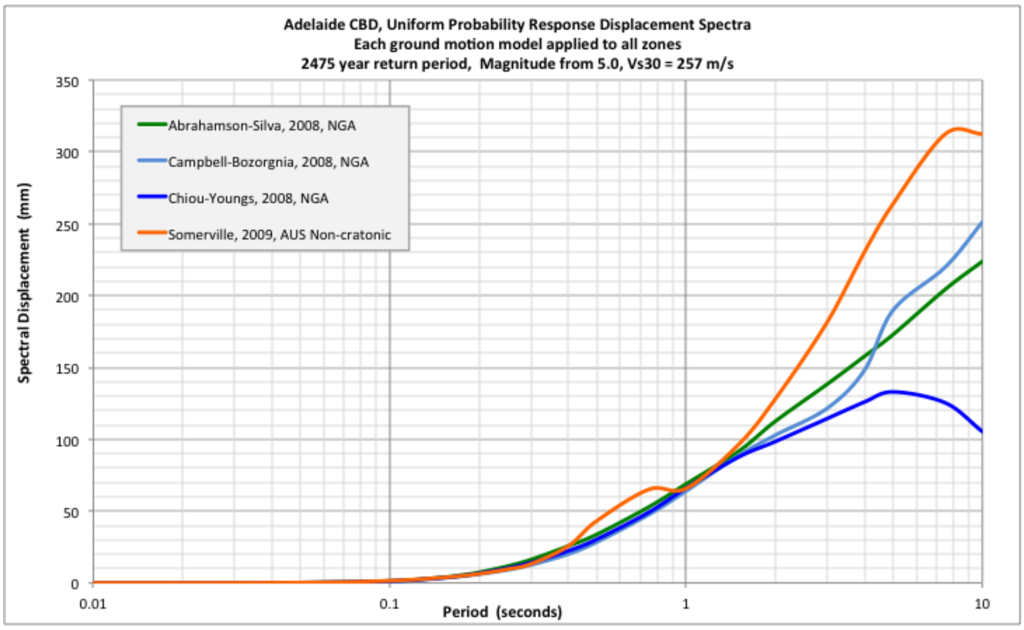

Figure 1 Elastic Displacement Response Spectra, Adelaide, Soft soil (Goldsworthy and Gibson 2012)

It is interesting to compare the spectra in Figure 1 with other design displacement spectra. In the band beam frame case study (Goldsworthy and Abdouka 2012), the 2500 year return period event was assumed to be a 7.0 Moment Magnitude earthquake producing a PGA of 0.2g at a firm ground site. The elastic displacement spectra (5% damping) was constructed using a corner period of 4.2 seconds, a maximum elastic displacement response at the corner period of 258 mm for firm soils, and a scaling up factor for very soft soils of 1.8, all of which were determined using formulae presented in (Priestley et al 2007) and (Faccioli et al 2004). An approximate bi-linear representation of this design displacement spectrum is shown in Figure 2. This spectrum is consistently higher than Somerville Cratonic spectrum in Figure 1, by a factor of between 1.2 and 1.8 in the period range between 0.5 and 5 seconds. If the moment magnitude of the earthquake causing the 0.2g PGA at the site were reduced (i.e a smaller earthquake at a closer distance), the initial slope of the displacement spectrum would remain the same, but the corner period and its corresponding displacement would be reduced (for example to 3 seconds and 328 mm (very soft soil site) respectively for M_w = 6.5).

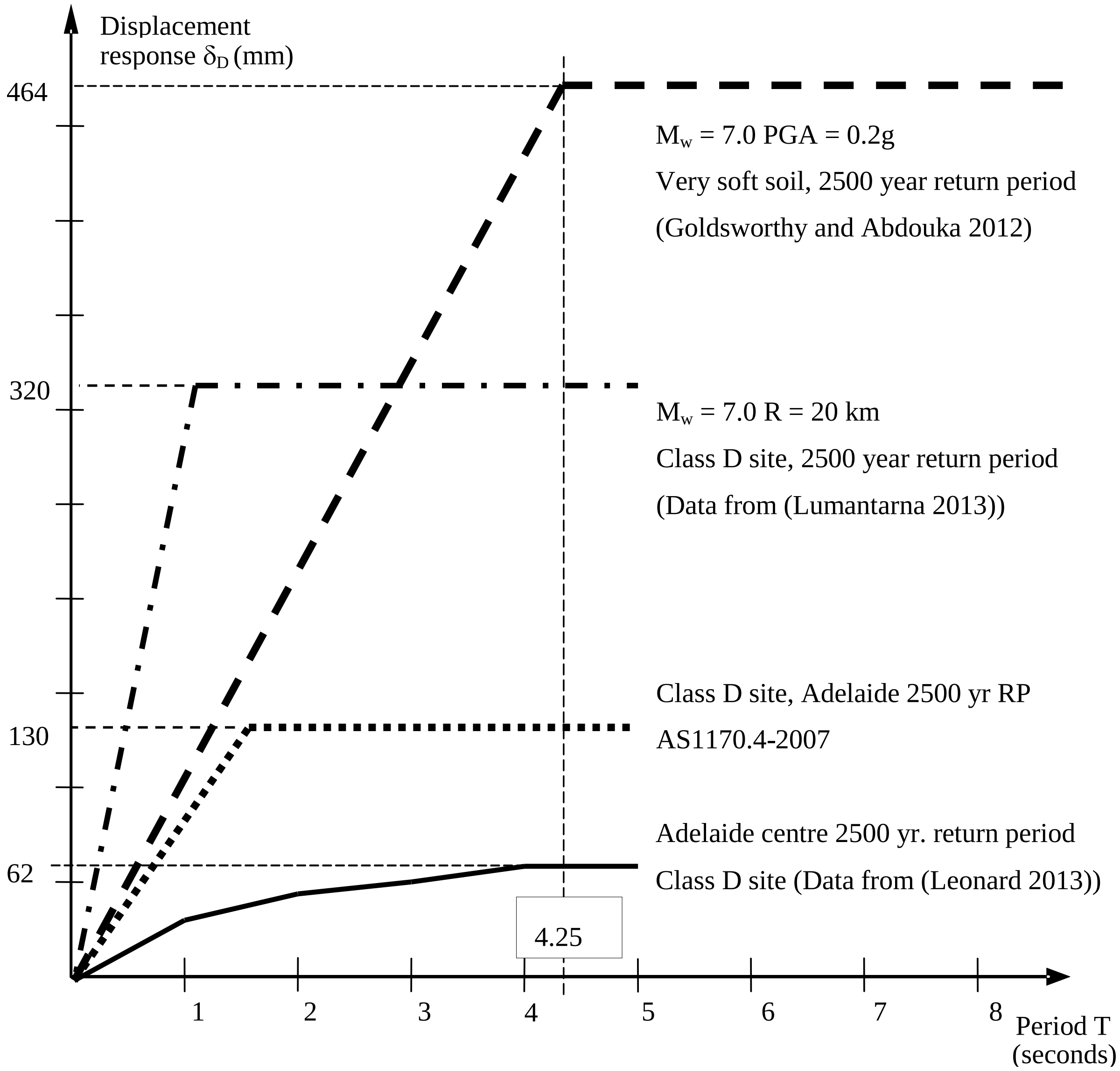

Figure 2 - Elastic Displacement Response spectra, Adelaide, Soft soil, 2500 yr RP, various sources

There is a considerable variation in the estimation of the corner period by different researchers, and this is an ongoing matter of debate (Lumantarna et al, 2010). A value of 1.5 seconds is used in AS1170.4 (Standards Australia 2007) and the code-compatible design displacement response spectrum for a Class D site in Adelaide, and with a 2500 year return period, is shown in Figure 2. The most recent Australian earthquake hazard map that has been developed by Geoscience Australia (Geoscience Australia 2012) has been used to generate acceleration response spectrum data (Leonard 2013) from which a design displacement response spectrum has been determined for a comparable site and return period, and this is also shown in Figure 2. It gives much lower values than any of the other models. A spectrum proposed by (Lumantarna 2012) is also shown in Figure 2. It has a higher initial slope than the spectra proposed by others (and hence is more conservative for lower periods), since the corner period is only 1.0 seconds (this corresponds to the site period).

Another issue that has been identified in the composite frame case study (Agheshlui et al 2012) is the questionable accuracy of calculating displacements and drifts using the AS1170.4 equivalent lateral force method. These values have been shown to be much greater than those calculated using the response displacement spectrum that is consistent with the acceleration spectrum used in the equivalent force method. Two reasons are given for this (Agheshlui et al 2012):

1) The equivalent lateral force method has not introduced any upper limit for calculated displacements

although the displacement spectrum has a clear cut-off at the corner period.

2) The AS1170.4 method assumes a shorter period for the case study building (1.2 seconds versus 1.87 seconds). Therefore a higher level of base shear is calculated for a stiffer structure and then applied to the bare frame model which is less stiff. This scales up the response displacements by a factor of $(T_{bare\ frame}/\ T_{code})^2$. This issue has not been addressed in AS1170.4 , however ASCE7 (2005) permits the elastic drifts to be calculated based on the fundamental period of the structure.

5.2.3 *Representation of the lateral load-resisting system*

In the case studies, gravity load-resisting elements and non-structural components have not been included in the structural analysis for earthquake response. However, it is recognised in (Goldsworthy and Gibson 2012) that it is important to take a holistic approach to the assessment of the building. For example, in the wall case study, given that columns and beam-column joints in the gravity-loaded frames have non-ductile detailing, the actions in these regions and the interaction between the frames and the walls need to be investigated when the building is subjected to the extreme deformations due to a very rare earthquake. Another consideration is that there may be torsional effects that would increase the drifts experienced by secondary framing, especially if the core were placed eccentrically. As has been witnessed in past earthquakes, failure in these "gravity load-resisting" elements can lead to catastrophic collapse. Also, the behaviour of the so-called non-structural elements would need to be quantified in a more sophisticated analysis. For example, if masonry infill is used in any of the frames this could have a pronounced influence on the structural behaviour (Mohyeddin et al 2013). Although this is recognised in the commentary to AS1170.4, it has not been well quantified.

5.2.4 *Displacement capacity*

Estimates of the displacement capacity of the lateral load-resisting system at the "near collapse" condition are often based on crude estimates such as the displacement at which the lateral load has dropped by 20 percent from its maximum value in a pushover analysis. In the case studies mentioned here, simple calculations have generally been performed based on specified material strain limits at the "near collapse" state. The elements and connections in the lateral load-resisting systems themselves have been considered although it has been recognised that a limit may need to be imposed on the drifts in the secondary (gravity load-resisting) framing systems as has been discussed previously.

6 CONCLUSIONS

- Because of the potential vulnerability of construction in Australia to very rare earthquakes, risk assessments are needed to determine if changes to the design philosophy can be justified. Due to the non-linear relationship between the impact of disasters on a society and the magnitude of the disaster, densely populated areas and built-up areas need special consideration.
- Displacement response spectra for the assessment of a building on a soft soil site in Adelaide have been derived from various sources. There is a significant variation between these. Some consensus is needed to establish which spectra are suitable for use in simple displacement-based assessments of buildings. The "very rare" event is of particular interest since the performance requirement of collapse prevention under this level of event is likely to govern.
- Case studies have been used to demonstrate problems with the current determination of displacement response spectra, estimation of drift demands in accordance with AS1170.4, and general problems with characterisation of structures such as the incorporation of the effects of non-structural elements. The assessment should consider the susceptibility of gravity load-resisting systems to drifts that are largely controlled by the lateral load-resisting system.

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