Performance Objectives for Low Damage Seismic Design of Buildings

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ABSTRACT: Low damage design is emerging as a way forward for building designers to implement damage reduction measures into seismic design. Although some low damage measures can be incorporated into conventional structural systems, most research is concentrated on developing new structural systems or devices which can deliver improved building performance. However, not all of the proposed systems will provide low damage outcomes, in the broadest sense.

The performance of buildings in the Canterbury earthquakes is discussed in relation to desired future outcomes, with an outline of the required limit states to be considered. A set of general performance objectives is proposed that such systems can be measured against. Design methodologies are discussed, noting that current Building Code approaches will generally not provide adequate verification methods. In addition, some consideration of future development criteria is offered, with the aim of ensuring that the credibility of the industry is maintained.

1 INTRODUCTION

1.1 General

Since the mid-70's, seismic resisting design has commonly followed a capacity approach, using ultimate limit state (or predecessor) design principles. The majority of designers have used a force-based approach, and the increasing accessibility and capacity of desktop computing has evolved to the point where many practices have the capacity to perform linear dynamic analysis of complex structures.

Early computer analysis typically was limited to the primary lateral load resisting structure, due to the limited capacity and accessibility of computers. This usually required designers to consider gravity loading and 'secondary' (non-lateral load resisting) structure independently of the lateral structure. This has had several implications for the resulting building designs, both positive and negative. Some include:

- Designers were therefore more directly and closely involved in the structural design and hence may have been more attuned to the building behaviour, although not necessarily in respect of the secondary 'gravity-only' structure.
- Where there was significant secondary structure, it may have provided additional lateral load resistance 'for free'.
- Conversely, although the secondary structure would be required to undergo the same displacement as the primary lateral load resisting structure, it was often not detailed for the same level of deformation capacity. This may therefore have resulted in an effective reduction in the overall performance.
- Significant design effort was required for the design, compared to what can now be achieved in a relatively short time with modern computers and software (noting that the actual analysis

engines driving the software have changed little over the last 30 years)

The subsequent improvements in desktop computing capability have moved conventional computer analysis from a niche skill to the structural engineering mainstream. Whole structures are usually modelled, including gravity loads. Many designers have access to automated design processes that allow variations of structural forms and ductility to be investigated and designs to be refined considerably beyond what was previously possible.

Capacity design has proven generally reliable in providing designs that meet the life safety performance objectives of the loading standard, but the often neglected truth is that these buildings are 'designed to fail'. Our engineering training had led us to believe that such buildings would be generally repairable following a minor earthquake but may require demolition after a major event. Communication of this to building owners and the general public was never fully achieved. Owners in particular either could not or would not understand the implications of this.

Recent experience in Christchurch has shown that following a major earthquake it is seldom economic to repair a ductile building that was designed following the principles of capacity design. This is an emerging picture, with considerable research required, but issues such as dynamic loading effects, low-cycle fatigue and the impact of concrete aging are a few of the possible contributing factors. However, even the repairs after the more moderate event of September 4 were not as simple as might have been expected.

More recently, alternative design methods have been proposed that may produce different outcomes. Whether it is termed Damage Avoidance Design (DAD) or Low Damage Design (LDD), the objectives are generally the same – to design new forms of lateral load resisting structure where damage is either suppressed or limited to readily replaceable elements, removing or reducing the expensive downtime and even the need to replace buildings that have been severely damaged in an earthquake.

The concept has in fact been in use for a long time. Base isolation has been used for buildings and bridges since 1978, and is arguably the original form of LDD in modern design.

A common factor in the past for predecessor forms of LDD has been the need to complete significantly more sophisticated forms of analysis and design, often incorporating non-linear time history analysis. Such complex analysis is not merited for conventional design, where the building behaviour can be controlled by the designer. However, the need for more sophisticated analysis has in part lead to methods such as base isolation becoming a niche activity.

LDD is now being proposed by some as a widespread substitute for conventional systems. However for this to happen, designers need to be given the means to readily complete such designs, and need to understand what it is that they are trying to achieve. A clear set of performance objectives needs to be agreed, and importantly, developers and builders of such structures need to have a clear and reasonable expectation of the building's likely behaviour. Arguably, the profession cannot allow a failure of communication of design outcomes to be repeated.

1.2 **Canterbury Earthquakes**

The Canterbury Earthquakes have prompted a need for significant reconsideration of the design process and performance objectives.

1.2.1 *SLS behaviour*

The Darfield (September 4th, 2010) earthquake is the most relevant of the Canterbury events for consideration here, noting that although it exceeded SLS1 limits for the pre-earthquake loading standard (R = 0.25, Z = 0.22), it is a reasonable approximation of the new standard (R = 0.33, Z = 0.3). One of the more concerning outcomes was the considerable amount of damage done in modern structures, generally to non-structural elements. Although estimates were never completed, many buildings suffered damage requiring weeks or even months of repair, some of which caused significant disruption to users through having to vacate space while work was completed. In the worst cases, this may have involved closure of buildings. Examples of the damage include:

- Significant cracking of precast floor systems (in the worst case leading to fracture of cold-drawn wire mesh reinforcement).
- Damage to partitions. The most critical damage was to firewalls, especially around fire escape stairs, requiring repair in order to restore fire cell integrity
- Failure of ceiling systems. In the extreme, entire ceilings systems fell, representing a significant hazard to occupants

The question this prompts is whether LDD design should be able to avoid or minimise this level of damage in what could have been considered an SLS event for much of the CBD.

1.2.2 *ULS behaviour*

The February 22nd, 2011 event was clearly a ULS level event (or larger) for the CBD, at least in terms of the ground accelerations, if not duration. As such, it was to be expected that there would be significant damage to modern buildings, even to the point that many would require demolition. However, it is important to note that with one exception, buildings designed since the introduction of capacity design did not fail to meet the basic life safety objective. The question could be asked though, whether more buildings may have failed in a significantly longer duration event.

As might have been expected, this provides little comfort to owners, tenants or insurers. Although it could be argued that this had been communicated and was a societal choice, the message had never been fully understood. Interestingly, owners on the whole are relatively content with replacing their aging buildings through insurance. However, the inconvenience to tenants and the overall impact on the country's economy of this is a more concerning issue. Although the economy will survive it, it cannot afford to have another similar event in the near future.

The question this prompts is whether there needs to be more focus on reduction of damage in such an event, particularly in the larger centres, where the overall impact of losing this concentration of buildings with subsequent disruption to the economy may not be an acceptable outcome. Specifically, can LDD design sufficiently reduce the impact of this level of damage to the point where the CBD could have returned to near full business activity in a relatively short period of time?

1.3 **MCE behaviour**

The Maximum Considered Earthquake (MCE) is not currently a definition that appears in the New Zealand Building Code (NZBC), apart from the commentary to AS/NZS1170.5. However it is used here for convenience, in reference to the code requirement of collapse avoidance in larger earthquakes.

Many have stated that the February 22nd event approximated the MCE for much of Christchurch, from the CBD to the east, including Lyttelton. If this is accepted, then the overall building performance (arguably) easily met the Building Code requirement, of low probability of collapse in such an event. Consideration may however be given as to whether the earthquakes have really been at this level – although the ground accelerations may have been, the duration was considerably shorter than would normally be expected for a MCE event. There is plenty of evidence that more buildings may have collapsed, had the duration been significantly longer.

The question this prompts is whether LDD systems can be demonstrated to meet this level of performance demand, ie acceptably low probability of collapse in a significantly longer duration shaking.

1.4 **Special characteristics of Christchurch and the earthquakes**

In considering these questions, it is worth noting that there are several aspects to the Canterbury earthquakes that may be unusual or unique.

Firstly is the nature of the earthquakes, with unusually high ground accelerations from relatively low magnitude events. This is of note as the impact of such events is relatively concentrated, but the CBD just happened to fall within the area of greatest impact. Should this be considered something that could happen anywhere, or is it an aberration? Observations are that most damage occurred within about a 15 km radius of the epicentre. This is coincidentally within the radius noted as the basis for the minimum seismic actions of AS/NZS1170.5 (noting that the minimum seismic loading is derived from the 84th percentile actions of a M6.5 earthquake at 20km radius). Accepting that, if a circle of 15km radius is drawn around the centre of Auckland, it could include the CBD, Waitakere City, Manukau City and the North Shore city centre. Given that Auckland's seismic hazard is approximately 40% of that in Christchurch, the result would clearly be more catastrophic, as all buildings would have significantly less strength. Added to that, the EPB threshold load is such a small proportion of this as to be hardly significant.

Secondly is the deep soil profile and possible basin effects in Christchurch, that have resulted in high period amplifications, liquefaction and high settlements. On one hand, this may have reduced lateral loads (by providing a form of base isolation), but on the other, it has resulted in many buildings having unacceptable differential settlements or lateral displacement, that otherwise performed well structurally. Clearly any LDD systems must take into account the ground performance.

Thirdly, is the probability that longer durations of earthquake in Christchurch will most likely come from more distant fault sources. Hence it is likely that the ground accelerations will be lower. Conversely, the possible basin effect may be more pronounced, possibly leading to higher amplification of the long period response. This was noted in Christchurch, in that for some sites, the long period response from the Darfield earthquake was proportionally higher measured against the pga than for February 22nd.

Finally, is the level of insurance carried, across all building types. It has been noted that in excess of 80% of properties carried earthquake cover, as opposed to international trends of more like 20%. This has likely led to demolition of buildings that would have been repaired in other countries. Changes in the New Zealand insurance market may see levels of insurance cover reduce as insurance premiums and excesses increase.

These factors could be taken into account when considering the possible benefits of LDD, particularly in the case of soil-structure interaction. There is little point in putting forward LDD systems that are not matched by an equivalent level of performance in the foundations, and arguably, in the surrounding infrastructure.

2 BUILDING PERFORMANCE LIMIT STATES

When assessing the seismic performance of low-damage design (LDD) systems the following building performance limit states should be considered:

- Serviceability Limit State (SLS)
- Damage Control Limit State (DCLS)
- Ultimate Limit State (ULS)
- Collapse Limit State (CLS)

These building performance limit states are described in more detail below.

2.1 *Serviceability Limit State*

Two serviceability limit states are defined in NZS 1170.5 (SNZ, 2004), these are:

(i) SLS1. Damage shall be avoided to 'the structural and non-structural components that would prevent the structure from being used as originally intended without repair.' and

(ii) SLS2. Applies to critical post disaster designation buildings only (i.e. Importance Level (IL) 4 buildings). 'All elements required to maintain those operations for which the structure is designed as critical, are to be maintained in an operations state or are to be returned to a fully operational state within an acceptable short timeframe (usually minutes to hours rather than days)'.

The SLS loads are defined in clause 2.4, and for IL2 (regular buildings), SLS1 equates to the 1 in 25 annual probability of exceedance. For IL4 buildings, SLS2 equates to the 1 in 500 annual probability of exceedance. This is generally the governing design criterion for an IL4 building.

2.2 *Damage Control Limit State*

Consideration of the damage control limit state (DCLS) is not required by NZS 1170.5 (SNZ, 2004). The damage control limit state has been defined by Priestley et. al., (2007) as the limit state whereby a certain amount of repairable damage is acceptable, but the cost of repair should be significantly less than the cost of replacement.

Damage to reinforced concrete buildings may include minor spalling of cover concrete requiring cover replacement, and the formation of residual flexural cracks requiring injection grouting. Inelastic strain in reinforcing steel is limited, to ensure adequate performance in future earthquakes. For structural steel buildings, flange or shear panel buckling shall be avoided.

Building drifts should be limited such that damage to non-structural components is limited to acceptable levels at the specified earthquake hazard level.

As this is not a limit state defined by AS/NZS1170.5, there is no pre-determined load level. Therefore this limit state becomes a matter for discussion with the building developer. It is expected that the damage control limit state would be set somewhere between SLS1 and ULS requirements. A reasonable correlation could be made between SLS2 and DCLS for many buildings.

2.3 *Ultimate Limit State*

Functional requirements for the ULS are defined in NZS 1170.5 (SNZ, 2004). These are:

‘(i) Avoidance of collapse of the structural system; and

(ii) Avoidance of collapse of loss of support to parts of categories P.1, P.2, P.3 and P.4; and

(iii) Avoidance of damage to non-structural systems necessary for emergency building evacuation, that renders them inoperative’

2.4 *Collapse Limit State*

Consideration of the collapse limit state is not required by NZS 1170.5 (SNZ, 2004), nor is any specific guidance provided in the standard in terms of what defines the CLS. However for new buildings, designed to modern design standards with ductile materials and configurations, NZS 1170.5 assumes that a margin of at least between 1.5 and 1.8 exists between ULS and CLS.

For building systems that are not covered by existing material standards, care needs to be taken to ensure that similar margins of safety exist beyond the ULS.

Additional consideration should also be given to how the building might behave for those LDD systems which have a hard limit (i.e. isolators where the rattle space is exceeded or dampers when the maximum stroke is exceeded) if the CLS displacements are exceeded.

3 **OBJECTIVES OF LDD**

The basic objective of LDD must be a form of structure that will survive a DCLS (which may be ULS) design level event with minimal damage and which may be reasonably easily and economically repaired, with minimal disruption to the building users. However, this needs some expansion, noting the recent experiences in the Canterbury earthquakes.

The effectiveness of low damage design systems may be characterised by the following properties:

- Damage mitigation effectiveness

- Reparability
- Self-centring ability
- Non-structural Damage
- Durability
- Affordability

These objectives are considered in more detail below.

3.1 *Damage Mitigation Effectiveness*

Essentially, how effective is the LDD system at preventing damage to structural and non-structural components? Some such as base isolation are proven systems which effectively mitigate building damage. Other LDD systems may be less effective and may not prevent damage from occurring to secondary elements.

If LDD is to become established as a preferred alternative to conventional design methods, it is critical that it does not lose credibility in its infancy. This will happen if it fails to live up to its principle promise, of minimising damage.

Articulating frame systems such as PRESSS or PRESLAM are examples of systems that provide some of the benefits of LDD without mitigating one of the most significant short-comings of conventional design. Both forms of construction rely on gapping of the beams at both top and bottom. This must result in tearing of the floor diaphragms, which therefore lose the ability to distribute seismic actions effectively. The observation that this is no worse than is expected in conventional RC moment frames is not an adequate explanation or counter to this criticism, if they are to qualify as LDD systems.

3.2 *System Reparability*

Can the structural system be economically repaired following a major event i.e. does it have ‘plug and play’ capability? Some systems have energy dissipaters which can be unbolted and replaced following a large event, for example Buckling Restrained Braced Frames (BRBFs) and U-shaped Flexural Plate (UFP) systems Other systems that rely on mild steel reinforcing encapsulated inside reinforced concrete elements, to act as energy dissipaters, are more difficult to repair.

As a general rule we should be aiming to use systems that can be easily repaired i.e. avoid encapsulated systems; or replaced i.e. can have external dissipaters easily retrofitted if the original dissipation system is damaged beyond use or repair.

3.3 *Self-centring Ability*

This section deals with those systems that can provide some self-centring capacity to a building such that the building returns close to its original vertical alignment with minimal residual deformations following the DCLS earthquake. It is unclear at time of writing how much of a premium should be placed on this capability.

Some low damage design systems, such as PRESSS type hybrid walls and frames, have active self-centring by means of unbonded post-tensioned tendons (notwithstanding the issues with gapping of the frames). Base isolation systems are typically fully or nearly self-centring.

Residual building deformations can also be minimised by ensuring that the post-yield stiffness of the whole building (i.e. including both primary and secondary elements) is at least 5% – 10% of the initial elastic stiffness (Pettinga et. al., 2007). As suggested by Pettinga et. al. (2007) in some cases the required post yield stiffness could be provided by introducing a secondary, more flexible, seismic resisting system which is designed to remain elastic for the DCLS earthquake. An example of this would be a BRBF building with some of the secondary ‘gravity’ frames detailed to provide the necessary BRBF post yield stiffness.

Experience from earthquakes has typically shown that conventional buildings that have not experienced foundation failures have generally sustained residual building drifts that are close to the tolerances permitted in material standards for new construction i.e.:

- Concrete buildings: h/400 (0.25%) or 25 mm (NZS 3109, SNZ, 1997)
- Structural steel buildings up to 60 m: h/500 (0.2%) or 25 mm (NZS 3404.1, SNZ, 2009)

However it has also been noted that the Christchurch earthquakes have been relatively short duration events and more significant residual building drifts might be expected for longer earthquakes.

McCormick et. al. (2008) have suggested an allowable residual drift of 0.5%. This limit has been derived with consideration given to both human comfort, building functionality and building safety. Buildings with residual drifts exceeding this limit may not be acceptable to occupy following an earthquake. In many cases it may not be practicable to straighten a building following a major earthquake. Because of this when designing new buildings we should have a target residual drift of no more than 0.5% at the DCLS.

3.4 *Non-structural Damage*

Large inter-story drifts associated with flexible systems can result in significant damage to non-structural components such as partitions and cladding elements. Conversely, stiff buildings typically have higher floor accelerations which can lead to higher levels of contents damage.

Both inter-story drifts and floor accelerations can be reduced by adding damping to a structural system, in excess of that traditionally assumed for conventional seismic design, i.e. 5%. The form of the damping is also relevant, with viscous damping offering the benefit of peak energy dissipation at peak velocity, out of phase with the maximum displacement. This may result in a more efficient structure and foundation system.

3.5 *Durability*

The economic life of a building is generally assessed at 50 years under the Building Code, although there is no consequent driver to require removal of the building. However it should be assumed that any LDD system must achieve this life, either through durability or an effective maintenance regime. A difficulty with this can be anticipation of the possible exposure of elements over time.

For example, in the case of friction damping elements, such as steel plate systems, what allowance should be made for possible leaks in elements at building façade lines? As an alternative, should the system have a fail-safe mechanism such that if the plates were to lock, an acceptable capacity designed failure mechanism (strong column/weak beam) would ensue?

Other systems such as external dampers or base isolation require durability testing or protection. In the case of systems such as PRESSS frames or walls, access for re-stressing should be allowed, but an effective maintenance and monitoring process should be followed.

3.6 *Affordability*

All projects have budgets and some LDD systems are more expensive than others. It is important that both the costs and benefits are fully considered when LDD systems are selected. This should include consideration of ongoing life-cycle costs such as monitoring and maintenance if the systems require it, and any increased design and compliance costs.

Ultimately, affordability must be determined by the developers and users, to the extent that they are involved in the decision. However, the costs must be declared at the outset if they are to make informed decisions. This may require development of alternative conventional systems at least to concept phase, for comparative costing.

4 SEISMIC DESIGN METHODOLOGIES

Traditionally seismic design has been undertaken following a force based approach. More recently this has changed to a deformation focused, force based design whereby a greater emphasis is placed on checking expected plastic deformations of critical structural elements. This is the methodology currently encapsulated within the NZBC and supporting documents.

Displacement based seismic design is also becoming more widely accepted in the design community as the importance of deformation, rather than strength, in assessing the seismic performance of buildings has become recognised (Priestley et. al., 2007). However the application of displacement based design to 'real' buildings is not always straightforward.

For irregular buildings, designed using displacement based design, a non-linear time history analysis is recommended as part of the design validation. Furthermore displacement based design is not covered by NZS 1170.5 (SNZ, 2004) and as such is treated as an alternative solution under the NZ Building Code.

In practice, to simplify regulatory requirements, an interim pragmatic approach might be to undertake a simplified displacement based design to determine basic detailing requirements and establish adequate performance at the DCLS, while a conventional force based design is carried out to demonstrate compliance with Building Code requirements. However, this depends on the nature of the system being used, some of which may have unique design aspects requiring a specific methodology to be followed. It is likely in either case that Building Consent Authorities will (or should) require independent peer review, requiring agreement amongst designers on the verification methods to be followed.

Eventually, some form of design appropriate to such systems may be standardised, allowing them to be consented through a conventional processes. But it is worth noting that since the development of base isolation in the 70's, there is still no codified design process in New Zealand, probably mainly because of lack of demand. Such systems may be designed using international standards and guidelines, but the adoption of such guidelines for use in New Zealand must be carefully considered to ensure it is appropriately calibrated against the NZBC.

Future design methodologies are likely to follow performance based design principles. This is likely to include greater flexibility for designers to select differing levels of performance in conjunction with their clients. Although this is still in the early development stages, it is understood at the time of writing, that ATC-58 has just been published in 100% draft form. Adaptation of this document may be a way forward for the profession in New Zealand.

5 DEVELOPMENT AND TESTING

In recent years there has been a significant research effort put into the development of LDD systems. Some of this has already translated to industry, with several friction-damped, PRESSS and PRESLAM systems built or nearing completion. It is the authors' contention though, that there is still further development necessary to demonstrate that these systems are capable of delivering the levels of performance that are ascribed to them (but not that they should be considered not to meet Building Code life safety standards). An example of this is the consideration of diaphragm damage to multi-bay PRESSS systems.

The verification of some of the performance objectives noted above is not a trivial task. For example, it requires real-time testing in order to verify the performance of damping devices. Three dimensional specimens may need to be tested in order to investigate concurrency and the performance of diaphragms in gapping systems, pushing the limits of current testing facilities in New Zealand. However, there is a need to ensure that these questions can be answered.

Design methodologies must be developed which offer at least the same margins of safety as are inherently included in the current Building Code. This requires that sufficient test data is assembled as to provide statistically meaningful results, as well as the underlying theory.

6 SUMMARY

Low Damage Design offers significant promise for the future as a means of reducing the impact of future earthquakes. As has been demonstrated in the recent Canterbury earthquakes, even insured losses can have tremendous consequential collateral damage to society and the economy.

It is likely that New Zealand will reconsider its approach to seismic design, with the possibility that our current life safety design objectives will be expanded to consider more damage reduction measures. But even if that is not the case and our life safety approach is reaffirmed, increased awareness among building users, owners and developers is likely to result in increased interest in damage reduction measures. It is vital for the ongoing credibility of the industry that LDD systems that are implemented will actually deliver low damage.

The authors have proposed a set of objectives that should be considered in the development of LDD systems, noting that although there are systems being developed that claim to deliver low damage performance, they may not satisfy all of these objectives.

Looking forward there is a clear need for the development of a consistent set of standards which can be used by Structural Engineers and Territorial Authorities to design and review LDD buildings. This is important, both to reduce compliance costs, and to ensure that consistent and adequate levels of structural performance are provided across the various LDD systems available. Ideally the standards would be performance based rather than prescriptive.

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