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PERFORMANCE-BASED SEISMIC ASSESSMENT: SIMPLIFIED METHODS AND COLLAPSE INDICATORS

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Abstract

Performance-based seismic assessment has developed significantly in the past 20 years. Despite our best efforts to improve the quantitative tools to characterize seismic hazard, nonlinear responses, elemental behaviour, uncertainties, damages and losses – the reality is an accurate prediction of building performance in earthquakes is still very challenging for researchers and practitioners alike.

We argue the overemphasis on numerical procedure and the departure from understanding the seismic behaviour is detrimental to the performance-based assessment outcomes. In our practice, the focus of a performance-based seismic assessment is on ascertaining the likely building behaviour and the governing inelastic mechanism such that informed decision can be made of the implied seismic risk and required seismic strengthening. This is generally done via simplified analysis methods and assessment of collapse indicators called severe structural weaknesses.

This paper challenges some of the fundamental concepts in performance-based seismic assessments in the context of international and New Zealand research. It presents some thoughts on the research needs for performance-based seismic assessment to truly fulfill its potential.

Keywords: performance based seismic assessment, collapse indicator, non-ductile building, seismic vulnerability

1. Introduction

Performance-based seismic assessment has developed significantly in the past 20 years. Despite our best efforts to improve the quantitative tools to characterize seismic hazard, nonlinear responses, elemental behaviour, uncertainties, damages and losses – the reality is an accurate prediction of building performance in earthquakes is still very challenging for researchers and practitioners alike.

From the authors experience in New Zealand, we observed that the over overemphasis on analysis and numerical procedures leading to a departure from understanding the seismic behaviour is detrimental to the assessment outcomes. In our observation, the research in this area is also taking a highly numerical and analytical direction, resulting in a divergence in practice-research in New Zealand and abroad.

The late Prof Alin Cornell and late Prof Helmut Krawinkler have wisely stated on the ultimate goal of performance-based seismic engineering [1]:

> *"The final challenge for PEER researchers is not in predicting performance or estimating losses; it is in contributing effectively to the reduction of losses and the improvement of safety. We must never forget this. It is easy to get infatuated with numbers and analytical procedures, but neither one of them is useful unless it contributes to this final challenge."*

In our practice, the focus of a performance-based seismic assessment is on ascertaining the likely building behaviour and the governing inelastic mechanism such that informed decision can be made of the implied seismic risk and required seismic strengthening. This is generally done via simplified analysis methods and assessment of collapse indicators called severe structural weaknesses.

This paper explores the development of performance-based seismic assessments in the context of international and New Zealand research and challenges some of the fundamental concepts on how to undertake performance-based seismic assessments. We attempt to present some suggestions on the research needs for performance-based seismic assessment to truly fulfill its potential.

2. Performance-based Seismic Assessment

2.1 From Seismic Design to Seismic Assessment

Traditionally, seismic design codes provide prescriptive "deemed to comply" criteria that specify minimum levels of strength, stiffness and ductility, and outline the acceptable materials, detailing and configuration in order to achieve the minimum levels of safety and performance of buildings in earthquake. Our experience is the past earthquakes shape these prescriptive criteria.

The expected performance of a building in earthquake is not generally assessed in new building design but a minimum level of performance is implied by the rules that are prescribed. These rules include the use of capacity design and minimum requirements for element detailing e.g. minimum tie spacing in concrete columns. The minimum levels of performance required extend beyond the design levels of shaking even though they may not be explicitly accounted for.

Following our collective experience of the poor behaviour of buildings designed and constructed prior to the introduction of modern seismic codes in the 1970s, it has been identified that seismic assessment and retrofit of these older building stock is a significant challenge for the earthquake engineering fraternity.

The New Zealand Society for Earthquake Engineering (NZSEE) published a code of practice in 1972 [2] to classify high earthquake risk buildings, targeting specifically at unreinforced masonry buildings. This document was subsequently expanded and updated in 1985 [3] and 1995 [4]. The earlier version was largely a qualitative broad checklist type of assessment whilst the 1985 and 1995 guidelines attempt to provide quantitative assessment using simple force-based analysis and assessment procedures.

Internationally, the Applied Technology Council (ATC) has been publishing guidelines for the seismic evaluation of existing buildings since the 1980s, from the ATC-14 1987 [5] and ATC-22 1988 [6] documents, eventuating in the widely used FEMA 178 1992 [7] and FEMA 310 1998 [8]. These early guidelines rely significant on the observations and lessons from earthquakes due to limited analysis tools and experimental data.

2.2 Why performance-based seismic assessment?

Existing buildings are expected to be non-conforming to the prescriptive rules in the seismic codes and the earlier generation of seismic assessment guidelines. By applying the modern seismic standards to these buildings will only yield a large number of 'non-compliance' buildings without necessary an understanding of their actual seismic risk or performance. Similarly, code writers struggle in providing a prescriptive set of rules that could capture all the possible variation of non-compliant existing buildings.

One key principle is that the understanding of how the structural components behave and respond under earthquake shaking becomes more important than necessarily meeting specific code clauses. Similarly a thorough understanding of the underlying principles of various code clauses is required such that ensure that contravening a rule does not unduly jeopardise the minimum levels of resilience that are inherently provided for in a new building.

Performance-based seismic assessment offers a consistent framework for engineers to evaluate how a building may behave in an earthquake, in particular when a building that does not meet a number conventional prescriptive requirements and responds in a highly non-linear manner with potentially mixed-ductility response. The modern seismic guidelines (NZSEE, 2006 [9]; ASCE-31, 2003 [10]; ASCE-41, 2006 [11]) provide the tools to assess the likely behaviour of the structure and the performance consequences for a range of earthquake ground shaking.

This framework also provides the means for engineers to both communicate and relate the diverse structural performance outcomes to the diverse objectives of building owners / society. This is particularly important in communicating the seismic risk profile of an existing building – imagine explaining to other road users that a particular group of cars do not have compliant lights, brake or seat-belts, but still allowed to be 'road-legal'. In some document e.g. the SEAOC Vision 2000 document [12], an explicit relationship between performance objectives, earthquake hazards/frequency and measurable performance levels is outlined in a performance matrix as shown in Figure 1.

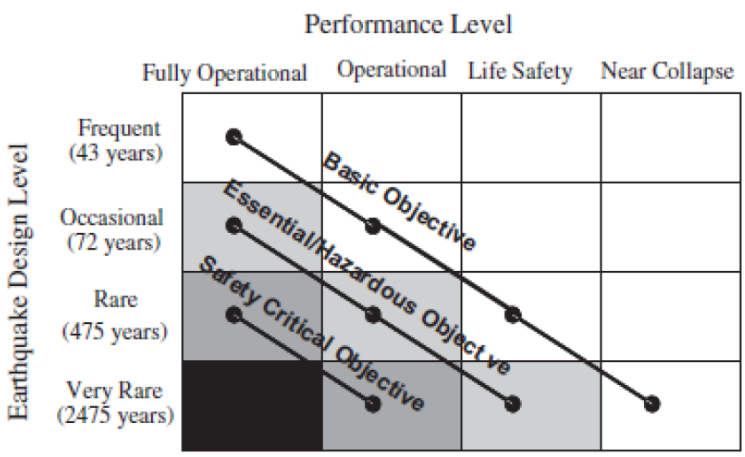

Fig. 1: SEAOC Vision 2000 performance matrix (1995) [12]

2.3 Performance-based seismic assessment guidelines

Recognising the needs as outlined in preceding section, a number of 'performance-based' seismic assessment guidelines were introduced in the late 1990s and early 2000s.

In FEMA-273 (1997) [13] pioneered performance-based seismic assessment of existing buildings using non-linear analysis procedures such as nonlinear static pushover and nonlinear dynamic analysis. FEMA-273 and its subsequent revisions (FEMA-356, 2000 [14] and ASCE-41, 2006 [11]) compiled and introduced definition of various performance levels and associated structural and non-structural 'performance', ways to use non-linear analysis techniques and definitions of "acceptance criteria" i.e. plastic capacity deformation and strength limits for various structural components.

A building's seismic performance (e.g. damage state of structural and non-structural components) is associated with various engineering demand parameters (inter-storey drift or floor acceleration), which are quantified using non-linear analyses and experimentally-calibrated local structural component capacities. Figure 2 illustrates both the discrete performance levels of these first-generation performance-based assessment methods; and the 'performance continuum' introduced in the second-generation guidelines [15].

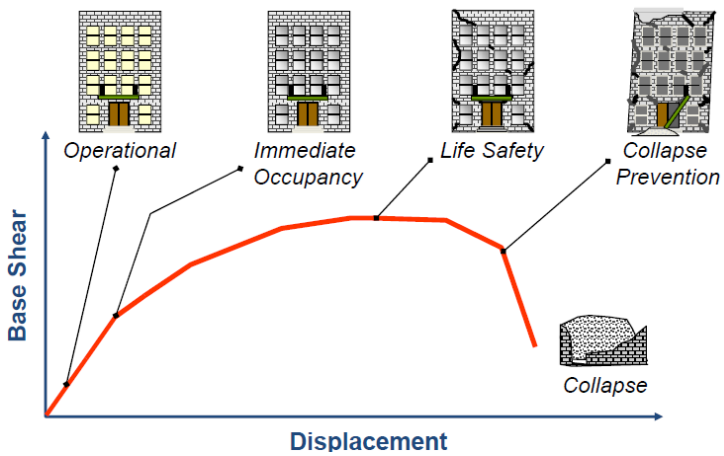

Fig. 2 – Relationship of building response/damage to member response (from [15])

Instead of a 'pass / fail' approach adopts in some guidelines, the New Zealand's performance-based seismic assessment requires the engineer to consider whether the failure of a particular structural component is likely to compromise the integrity of the gravity and lateral load resisting systems leading to structural behaviour exceeding the acceptable performance levels e.g. collapse or life safety risk?

In the 2006 revision of the NZSEE guidelines [9], non-linear displacement-based seismic assessment and the Simplified Lateral Mechanism Assessment (SLaMA) procedures as 'reverse capacity-design' approaches to forming an understanding of the seismic performance of buildings. A whole new set of rules, procedures and a mind-set change is needed as part of this framework in order to provide a consistent framework to assess buildings falling outside the prescriptive code requirements and that could potentially have a range of performance outcomes.

2.4 Next generation of Performance-based seismic assessment guidelines

Research of the next generation of performance based seismic assessment methodology suggests that the improved understanding of building behaviour would allow either a qualitative or quantitative estimate of seismic performance including risks of casualties, building damage and downtime, and resulting economic loss that may occur as a result of earthquakes (Deierlein et al, 2003 [16]; FEMA 445 2006 [17]; FEMA-P58 2012 [18]).

The proposed FEMA P-58 procedure is probabilistic with uncertainties explicitly considered, as it is recognized exact performance prediction is not possible with the many inherent uncertainties that exist. Seismic performance is expressed, in terms of the probable consequences i.e. human losses (deaths and serious injuries), direct economic losses (building repair or replacement costs), and indirect losses (repair time and unsafe placarding) resulting from building damage due to earthquake shaking.

Structural engineers and non-engineers alike may be excited by the prospect of being able to discuss seismic risk and its mitigation in terms risk, damage, reparability, cost – terminologies that may have more meanings than structural ductility, flexural hinging and base shear; but need to recognize that past deterministic thinking needs to change if the concepts are to be adequately conveyed.

3. Issues with Current Paradigm of Performance-based Assessment

While significant improvements in this field have been made in the past decade, there are several unhealthy trends that have crept into the research and practice of performance-based seismic assessment. In our view, these can divert practitioners away from forming a better understanding of the seismic behaviour of buildings and can inhibit the objective of understanding and reducing the seismic risk. A brief summary of these issues are provided below and an extended version is available in Kam and Jury, 2015 [19].

3.1 Reliability of our prediction tools

The premise of first-generation performance-based earthquake engineering in the North-American guidelines (FEMA-273, ASCE-41 and FEMA-P58) is that building seismic performance can be predicted and evaluated with quantifiable confidence. The ASCE-41 standard for example, makes reference to performance levels and structural component acceptance criteria related to loss of intermediate occupancy and building near-collapse. Recent research has also promulgated in several guidelines on collapse prediction and assessment (FEMA P-695; NIST, 2013 [20])

However, numerous evidence from analytical-experimental and post-earthquake reconnaissance studies (e.g. Maison et al, 2009 [21]; Kam et al, 2011 [22]) has shown that structures generally perform better than what the sophisticated non-linear assessment procedure may suggest. The fact is they need to otherwise the Building Code objective of an acceptable risk of collapse, considered holistically, will not be met.

This is also evident from a comparison of the "code-level" design spectra and the response spectrum of the recorded ground motions from the Christchurch February 2011 earthquake as shown in Figure 3. The recorded ground shaking demand greatly exceeded the "design level shaking in all CBD recording stations, but only a small number of buildings, mostly earthquake-prone unreinforced masonry construction (plus, of course, the CTV and PGC buildings), collapsed.

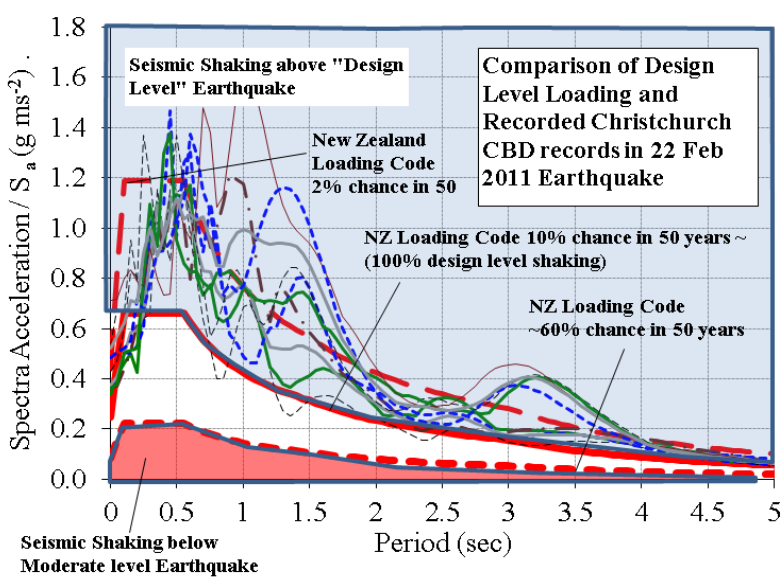

Figure 3: Comparison of "Design Level" and Moderate Level Shaking (33%) with the recorded horizontal acceleration response spectra in Christchurch CBD from the 22 February 2011 earthquake.

An assessment using force-based linear analysis would suggest a large number of the buildings in Christchurch should have collapsed under the recorded seismic loads, which were 2 to 6 times the calculated 'design' capacity. Explanations are offered regarding, for example the relatively short duration of shaking in this particular earthquake, but the fact remains that similar observations have been made after every earthquake affecting a region that has had some historical earthquake resistant design requirements.

The next generation performance assessment guidelines, FEMA P-58, recognises the inability to precisely assess the seismic performance of a building due to the cumulative results of uncertainties in the various components of the performance assessment (hazard, modelling, damage and losses). It advocates the use of performance functions that incorporate probability and uncertainties; but relies on the use of Monte Carlo simulation of specific analytical model with calibrated probability functions. However initial benchmarking studies of the FEMA P-58 suggest losses and damage using these techniques were over-predicted when compared to past earthquake data [15].

When the performance-based seismic assessment is extended for a large group of buildings to predict probabilistic hazard, damage, loss, fatality, and downtime etc., the validity of the predictive models is even more questionable due to the numerous uncertainties and sensitivity of the input parameters (Lin et al, 2012 [23]; Liel and Deierlein, 2012 [24]). This type of analysis may provide some insights into the general trend of the building population seismic performance to inform code development, but in our opinion, it is not necessarily very meaningful for assessment of individual buildings.

Bradley [25] rightly pointed out that the little to none emphasis in the research for performance-based earthquake engineering in terms of the "high level" uncertainties in the constitutive model and modelling methodology. The problem being a rigorous consideration of modelling uncertainties is simply impractical as it requires a good understanding of likely building inelastic behaviour prior to the refined and sophisticated analysis – and this is only practically achievable with simpler analysis tempered with engineering judgment; though it is acknowledged this may be a low variance high bias outcome noted by [25].

3.2 More sophisticated analyses will improve accuracy and reliability

While the response of buildings to earthquake is a complex, three-dimensional, non-linear and dynamic problem –a similarly complex non-linear dynamic analysis may not necessarily provide the silver bullet towards aiding our understanding of a building's seismic behaviour.

Advanced and sophisticated analyses such as non-linear time-history are useful in understanding the non-linear and dynamic behaviour of the building. However, it is important to recognise that any output of a non-linear time history analysis is only a snapshot representation of the building response to one particular earthquake record and then highly dependent on the ability to adequately model the non-linear element behaviour. The actual performance in an actual earthquake is contingent on a number of other variables that may or may not be modelled [20].

Post-earthquake forensic investigation of collapsed buildings, including those presented at the Canterbury Earthquake Royal Commission for the collapsed CTV and PGC buildings (CERC, 2012 [26]) have also shown further challenges in predicting the exact performance of a single building even with the very sophisticated non-linear time-history analysis techniques that are now available.

There is a tendency to think that more sophisticated analyses "should "always provide a more accurate answer, resulting in a more efficient and less conservative design" (Chambers and Kelly, 2004 [27]; Searer, 2008 [28]) and a better assessment for an existing building. However, this is not always the case as a more complex analysis needs more input parameters, each subject to judgement, probabilistic outcome and potential errors. There is a fine balance between accuracy, reliability (or precision), cost and complexity in structural analysis, as illustrated in Figure 4. It would be erroneous to conclude that the need for judgement decreases as the sophistication of the analysis increases. The more sophisticated the analysis, the greater the probability that something unintended has been modelled, and the greater the reliance on judgment that this will be recognised.

Contrary to complex analysis of individual buildings undertaken for research projects, there is often a higher cost and limited time for similar rigor for commercial projects. Thus, there is limited opportunity for sensitivity and parametric analyses, and limited opportunity for "testing" different input ground motions etc. This results in a limited number of results / runs for a particular building, which limits the usefulness of nonlinear time-history analysis to anything other than another tool in the engineer's toolkit to aid in the understanding of how a building might perform

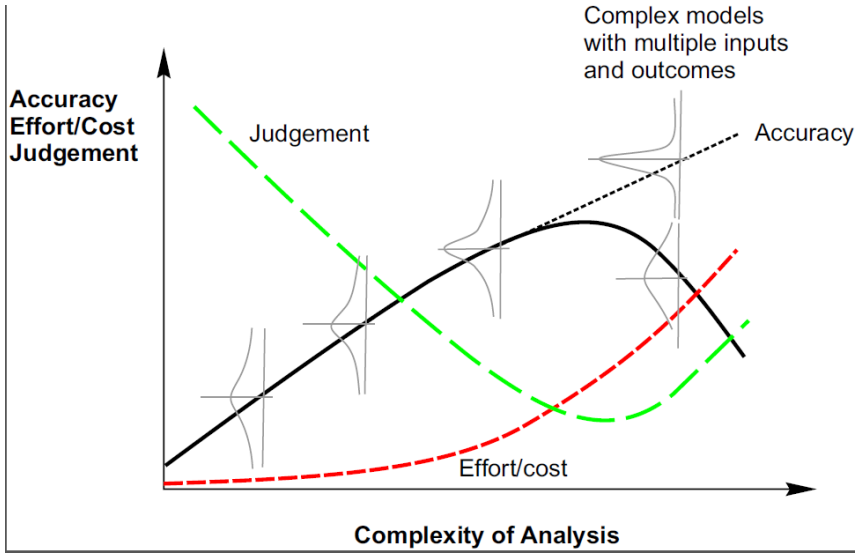

Figure 4: Trade-off between accuracy, engineering judgement, cost and complexity of structural analysis

A blind prediction contest sponsored by UCSD, the Portland Cement Association and the Network for Earthquake Engineering Simulation (NEESinc) in 2006 [29] illustrates the challenges and uncertainty in relation to prediction of structural behaviour, let alone performance (i.e. damage, losses etc.). It involved a large scale shaking table test of cantilevered reinforced concrete walls, and researchers and professionals were invited to predict the response of the test wall to the four input earthquake motions.

The 'best estimate' results itself showed a significant scatter despite the structure being 'capacity-designed' to ensure a specific inelastic flexural mechanism formed at the base of the walls. The overall system lateral capacity and stiffness was typically underestimated, as some unexpected interaction between the wall, slab and gravity columns were not adequately modelled [29-30].

A few years later, UCSD organised another blind prediction contest, and this time of a simpler reinforced concrete cantilevered column sub-structure (PEER, 2010 [31]). The organisers concluded, "*the results individually and collectively show a very high level of analytical capability, but also suggest we have a ways to go to be able to predict both the global and local responses of even simple structures*". Despite the use of the most sophisticated analysis available to practitioners and researchers, the experimental-analytical prediction of the maximum lateral displacement of a single-degree-of-freedom system still varies significantly.

In our view, while there is a place for sophisticated analysis such as non-linear time history in performance-based seismic assessment, this ought to be tempered with simpler analyses that provide some indication of the likely building behaviour and the governing inelastic mechanism. We believe that a black box non-linear time history analysis presents significant risks of complacency and a potential focus on localised element rather than a holistic view of building behaviour.

Any advanced analysis requires significant effort and engineering judgement to ensure the validity of the outputs. Fundamentally, such an analysis should be treated as another tool to approximate behaviour and probable outcome, rather than a predictive tool with an exact output. While the accuracy may have increased with the use of complex and sophisticated analysis, the uncertainties, precision and reliability remain a function

of the level of checking and rigor of the analysis (number of runs, sensitivity analysis and well-defined analysis parameters).

Some consultants in California e.g. [28] have observed that the use of these sophisticated analyses (e.g. 3D modal analysis or non-linear pushover) by engineers who are inexperienced in earthquake engineering may result both in a false sense of security and in buildings that will not behave as expected. This is consistent to our observation in New Zealand.

From our experience and observation, many buildings are still assessed using relatively simple force-based procedures that have little consideration of the nonlinear behaviour of local components and global building, displacement and ductility compatibility between dissimilar elements, and realistic estimate of displacement demands on the building.

3.4 Seismic assessment focusing on the lateral load resisting systems only

We have observed that a significant amount of seismic assessment effort is typically expended assessing the ability of the primary seismic lateral load resisting system to resist the require demands and often the other elements in the building receive, at best, only scant attention.

It is clear, however, that collapses of buildings occur due to failures in the gravity load resisting systems. These occur because the primary seismic system has provided inadequate protection to the gravity system, particularly when the lateral and vertical resisting systems are separate and the gravity systems are heavily loaded.

While properly detailed and configured primary lateral load resisting systems have rarely failed due to a lack of strength capacity there are numerous instances of gravity failures due to inadequate deformation capacity in the gravity system to sustain the applied deformations.

Often the deformations that have been applied have significantly exceeded those which would have been estimated during the design process due to unexpected behaviour of the lateral system once this goes inelastic, particularly when this also causes torsion behaviour due to unexpected eccentricities.

It is clear that assessment of seismic performance must include an adequate assessment of both the lateral and gravity systems, with perhaps greater attention placed on the gravity system and how well protected is the load path (e.g. diaphragm detachment and punching shear of flat slab-to-column connection). Higher levels of conservatism may be required in assessing gravity systems particularly when high levels of ductility are expected in the lateral system, irregularities are present and when the gravity system is heavily loaded.

4. Simplified Methods for Nonlinear Assessment

In our practice, the focus of a performance-based seismic assessment is on ascertaining the likely building behaviour and the governing inelastic mechanism such that informed decision can be made of the implied seismic risk and required seismic strengthening. This is generally done via simplified analysis methods and assessment of collapse indicators called severe structural weaknesses.

4.1 Performance-based seismic assessment with the NZSEE Guidelines

The current NZSEE guidelines [9, 32] provide several concessions to existing buildings which in effect allows a form of performance-based seismic assessment. It requires an understanding of the eventual non-linear behaviour of the building as a whole (not just a predetermined earthquake load level). These include:

- use of probable capacity of the structure – including probable material strengths (and therefore strength reduction factors approaching 1.0) and probable mechanisms (comparison of means expected capacities)
- mobilization of available inelastic mechanisms - i.e. the seismic probable capacity is not governed by first elements exceeding its strength/ductility capacity if it doesn't lead to a mechanism. Available inelastic mechanisms can be utilised as long as the gravity-load carrying capacity and global stability are not compromised, with appropriate levels of reliability e.g. in [33].

- Remove the need to check all the code clauses - and focuses on understanding/assessing the behaviour and life-safety performance of building under ULS levels of earthquake load. The expectation, however, is that the expected effect on the resilience of the building of not complying with a particular requirement is understood and considered to be acceptable in the context of the performance expected.
- Focus on life safety risk - and not necessarily damage/serviceability issues - as such some elements are allowed to 'fail' prematurely without affecting the seismic assessment outcome. What does constitute and not constitute a significant life safety hazard will rely to some extent of the judgment of the assessor.

Assessing the seismic capacity of an existing building is fundamentally different from designing a new building for seismic actions. Seismic assessment requires a clear understanding and reliable evaluation of the existing load paths, the probable inelastic deformation mechanisms, the probable "collapse mechanism", and the available ductility/displacement capacity of the structure.

This relies on an understanding of the hierarchy of strength and sequence of failure of a structure in an earthquake, by undertaking simplified hand calculation such as the Simplified Lateral Mechanism Analysis (SLaMA) outlined in the NZSEE guidelines [9, 32] – see next subsection.

Fundamentally, we suggest that there is a need to re-set the thinking on the fundamental idea that 'earthquake does not read the loading standards' and that our quantitative analysis, however sophisticated, is at the end of the day, a proxy measure of expected seismic behaviour.

4.2 SLaMA - Reverse Capacity Design

A SLaMA is a simplified technique for determining the probable inelastic deformation mechanisms and their lateral strength and displacement capacity by examining load paths, the hierarchy of strength along critical load paths, the available ductility/displacement capacity of the identified mechanisms and the manner in which various mechanisms might work together. The outcome from the SLaMA is an understanding of the nonlinear mechanism and an estimate of the global probable capacity of the primary lateral structure of the building

When undertaking a SLaMA, the focus is on assessing the capacity of the mechanism at the point a significant life safety hazard is created. Assessing the capacity at the point when the first member or element fails, provided that failure does not results life-safety issues, may result in overly-conservative building capacity estimates and corresponding strengthening regimes.

A simple example is shown in Figure 5. The first step for SLaMA is to assess the load path and confirm that the building has a cantilevered structure (wall or single-storey column) system. Next is to calculate the relevant probable strength and deformation capacities for cantilevered elements including its flexural and shear capacities, the foundation flexural and axial capacities and the corresponding deformation levels.

Then, by checking the hierarchy of strength of the interconnected components, the potential inelastic mechanism for element can be determined. In Figure 5 for instance, the horizontal shear force required to yield the flexural reinforcing of the wall/column is significantly less than the nominal shear capacity – therefore the flexural mechanism will dominate the inelastic response. Next, the global behaviour is assessed by extending the understanding at local elemental level s to overall subsystem (bracing line) and global system. The probable base shear can be calculated by summing up the probable capacity of individual mechanisms and subsystems, whilst the global displacement capacity will be limited by the mechanism with the lowest displacement capacity.

For simple element system in Figure 5, then the calculated displacement capacity of the cantilevered wall / column is the displacement capacity of the system.

SLaMA can be used in conjunction with more sophisticated analysis techniques. In general, the complexity and extent of the analysis should reflect the complexity of the building. Start with simpler analyses, progressing to greater sophistication only as necessary.

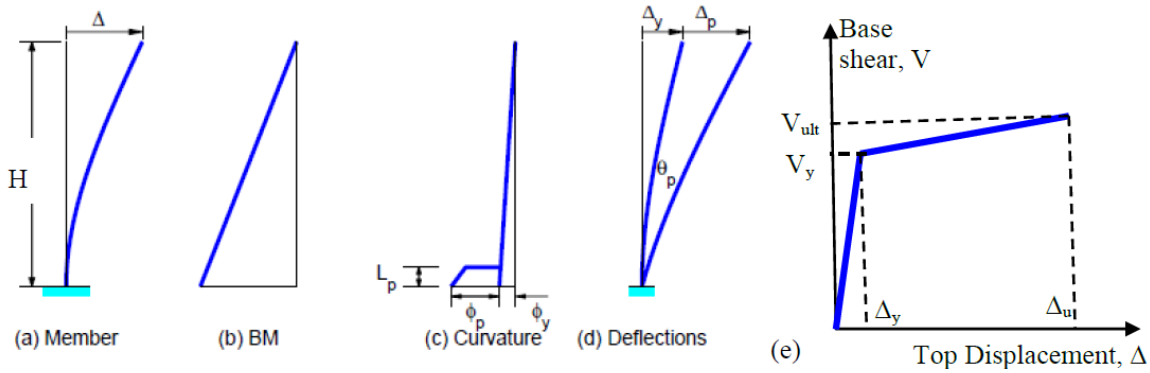

Figure 5: Illustrative example of method to derive the probable strength and deformation capacities for flexural cantilevered member

For more complex structures, SLaMa relies on an understanding of expected deformed shape profiles to correlate the relationship between the local mechanism and the global behaviour. The method will depend on the structural configuration and the identified local mechanism. For example, for moment-resisting frames system - the Sway Index [32] can be used to determine the likely hierarchy of plastic hinge formation. The inelastic deformed shape profile will depend on the hierarchy of plastic hinge formation as shown in Figure 6.

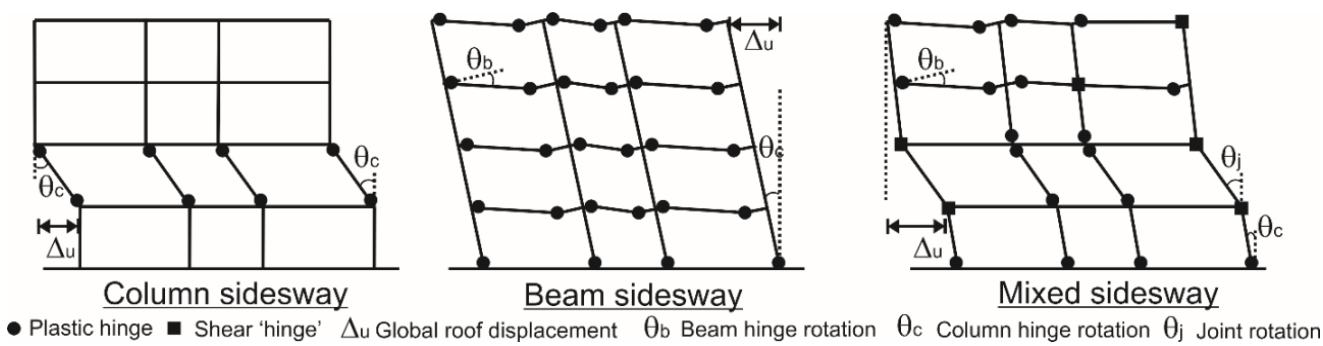

Figure 6: Possible mechanisms of post-elastic deformation of moment resisting frames (from [32]).

4.3 Considerations for analysis techniques for assessment

Structural analyses will be required to establish the relationship between the element/component actions and the global capacity of the building. In general, the complexity and extent of the analysis should reflect the complexity of the building. Start with simpler analyses, progressing to greater sophistication only as necessary.

Some considerations for the analysis method for seismic assessment:

- For relatively simple structures that conform with certain established criteria where complex analysis may not be warranted, the calculated demand and capacity of the building may be modified with appropriate factors based on the identified governing inelastic mechanism e.g. the use of different allowable strength reduction or spectra reduction factors (μ/ Sp and φ-phi).

- For complex / more significant structures (e.g. with regard to occupancy, consequence of failure), and/ or where greater levels of reliability of assessment are sought, the assessor is expected to qualitatively and quantitatively predict the seismic behaviour of the building across a range of seismic shaking with consideration of the inelastic behaviour and eventual governing mode(s) of failure, using appropriate methods.

Irrespective of analysis methods, consideration of non-linear behaviour is fundamental for a performance-based assessment and the assessor can mobilise all available inelastic mechanisms - i.e. the seismic performance result is not necessarily governed by the first elements exceeding its strength/strain/ULS capacity provided that local behaviour doesn't lead to loss of gravity-load path and/ or lateral instability that could reasonably lead to a life safety issue.

For simplified nonlinear hand analysis techniques such as SLaMA and displacement-based assessment [32, 34], 3D effects such as amplification to the displacement/ductility demand due to inelastic torsional effects should be accounted by additional analysis techniques.

For some complex circumstances where commonly used analysis is unable to predict the behaviour well, a set of prescriptive clauses or assessment of specific critical structural weaknesses would be put in place to penalise the building design (e.g. torsional instability of irregular and ductile systems).

5. Consideration of Resilience - Collapse Indicators

5.1 Assessing resilience and collapse implicitly

There is a misconception that building assessed to a specific level of shaking (say 500 years return period motion) will only deliver acceptable performance at that level of shaking i.e. it is acceptable for the building to perform poorly and collapse could well occur if the shaking level is higher. The introduction of collapse prevention check at a higher level of shaking (say 2500 years return period) is necessary to confirm an adequate level of resilience. This concept of two-level check is somewhat inherent in the FEMA-273 / ASCE-41 guidelines.

Given the uncertainties in predicting when a building will collapse, the authors do not consider it practical or necessary to design new buildings to a collapse limit state.

A review [22] of the performance of reinforced concrete structures in Christchurch concluded that there is a need for a departure from a force-based emphasis in the seismic design practice to a more displacement-based and ductile inelastic mechanism-focused approach (irrespective of the prescribed loading).

We argue that buildings are not expected solely to survive 100% of the design demand shaking, and then to suddenly fail and collapse in 110%, 150% or even 180% of this shaking. Figure 3 reinforces that this is largely achieved in practice, at least for buildings in Christchurch. While there are instances where the levels of inherent resilience assumed to be available by AS/NZS1170.5:2014 [35] are not always achieved, these have generally been due to a mode of behaviour that has not been considered in design rather than simply running out of code defined capacity.

5.2 Treatment of seismic resilience and performance beyond the “design level shaking”

One of the challenges of performance-based seismic assessment is to be able to assess the performance of buildings over a full range of earthquake shaking, not just the arbitrary design level shaking, but in a manner that does not overly complicate the process or not be cognisant of the considerable uncertainties involved in predicting seismic performance.

ASCE-41 and its earlier versions have elected for a two-limit states check – at “life safety” and “collapse prevention” levels, as a means to assess the seismic resilience beyond the traditional “life-safety” limit state. We believe that the collapse prevention approach of ASCE-41 has several disadvantages:

- It assumes that the point of collapse can be reliably predicted – which observations suggest it can’t
- It has the, perhaps unintended, consequence of suggesting to the assessor that there is an upper limit to the level of shaking that a building can be subjected to – which clearly is incorrect. This may lead to a building satisfying the ASCE-4 collapse prevention criteria but still being very poorly configured for larger shaking.
- Because of the difficulties with analysing close to a collapse point the criteria set for this performance state are often simply factored up criteria for lower performance states – this negates the benefit of the additional effort involved in more sophisticated analysis.

5.3 Severe Structural Weaknesses – Collapse Indicators

The NZSEE 2016 Guidelines recognise that there are potentially some aspects of a building's seismic behaviour which may not be adequately analysed within the general assessment procedure, but are likely to have a step-change response of the building resulting in sudden (brittle) and/or progressive, but complete (i.e. pancaking) collapse of the building's gravity load support system in shaking greater than that represented by the design level earthquake.

To account for the above, the 2016 NZSEE Guidelines introduced a category of collapse indicator called "severe structural weaknesses" (SSW) in which the assessed capacity of the SSW is applied a factor of 0.5 in order to provide a margin of at least two against catastrophic failure at the design level shaking used in the assessment.

In determining the SSW, three criteria have to be satisfied:

- **It has a demonstrated lack of structural resilience** so that there is very little margin between the point on onset nonlinear behavior (e.g. cracking of a structure or large deformation of soil) and step change brittle behaviour of the building resulting in catastrophic collapse.
- **It has a severe consequence** if catastrophic collapse occurs. A severe consequence is intended to be associated with building topologies with more than two storeys and where multiple fatalities (>20) would be possible if one or more storeys were to suffer full collapse
- **There are recognized limitations in the analysis and assessment of the behaviour** so the reliability of the assessment of probable capacity of the expected mechanism is low. This could simply because there is currently considered to be insufficient research data or experience to confirm the behaviour to a generally accepted levels of reliability.

Six SSWs are defined in the first draft of the NZSEE 2016 Guidelines, as listed below and illustrated in Figure 7:

- Highly axially-loaded and lightly reinforced concrete column and/or beam-column joints susceptible to axial-shear failure.
- Single core shear walls with no redundancy and a brittle shear-failure mechanism
- Flat-slab system susceptible to punching shear failure at low level lateral drift
- Diaphragm with no positive connection (no starters or collectors element) between diaphragms and primary lateral load resisting structure
- Complex slope failure resulting in significant uncontrolled slope movement and loss of building platform
- Liquefiable ground supporting poorly tied together multi-storey URM buildings

These collapse indicators are important to ensure the seismic performance for an existing building have a good degree of protection and structural resilience against catastrophic collapse in a range of earthquake shaking, even those levels used for conventional new building design (e.g. 1 in 500 years earthquake for normal building).

Figure 7: Severe Structural Weaknesses (SSWs): a) non-ductile reinforced concrete column with axial-shear failure; b) Pyne Gould Corp building collapse in Christchurch – non-ductile single core wall without redundancy c) flat-slab punching shear failure d) diaphragm detachment failure e) loss of building platform due to mass slope failure f) splitting and near-collapse unreinforced masonry church due to liquefaction and lateral spreading.

6. Future Research

These world conferences in earthquake engineering often provide a snapshot of where research and practice of earthquake engineering has progressed to. It is authors' hope that some considerable research effort is allocated to progress performance-based seismic risk assessment:

- Research to develop simple quantitative tools to reliably identify "killer" non-ductile (masonry, concrete and steel) buildings with very little resilience. Further development of the tools to assess the severe structural weaknesses outlined in Section 5 would be warranted.
- Better analytical tools to assess the "collapse capacities" of axially critical mechanism such as diaphragm unzipping, column axial-shear failure, beam-column joint axial failure, flat-slab punching shear failure e.g. research [36] on non-ductile columns effects on building collapse.
- Improved tools to relate component-level behaviour to global response such that engineers are assessing global building responses rather than individual components compliance. The SLaMA method outlined in Section 4.2 and the NZSEE Seismic Assessment Guidelines (2016) are some options for further research.
- Simple and consistent approach to account for the influence of geotechnical behaviour to overall building performance, without necessarily resolving to extensive site specific testing and complex soil-structure interaction modelling. [11, 32, 37]

7. Conclusions

The development of state-of-the-art structural analysis techniques and improved understanding of nonlinear behaviour of structure in earthquakes has enabled the development of performance-based seismic assessment of

existing buildings. More than fourteen years have gone by since the late Prof Cornell and Prof Krawinkler wrote their influential letter on their vision for the performance-based earthquake engineering (and by extension seismic assessment).

While significant progress has been made in the past decade of research, the late Prof Krawinkler reflected on the future challenges of performance-based earthquake engineering in 2011 [38] when a number of similar issues were discussed:

- Our understanding of ground hazard and seismicity is always stretched and improved by major events such as the Canterbury earthquake sequences. Research will be on-going to address near-fault effects and other site-specific effects.
- The ability of structural analysis model to determine damage parameters such as local deformation, large deformation effects, brittle failure mechanism, residual drift, and floor acceleration is still limited.
- The library of damage data and fragility function is incomplete and limited even after significant testing in the 10 years of the ATC-58 programme.
- Limited ability to precisely translate structural behaviour to 'performance metric' such as financial losses, downtime and fatalities/injuries. Available module is often based on limited data and judgement.
- Lack of consideration of the seismic resilience and vulnerability of the wider community and city; and post-disaster functionality, repair and recovery.

We have observed a gradual drift in the research and code-development fraternity in both North American and Europe towards a more analytically rigorous approach towards performance-based seismic assessment.

On the other hand, we have promoted in this paper what we considered a more pragmatic first-principle approach in which is possibly 'less sophisticated in practice' and relies on a good fundamental understanding of seismic behaviour of structures.

The future of performance-based seismic assessment should be pragmatic and underlined with good fundamental understanding of nonlinear behaviour without necessarily focusing on a computational intensive approach while resulting in less time and opportunity for engineers to spend thinking about a building's likely structural behaviour or investigating the actual structure on site.

Engineers need to be cognizant that seismic assessment of buildings is only one aspect of seismic risk mitigation for any particular asset owner and only one particular type of natural hazard risks. The challenges are for earthquake engineers to think more broadly of seismic risk and resilience, to beyond individual building and individual clients – but on a broader community, regional and national level.

8. Acknowledgment

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