

Performance versus Compliance of Buildings in the Seismic Context

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2017 NZSEE
Conference

ABSTRACT: With the development in recent years of performance-based seismic design and assessment methods, are we losing sight of the distinction between minimum code-defined compliance measures, and the expected performance these compliance measures are intended to achieve? The distinction between performance and compliance is becoming increasingly relevant as we move away from conventional structural systems and towards more innovative and alternative solutions. What is the relationship between design that is compliant with design standards, and performance? What is tolerable or acceptable performance? How do we ensure that a new system has adequate resilience, such that it still has an acceptable probability of damage or collapse considering all levels of earthquake shaking?

In this paper, the authors discuss the difference between performance and compliance as they see it, and through application of a simplistic probabilistic investigation aim to dispel some common misconceptions they have encountered with the introduction of so-called “performance-based” seismic design and assessment methods.

1 INTRODUCTION

In recent years, much effort has been expended internationally in the development of performance-based seismic design codes. These codes have requirements, often qualitative, of the way the building should perform. Given that **performance** is what physically happens when the building is put through its paces, how can its design be assessed for **compliance** before it is built, using requirements based around its predicted performance?

Such codes have an obvious advantage in that they have few restrictions on the design approach, structural system, configuration or materials that can be used, as was present in the more prescriptive codes of yesteryear.

But with performance requirements in general, how can a completely new or alternative solution demonstrate that it will meet the required performance? How do we ensure that these innovative solutions are achieving the required minimum level of life-safety risk?

In this paper, the authors illustrate the distinction between performance and compliance of a building to withstand earthquakes, and use the results of a simplistic probabilistic investigation to highlight the importance of ensuring resilience in a building’s seismic system.

2 PERFORMANCE VERSUS COMPLIANCE

In order to appreciate the distinction between performance and compliance, the authors have used the following definitions of performance and compliance as applied to buildings designed to resist seismic loading.

Performance is the physical way or manner in which the building performs when it experiences earthquake shaking. Its performance is often described by the extent of damage it has suffered, and the effects of this damage in terms of functionality and extent of casualties. Performance is a spectrum, ranging from no damage or effects, to total collapse. It can be defined, but only in a probabilistic sense (traditionally using fragility relationships). Actual performance can only be assessed following an event,

and performance in one event does not guarantee a particular performance in another event. Performance in a particular earthquake cannot be predicted without also recognising the uncertainties involved in doing so. The level of performance expected will/should vary depending on the level of shaking the building has been exposed to but is not absolute.

Compliance is the process that confirms whether or not the building design meets the required standard. For the purposes of this paper, the authors are restricting this term to describe the normal design/assessment process where a design is “checked” against various defined measures to confirm that the capacity of the building meets minimum requirements. Compliance is typically checked at defined levels of shaking (e.g. 1/500 years for the ultimate limit state) that are significantly simplified from reality – as this is considered to be the only practical approach - with the expectation that an acceptable minimum level of actual performance will actually be achieved overall.

In the following section, performance-based building codes are explained, including their advantages and the challenges associated with assessing compliance with them.

2.1 Performance-based building codes

Up until the late 20th century, most design codes and standards around the world were of a prescriptive nature. In essence, these codes mandated specific construction practises that were permitted to be employed. While having obvious benefits including being easy to interpret, these codes were considered to stifle creativity, giving little chance for innovative techniques to emerge.

With the release of the Building Act of 1991 and Building Regulations of 1992 (the New Zealand Building Code), New Zealand was one of the first countries in the world to introduce a “performance-based” building code.

A performance-based code can be described as one that requires a building to meet certain performance requirements or criteria, but with very few restrictions on the means to achieve it. A performance-based code will usually contain qualitative objectives, functional requirements and performance criteria. An example from the current New Zealand Building Code Clause B1 Structure is the first performance criterion:

Buildings, building elements and sitework shall have a low probability of rupturing, becoming unstable, losing equilibrium, or collapsing during construction or alteration and throughout their lives.

This criterion sets out the way the building is expected to perform, with no mention of any specific way of achieving it in design – it only contains the qualitative requirement of “low probability of ... collapsing”. In essence, it is requiring a certain performance when, at the design stage, no performance checks have occurred.

Clearly, this type of code provides a great opportunity for innovation by allowing new materials, structural systems and techniques to be employed. However, for seismic design, there are challenges in being able to set the performance objectives given the large range in demands a building may be subjected to and the difficulties in precisely determining how the building resists these demands (its capacity). The infrequent nature of large earthquakes also limits the opportunity to “test” a building in a real situation. These issues present obvious challenges when attempting to both demonstrate, and verify, that innovative or alternative solutions will meet the performance objectives.

In recent years, there has been the emergence of performance-based design approaches where an attempt has been made to define the performance objectives in more precise terms, and then to test these using nonlinear time-history techniques. These approaches tend to revert to a compliance exercise as it is impossible to test all demands, and very difficult to represent building capacity in a probabilistic fashion (Monte Carlo simulations are sometimes used to address this), but with the disadvantage that often a minimum level of performance has been implied/predicted for a given level of earthquake shaking.

2.2 Compliance with performance-based building codes

The New Zealand building control system contains two main ways to achieve compliance; the deemed-to-comply route utilising current verification methods and acceptable solutions, and the alternative solution route (Figure 1).

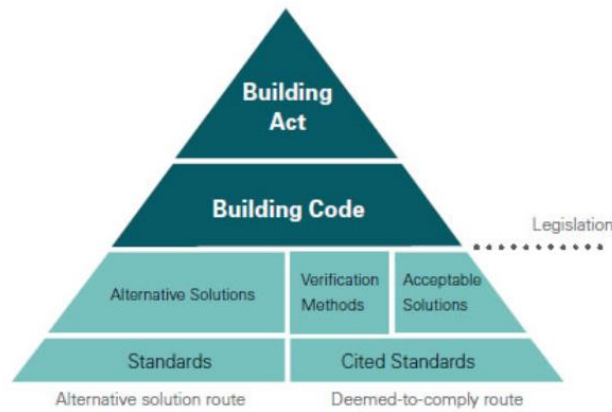


Figure 1. New Zealand building control system

Traditionally, most structural engineers have followed the deemed-to-comply route, whereby verification methods and associated cited standards (e.g. the suite of loadings standards NZS 1170, concrete structures standard NZS 3101 etc) contained design processes and methods that provide quantitative measures of compliance. A building design's compliance with a current and applicable verification method is deemed to comply with the qualitative performance requirements of the Building Code. This approach is able to be taken if the building design (including design approach, structural system, configuration and materials) falls within the scope of the cited standards.

Focusing on life-safety, there is no need to calculate the probability of collapse, as it is inherent within the verification methods and acceptable solutions that an acceptably low probability of collapse is achieved. In practise, this has been a gradual process of learning, including from performance in past earthquakes, new knowledge through research, and society's feedback that the actual earthquake performance of buildings has been acceptable or in some cases unacceptable.

Limit state design, specifically the ultimate limit state, is the process used in the verification methods and acceptable solutions to verify an acceptable life-safety risk is achieved. The ultimate limit state involves meeting certain easily calculable configurational requirements and checking that demands on elements of the structural systems are below a critical limit (e.g. column and beam plastic curvature limits) for a defined level of shaking input. These plastic curvature limits, combined with detailing requirements, capacity design principles and other criteria such as p-delta stability and global response limits (such as storey drift limits) leads to a resilience in traditional structural systems that means collapse of the structure is still acceptably unlikely at earthquake shaking much greater than (perhaps many times in the view of the authors) the level corresponding to the ultimate limit state. This is consistent with the expectation stated in the Commentary to NZS 1170.5 that there "will be a very low risk at the ULS of structural collapse".

Existing verification methods and acceptable solutions cover traditional structural systems (eg moment resisting reinforced concrete and steel frames, steel eccentrically and concentrically braced frames). For new systems that fall outside the scope of existing verification methods and acceptable solutions, compliance must be demonstrated as an alternative solution. If the design deviates enough from the existing compliance methods, it is difficult to verify that the system meets the current qualitative New Zealand Building Code performance requirements as they provide little clarity around what is intended by the term "low probability".

The authors are concerned that without more guidance around the intended qualitative ballpark of the acceptably low probability of collapse (for both designers and those responsible for verifying the compliance of a design), innovative solutions may be produced that have a probability of collapse many times greater than traditional systems. This paper aims to shed some light on the likely magnitude of the acceptable probability of collapse, and investigates how it is affected by different building typologies through a simple statistical investigation.

3 ANNUAL PROBABILITY OF COLLAPSE

Due to the probabilistic nature of both the demand on, and capacity of, a structural system designed to withstand earthquake loads, the prediction of the point of collapse of this system is also probabilistic.

This being the case, the acceptability (compliance) of the design of a structural system to resist earthquake loads effectively requires a limit on its probability of collapse, or maximum acceptable probability of collapse. This probability is also known as the structural reliability target (ISO 2394).

Furthermore, due to the probability of intensity of earthquake shaking being expressed in terms of a return period, the probability of collapse must have reference to a time period.

Useful time periods might be the design life of the building (usually 50 years), or per year. To facilitate comparison with other risk studies, probabilities are usually expressed on an annual basis. Annual probabilities will be used in this paper. However, annual probabilities can easily be converted to another time period using the following:

$$P_n = 1 - (1 - P_1)^n \quad (1)$$

where:

P_n is the probability in n years

P_1 is the annual probability.

There is at least some argument that would suggest that the period should not exceed one year when life-safety aspects are being considered.

The following section provides an overview of the use of structural reliability in building codes and standards, and an investigation by the authors involving the calculation and comparison of annual probabilities of collapse in order to assess relative risk of different building types.

3.1 Structural reliability and building codes

As mentioned above, The New Zealand Building Code Clause B1 Structure contains only a qualitative criterion for the acceptable maximum probability of collapse of a structure in order for its design to be deemed compliant. No guidance or commentary at Building Code level currently exists on the quantitative value intended by the term “low probability”.

However, the New Zealand loadings and materials standards and other building codes from around the world do provide targeted values, a study of which could suggest a likely range.

For example, both the Building Code of Australia (NCC 2015) and the American Society of Civil Engineers (ASCE 7-10) use structural reliability, or beta (b), factors to provide a limit on the probability of failure as calculated at the design stage.

Table BV1.1 in NCC 2015 provides a limiting annual structural reliability index, b , of 3.4 for earthquake actions for an Importance Level 2 building. A b -value of 3.4 corresponds to a maximum annual probability of 4×10^{-4} . Converting the 50-year beta factor provided in clause c2.3.2.2 of NZS 3101:2006, or c3.1(c) of NZS 3404:1997, for earthquake forces (b of 1.5 to 2.0) also returns a similar value for the annual probability.

The Commentary to the New Zealand earthquake loadings standard (NZS 1170.5 Supplement 1:2004) also refers to annual probability of collapse:

Internationally, an accepted basis for building code requirements is a target annual fatality risk in the order of 10^{-6} (ISO 2394:1998). In design terms, it is generally accepted that fatality risk will only be present if a building fails, i.e. collapses. The maximum allowable probability of collapse of the structure is then dependent on the probability of a person being killed, given that the building has collapsed. This conditional probability will be dependent on the structural type and other factors and is likely to be in the range 10^{-1} to 10^{-2} (indicative probabilities have been proposed as part of the FEMA 2001 project and are reported in McGuire 2004).

Accordingly, maximum acceptable annual probabilities of collapse might therefore be in the range 1×10^{-4} to 1×10^{-5} . This range is consistent with the value obtained above by converting the beta factors from BCA, NZS 3101 and NZS 3404. It should be noted that these values are applicable to normal structures, and structures of higher importance are required to have greater reliability, and therefore lower probability of collapse.

3.2 Methodology

The authors have conducted a simple investigation into annual probabilities of collapse of structures, and the relative risk posed by different theoretical building types. The objective was to observe trends rather than confirm actual probabilities.

Much literature (e.g. McGuire 2004, Hadjian 2004, Zareian and Krawinkler 2007) has been written on the calculation of probabilities of collapse including the use of fragility curves, so only a brief description of our methodology will be provided.

In general terms, ‘collapse’ for a given building occurs when the demand from an earthquake, E , is greater than the collapse capacity C (Figure 2). Collapse itself can mean many things, but in this context is used to describe partial or global collapse of a building for which a life-safety risk is presented. Assuming that E and C are continuous statistically independent random variables, the calculation of annual probability of collapse involves the integration of the sum of the probability density functions of the earthquake demand, E , and the collapse capacity for a given structure C , for all levels of earthquake shaking.

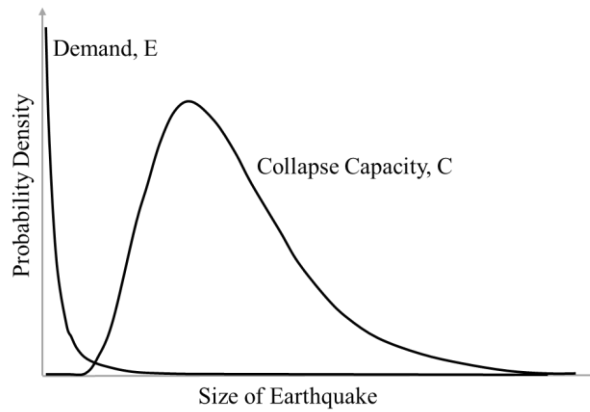


Figure 2. Earthquake demand versus collapse capacity as probability distributions

Expressed in numeric integration form:

$$P(C) = \sum_e P(E = e) * P(C|E = e) \quad (2)$$

where:

$P(C)$ is the probability of collapse over a given time period (annually in this case)

$P(E=e)$ is the annual probability of an earthquake of intensity e

$P(C|E=e)$ is the conditional probability of collapse given earthquake of intensity e

In words, the probability of collapse is equal to the sum, for all intensities (or return periods) of earthquake shaking, of the product of the probability of an earthquake of intensity e and the conditional probability that the structure collapses given an earthquake of intensity e .

3.2.1 Earthquake Demand

The probability density function for earthquake demand was taken from the return periods and associated design factors from NZS 1170.5, extrapolated for return periods greater than 10,000 years (Figure 3).

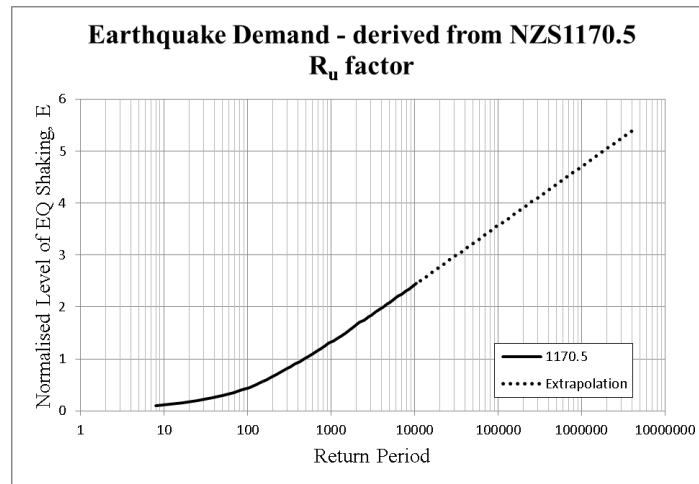


Figure 3. Earthquake demand

The intensity of earthquake shaking being considered has been normalised by 1 in 500 year return period shaking.

It should be noted that this approach to quantifying earthquake demand in probabilistic terms is highly simplified and is only for the purposes of providing a dependent variable in this investigation.

3.2.2 Collapse Capacity

Collapse capacity probability distributions are often represented using fragility curves, which are effectively cumulative distribution functions that portray the probability of collapse given the intensity of shaking of an earthquake.

There are multiple ways to produce collapse fragility curves, all fraught with uncertainty. These can include empirical studies, complex nonlinear time history analyses or curves generated using expert elicitation.

For our investigation, we used a fragility curve developed during an exercise carried out by the Ministry of Business, Innovation and Employment (MBIE) Engineering Advisory Group as a starting point. This curve was generated by expert elicitation, whereby experts were asked to provide the tolerable probability that a particular building typology would reach certain damage states in shaking with certain return periods. Of interest to us was the tolerable probabilities of collapse for what will be referred to as the 'baseline building' for our investigation – a new, Importance Level 2 (IL2), ductile, reinforced concrete frame building in Wellington. We took the tolerable probabilities as representative of the expected probabilities for the purposes of this exercise but recognise that differences may exist between these two concepts.

The experts were asked to provide probabilities for return periods up to 10,000 years. To assist with numerical integration computation, we fit a lognormal distribution through the data points and extrapolated to include higher levels of earthquake shaking based on our judgment of the collapse probability at higher levels of earthquake shaking. Figure 4 presents the data points and the lognormal fragility curve applied by the authors.

It should be noted that the total annual collapse probability calculated using this method is the summation of the product of two probabilities; one representing the probability of collapse given an earthquake of a certain intensity of shaking, and one representing the probability of an earthquake of that intensity of shaking. This is a continuum, and at either end one of the probabilities in the product will be very small, giving an insignificant product. For example, at very small levels of earthquake shaking, there is a high annual probability of that level of earthquake shaking, but a very low conditional probability of collapse of the building given that level of earthquake shaking, and vice versa at very high levels of earthquake shaking. The significance of this is that it is not critical to know the exact shape of the fragility curve at high levels of earthquake shaking, nor the exact shape of the hazard curve at low levels of earthquake shaking.

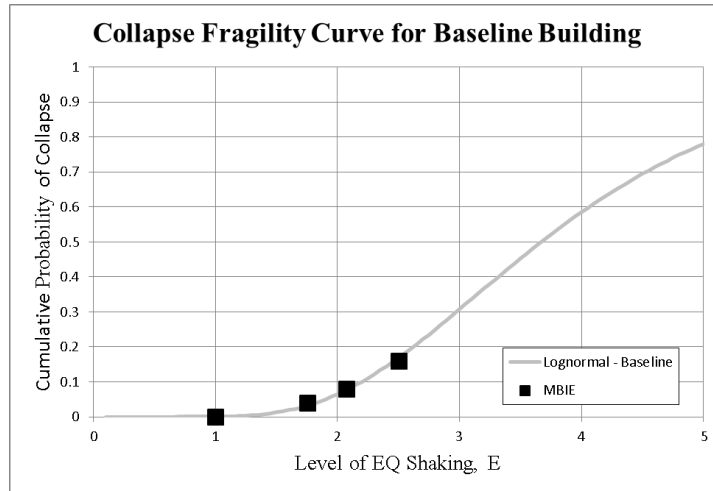


Figure 4. Collapse fragility curve for baseline building including MBIE data points

To assess relative risk, we modified the baseline fragility curve shown in Figure 4 to represent other theoretical building typologies:

1. The same building (i.e. Wellington, ductile, IL2) but with lower seismic rating than a new building including:
 - a) 67%NBS
 - b) 50%NBS
 - c) 34%NBS
 - d) 20%NBS
2. The same building (i.e. new, Wellington, ductile) but of a higher Importance Level including:
 - a) Importance Level 3 (IL3)
 - b) Importance Level 4 (IL4)
3. Less resilient buildings:
 - a) A building where collapse is actually expected around 100% ULS shaking.
 - b) A building where collapse is actually expected around 180% ULS shaking

For (1) and (2), the baseline fragility curve was scaled along the x axis while keeping the general shape the same (e.g. for 50%NBS, all ordinates move left to half the value of E for the baseline building). For (3a), the shape of the baseline building curve was adjusted to be very steep just after 500 year return period shaking. For (3b), both the shape and the scale of the baseline building curve were adjusted. The resulting fragility curves are presented in Figures 5, 6 and 7 below.

By adjusting the shape of the curve to be steeper for a more brittle building, it is assumed that the lack of ductility or resilience severely increases the likelihood the building will collapse once its strength has been reached. Whereas for a ductile building, even for very large levels of earthquake shaking (say three or four times the design or relative compliance level of shaking), there is a reasonable likelihood (approximately 50%) that collapse will not have occurred.

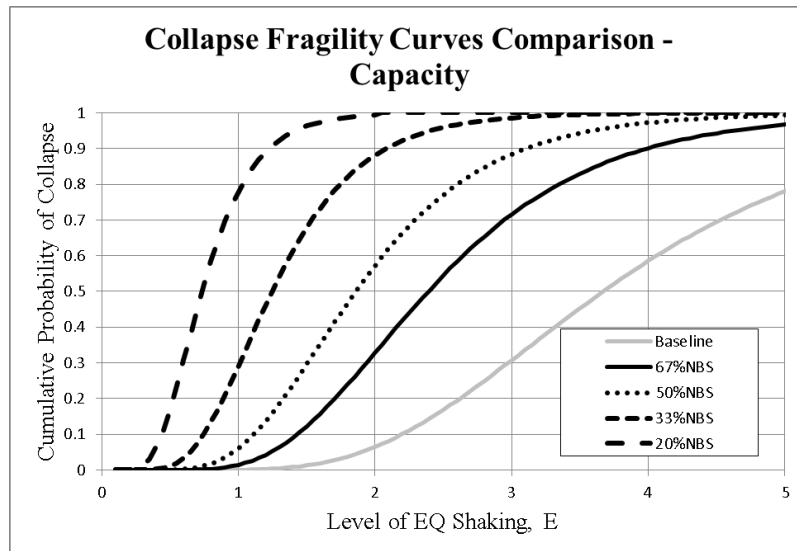


Figure 5. Collapse fragility curve comparison – capacity

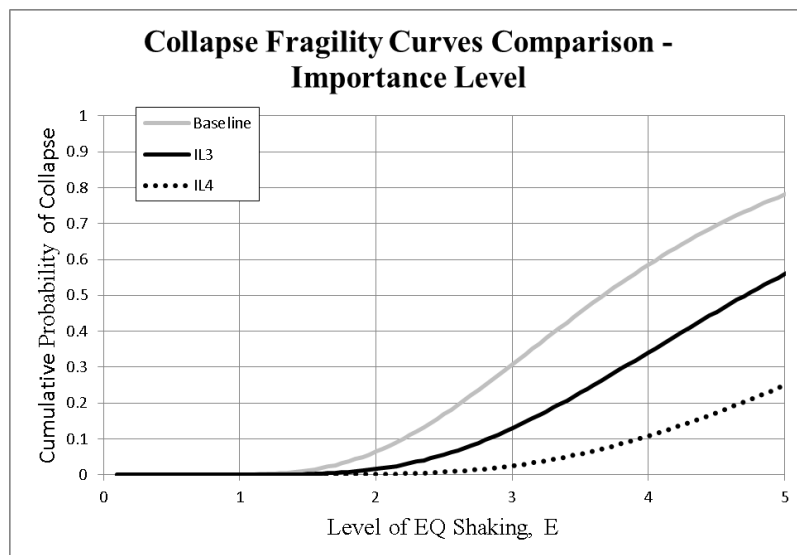


Figure 6. Collapse fragility curve comparison – importance level

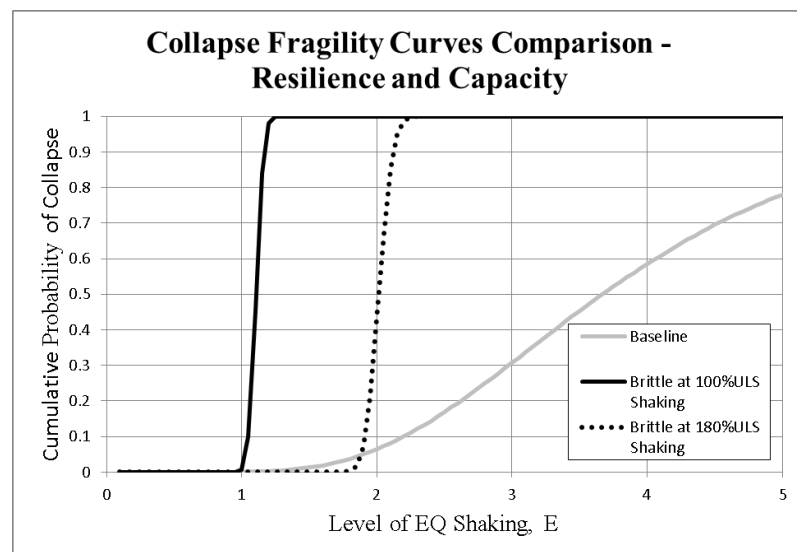


Figure 7. Collapse fragility curve comparison – resilience and capacity

3.3 Results

Table 1 below presents the results from this investigation reporting the annual probability of collapse and inferred relative risk of each building typology considered to the baseline building. The authors reiterate that these results are not intended to be definitive. They are simply the output of the methodology we have described. However, they do indicate the importance of considering building behaviour over a significant range of earthquake shaking when performance is being evaluated holistically.

Table 1. Summary of annual probabilities of collapse and relative risk.

Building Type	Annual Probability of Collapse	Relative Risk
Baseline	1.3×10^{-4}	1
67%NBS	6.5×10^{-4}	5
50%NBS	1.5×10^{-3}	12
33%NBS	4.6×10^{-3}	36
20%NBS	1.8×10^{-2}	145
IL3	4.1×10^{-5}	0.3
IL4	7.3×10^{-6}	0.06
Brittle at 100% ULS shaking	3.7×10^{-3}	29
Brittle at 180% ULS shaking	4.9×10^{-4}	4

3.3.1 Baseline building

The annual probability of collapse calculated for the baseline building of 1.3×10^{-4} agrees well with the range discussed in the Commentary to NZS 1170.5:2004 for maximum acceptable annual probabilities of collapse of 1×10^{-4} to 1×10^{-5} , and that inferred by the beta values of the BCA, NZS 3101 and NZS 3404.

3.3.2 Existing buildings

For buildings of different seismic rating, as expected, the risk increases with a decrease in seismic rating. The NZSEE Guidelines provides analogous values, which are plotted in Figure 8 below alongside the values determined in this investigation.

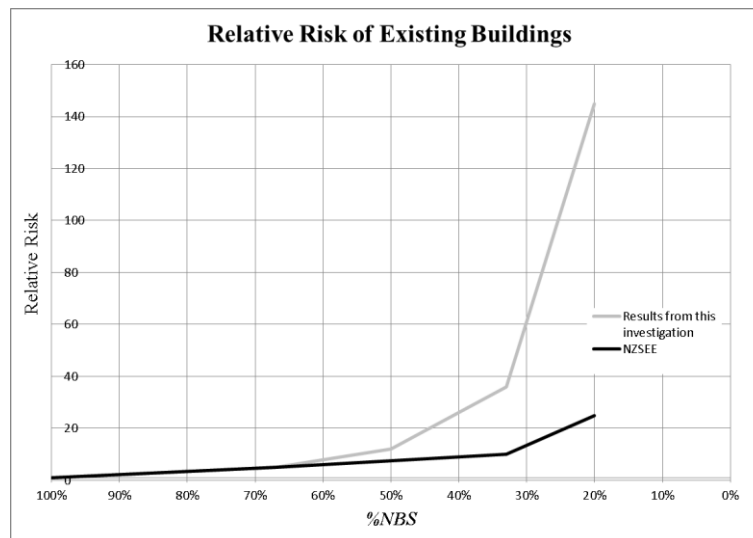


Figure 8. Relative risk of existing buildings

The values from our investigation match well with the NZSEE values until the seismic rating is below 67%NBS, at which point our values become increasingly greater.

3.3.3 *Less-resilient buildings*

The importance of resilience in seismic structural systems is demonstrated by the relative risk of the ‘Brittle at 100% ULS shaking’ building type, at approximately 30 times the risk of a ductile equivalent. This building type is intended to represent one with a critical structural weakness that will lead to step change behaviour at 100% ULS shaking.

Even for a building with plenty of strength, but still with step change behaviour, the risk is many times greater than the ductile baseline building as demonstrated by the ‘Brittle at 180% ULS shaking’ building type. This building is intended to represent the theoretical case of one with no resilience beyond 2500 year return period shaking.

3.3.4 *Contribution to annual probability of collapse*

As mentioned above, the annual probability of collapse value is calculated numerically as the summation of the product of two probabilities. At both very low and very high levels of earthquake shaking, the contribution to the sum total is expected to be negligible due to one of the probabilities in the product being near-zero. To evaluate the range of earthquake shaking that contributes most significantly to the sum total, a smoothed curve of the probability product was plotted against the intensity of earthquake (Figure 9).

Figure 9 reveals that the most significant range of earthquake shaking with respect to total probability of collapse is between one and three times 500 year return period shaking. Furthermore, only 50% of the total probability of collapse is accounted for up to 1.8 times the design level of shaking, or 2500 year return period. It is of interest that the highest contribution to the risk occurs around 180% ULS shaking.

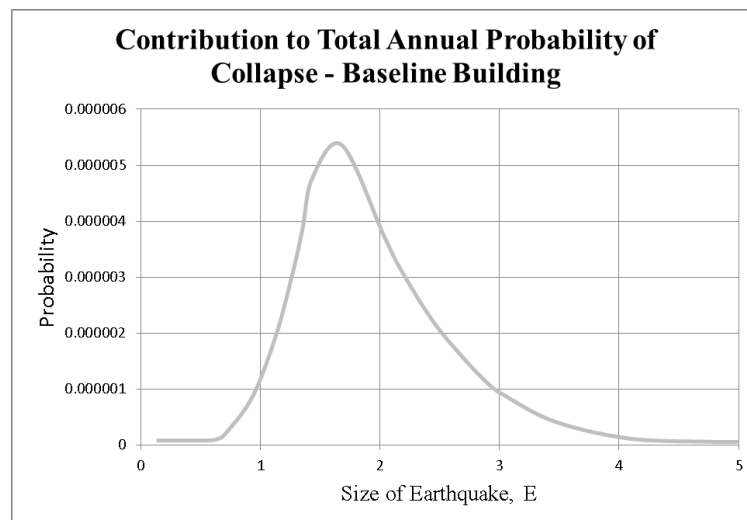


Figure 9. Contribution to total annual probability of collapse for baseline building

4 DISCUSSION

The results of our investigation have highlighted the importance of ensuring resilience in a building’s seismic systems. Furthermore, the investigation indicated that to ensure an acceptably low probability of collapse for a new IL2 building, the performance of a building needs to be considered for levels of earthquake shaking up to at least three times the normal design level (ULS shaking).

The significance of this finding is particularly relevant to innovative or alternative solutions that deviate far from the verification methods and acceptable solutions associated with the more conventional structural systems and also when considering the viability of performance-based approaches aimed at a minimum level of performance at design levels of shaking even when these are taken out to 1.5 to 2

times the usual ultimate limit state shaking levels. While it is impractical, and perhaps impossible, to require designers to calculate the annual probability of collapse of their building design, it is paramount that designers are considering that resistance against collapse is still important for shaking well beyond the 2500 year return period estimates albeit the probability of collapse will increase the more severe the shaking is.

5 CONCLUSIONS

The authors sought to investigate what acceptable seismic performance is, and the complex relationship between building capacity and acceptable performance generally which requires consideration of all levels of shaking.

We have shown, using a simplistic risk-based methodology, that the need to consider performance from a life-safety point of view is still relevant at levels of shaking out to three times that typically considered for design (ULS shaking). We have also shown that in the context of true performance-based design it is very important that the required performance is articulated in a way that can be interpreted by code writers preparing compliance requirements.

The implications for seismic design and assessment are:

- Normal design methods (referred to as Verification Methods in the New Zealand context), which employ a compliance approach (component checking of defined capacity against a defined demand) need to provide confidence that acceptable performance is achievable out to significant levels of shaking (something like three times ULS shaking levels is indicated in this investigation).
- The expected collapse probability increases as the level of shaking increases. This is expected and should be considered acceptable. The challenge is to come up with a compliance approach to design that can be expected to deliver the required performance objective. Even when an additional limit state (MCE at 150 to 180% ULS shaking) is introduced there is still a need to consider the potential performance at much higher levels of shaking if the performance objectives are to be met. In doing so, the compliance measures adopted for any shaking levels beyond ULS shaking need to reflect that the same reliability of behaviour need not be maintained at these higher shaking levels if the risk of unnecessary conservatism, especially for assessment of existing buildings, is to be avoided.
- Performance-based design methods not reflecting the probabilistic nature of building performance should be considered as simply compliance methods in another form. There should be no expectation that such methods can predict performance, and, in the authors opinion, to imply that they do presents significant risks to designers and assessors of existing buildings.
- Establishing a true performance-based approach to the seismic design of buildings is not currently considered viable. This has implications for implementation of the New Zealand Building Code objective for innovative systems which must by necessity be based around a compliance-based approach based on the verification methods.

ACKNOWLEDGEMENTS

Ministry of Business, Innovation and Employment.

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