
A Geotechnical Perspective on the 1.5 CALS Scaling Factor: Why Site-Response Analysis Is Not the Solution

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ABSTRACT

The 2022 New Zealand National Seismic Hazard Model (NSHM 2022) and SNZ TS 1170.5:2025 indicate higher seismic demands across Wellington and other high-seismic regions, prompting renewed discussion around the use of the 1.5 or 1.3 scaling factor between the Ultimate Limit State (ULS) and the Collapse Avoidance Limit State (CALS). In practice, geotechnical engineers are sometimes asked to justify or reduce this factor through site-response analysis (SRA). This paper discusses why such an approach can be conceptually inconsistent with risk-based design principles, as it conflates two distinct aspects of seismic design: modification of the ground-motion hazard and calibration of structural risk. SRA provides insight into site-specific ground-motion amplification or de-amplification, but does not address the probabilistic basis of Ψ_{CALS} , which represents the target collapse probability embedded within NZS 1170.5. The 1.5 or 1.3 factor is therefore not a correction for site response but a simplified national-level proxy for the relative exceedance probabilities between ULS and CALS performance levels. Using a hazard–fragility integration framework, this paper explores how the CALS-to-ULS demand ratio varies with the slope of the site-specific hazard curve, structural period, and fragility dispersion. The results reinforce that geotechnical input should focus on defining site-specific hazard curves, while structural engineers calculate $\Psi_{\text{(CALS)}}$, using fragility-based risk targets.

1 INTRODUCTION

Recent updates to seismic design guidance in New Zealand have led to a step-change in calculated seismic demands for many regions. The release of the 2022 New Zealand National Seismic Hazard Model (Gerstenberger et al., 2022) and its implementation through SNZ TS 1170.5:2025 have resulted in substantially higher spectral accelerations, particularly in high-seismicity regions such as Wellington. These increases have renewed industry focus on the relationship between the Ultimate Limit State (ULS) and the Collapse Avoidance Limit State (CALS), and in particular on the continued use of nominal scaling factors of 1.5 or 1.3 between ULS and CALS design actions.

Following some discussion with structural engineers, the interpretation of the CALS scaling factor, Ψ_{CALS} , varies widely. It is often treated as a generic “fudge factor”, alternately viewed as a structural factor of safety, a proxy for increased return period, or a requirement without a clear physical basis. This ambiguity has contributed to increasing expectations that geotechnical engineers should justify, refine, or reduce the CALS scaling through site-response analysis (SRA), particularly in regions such as Wellington, where seismic hazard levels are exceptionally high under NSHM 2022 and SNZ TS 1170.5. Such expectations raise a fundamental conceptual issue. Ψ_{CALS} is frequently conflated with site-response effects, despite the fact that Ψ_{CALS} is not designed to correct for soil amplification or soil nonlinearity.

2 DEFINITION OF CALS

2.1 NZSEE Guidelines

Based on NZSEE Seismic Isolation Guidelines (NZSEE, 2019), CALS is defined as the collapse avoidance limit state at which collapse of the isolated structure is to be prevented with reasonable reliability. NZS 1170.5 requires that there should be a reasonable margin to prevent collapse in a ‘rare’ earthquake beyond the ULS demand. Section 4.2.4 indicates the CALS factor is 1.5 for Important Level 2 (IL2) and 3 (IL3) buildings and 1.3 for Important Level 4 (IL4) buildings.

This definition is reaffirmed in subsequent guidance, NZSEE Interim Advice for Seismic Isolation Design using TS 1170.5 or the SESOC Design Guide for NLRHA (NZSEE, 2025), where Tables 1 to 6 consistently adopt $\Psi_{\text{CALS}} = 1.5$ for IL2 and IL3 buildings and $\Psi_{\text{CALS}} = 1.3$ for IL4 buildings.

2.2 SESOC Guidelines

The SESOC Nonlinear Response History Analysis (SESOC, 2025) provide the same definition as NZSEE (2019). SESOC (2025) further note that NZBC Verification Method B1/VM1 does not explicitly require CALS assessment, as the margins inherent in ULS design procedures within the material standards are considered sufficient to achieve acceptable collapse and fatality risk. However, the same guidance acknowledges that, in the absence of data defining appropriate values for the scaling factors Ψ_{CALS} and given the uncertainty in actual structural deformation capacity, the resulting collapse and fatality risk associated with current NLRHA procedures remains uncertain. SESOC (2025) in section 2.5 recommend an appropriate value of Ψ_{CALS} to be 1.5.

2.3 NZS 1170.5 Supp 1:2004 - Commentary

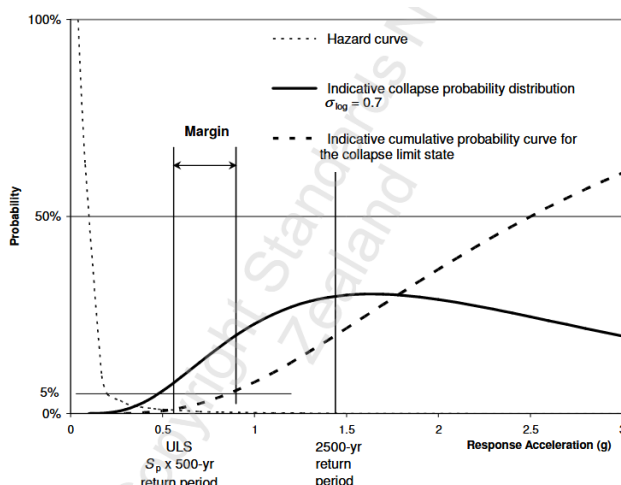


Figure 1: Hazard and Capacity Probability Distributions Compared

The main body of NZS 1170.5:2004 provides no information on the CALS. However, Commentary Section 2.1 indicates that a target annual earthquake fatality risk in the order of 10^{-6} . Given the state of knowledge at the time the standard was written regarding the variables and inherent uncertainties involved in reliably predicting when a structure will collapse, it was not currently considered practical to either analyse a building to determine the probability of collapse or to base a code verification method on a collapse limit state. Therefore, NZS 1170.5 employs different approaches, namely ULS. It has been assumed that a margin of at least 1.5 to 1.8 will be available from ULS. Figure 1 from Figure C2.2 NZS 1170.5 Supp shows the typical

relationship between the probability functions for the hazard and the capacity at the collapse limit state and the ULS design load for a typical building type.

2.4 ASCE 41-17 and ASCE 7-22

ASCE 41-17 and ASCE 7-22 have been referred to by many NZ structural engineers. The terminology of Maximum Considered Earthquake (MCE) is mentioned in NZSEE (2019, 2025) and SESOC (2025) as associated / equivalent to CALS. ASCE 7-22 and ASCE 41-17 clearly distinguish MCE for structural (MCE_R) and geotechnical (MCE_G).

Structural MCE_R is defined as Risk-Targeted Maximum Considered Earthquake Ground Motion Response Acceleration. The most severe earthquake effects considered by this standard are determined for the orientation that results in the largest maximum response to horizontal ground motions (Rot_{D100}) with adjustment for targeted risk. The MCE_R is determined as the lesser of:

1. 1% probability of collapse within a 50-year period (probabilistic), or
2. 84th percentile 5% damped spectral response acceleration in the direction of maximum horizontal response (Rot_{D100}) computed at that period (deterministic).

Geotechnical MCE_G is defined as Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration. The most severe earthquake effects considered for geometric mean peak ground acceleration and without adjustment for targeted risk. The MCE_G peak ground acceleration adjusted for site effects (PGA_M) is used for the evaluation of liquefaction, lateral spreading, seismic settlements, and other soil-related issues. The MCE_G is determined as the lesser of:

1. The geometric mean peak ground acceleration with a 2% probability of exceedance within a 50-year period (or 1/2500 years AEP in NZS 1170.5).
2. The largest 84th percentile geometric mean peak ground acceleration for scenario earthquakes on all known active faults within the site region (deterministic).

From the ASCE definition, CALS is equivalent to MCE_R probabilistic only (item 1).

2.5 SNZ TS 1170.5:2025 and SNZ TS 1170.5 Supp 1:2025 - Commentary

No information regarding CALS in this standard.

3 CALS AND SRA

In summary, CALS in New Zealand is defined as a structural performance limit state intended to ensure acceptably low collapse and fatality risk, with Ψ_{CALS} prescribed by standard and guidance documents. None of the reviewed documents defines Ψ_{CALS} as a function of site response or soil nonlinearity. Rather, it represents a simplified means of linking different performance objectives to different exceedance probabilities within a risk-based design framework.

While SRA provides valuable insight into site-specific modification of ground-motion demand, it does not address the probabilistic basis of CALS, nor the target collapse reliability embedded within seismic design standards.

We, therefore, have proposed the following to structural engineers to place the Geotech position of site response analysis in structural assessment or design:

1. Definition of MCE for Geotechnical Assessments

The Maximum Credible Earthquake (MCE) from a geotechnical perspective is based on the 1/2500-year event following ASCE 7-22 MCE_G . This represents the most severe level of ground shaking expected for

an Importance Level 2 (IL2) building with a 50-year design life and does not include any adjustment for collapse probability.

2. Use of 1/500-Year ULS Analysis, not CALS.

Geotechnically, a 1D SRA can be performed for the 1/500 year (ULS) based on either NZS1170.5 or TS1170.5 Class B rock, which would likely produce a much lower result than the spectral curves provided in TS1170.5.

3. Distinction Between Structural and Geotechnical CALS/MCE

The structural CALS/MCE in structure is different from a geotechnical perspective, where Ψ_{CALS} is a structural design factor that considers the annual probability of collapse (fatality risk) due to earthquake shaking. This factor is embedded within the NZBC framework and should not be assigned or modified by geotechnical engineers, as it is a structural design parameter.

4. Limitations of SRA Regarding Ψ_{CALS}

While a 1D SRA can be used to refine ULS seismic loads, it does not provide recommendations on an appropriate Ψ_{CALS} factor.

5. Approach to Developing Ψ_{CALS}

Determining Ψ_{CALS} requires integrating a site-specific hazard curve with a structural fragility (collapse probability) curve, as illustrated in Figure C2.2 of NZS 1170.5 and described in ASCE 7-22 for risk-targeted MCE_R development. In practice, a geotechnical engineer can develop the site-specific hazard curve using PSHA, while a structural engineer can undertake fragility curve analysis and define the target collapse probability. Subsequently, geotechnical/structural engineers can integrate these datasets to derive a suitable Ψ_{CALS} value.

6. Incorporating site-specific Non-Ergodic PSHA

A more advanced approach involves combining site-specific 1D SRA with PSHA to form a site-specific non-ergodic PSHA. This approach explicitly incorporates the local site response into the hazard model, removing the regional “ergodic” assumption and reducing aleatory uncertainty in ground-motion estimates. This can provide a more accurate hazard curve for design, which would likely produce lower Ψ_{CALS} than using standard PSHA.

4 RECENT DEVELOPMENTS BY THE SEISMIC RISK WORKING GROUP (SRWG)

The Seismic Risk Working Group (SRWG), responsible for the development of SNZ TS 1170.5, has undertaken a series of studies to ensure alignment with the New Zealand Building Code (NZBC) objective of safeguarding people from injury during seismic events (Hulsey et al., 2025). These studies examined the relationship between seismic hazard, structural performance, and societal risk, with particular focus on annual individual fatality risk (AIFR).

SRWG analyses indicate that, for Importance Level 2 (IL2) buildings with a design life of 50 years and seismic actions based on a 1/500 annual exceedance probability (AEP), designs in accordance with SNZ TS 1170.5 typically correspond to an AIFR in the range of approximately 10^{-6} to 10^{-5} (Hulsey et al., 2025). The lower bound of this range is consistent with the target AIFR of 10^{-6} that underpins NZS 1170.5, as articulated in the commentary to the 2004 standard.

A key distinction between NZS 1170.5 and SNZ TS 1170.5 lies in how this risk target is addressed. NZS 1170.5 relied on implicit safety margins embedded within force-based ULS design procedures, with it being assumed that a margin of approximately 1.5 to 1.8 beyond ULS demand would be available to prevent collapse. In contrast, SNZ TS 1170.5 makes the calibration of collapse risk more explicit by using updated seismic hazard models and incorporating fragility-based performance assessment.

Notwithstanding this shift toward explicit risk calibration, the numerical values of the CALS scaling factors, $\Psi_{\text{CALS}} = 1.5$ for IL2 and IL3 buildings and $\Psi_{\text{CALS}} = 1.3$ for IL4 buildings, have been carried forward from earlier guidance. To the author's knowledge, as mentioned in Section 2.5, these Ψ_{CALS} values have not yet been formally re-evaluated against the updated NSHM 2022 hazard model or reassessed in the context of site-specific hazard–fragility relationships.

5 EXAMPLE CALCULATION OF Ψ_{CALS}

To illustrate how the CALS scaling factor Ψ_{CALS} arises from risk-based principles, an example calculation is presented for a representative site in Wellington. The intent of this example is not to derive a project-specific design value, but to demonstrate the mechanisms that control the ratio between CALS and ULS demand and to highlight why Ψ_{CALS} cannot be determined directly from site-response analysis.

5.1 Probability of collapse target

The preliminary assessment by Husley et al. (2022) indicated AIFR of 1.5×10^{-5} was comparable to the risk of AEP 1/500 (IL2 building with 50 years design life). This is equivalent to an annual collapse rate of 1.5×10^{-4} , or ~approximately 0.7%, over 50 years, just below the ASCE 7-22 1% target (see Section 2.4). We run both 0.7% and 1% probability of collapse in 50 years.

5.2 Site-specific hazard curves

A site-specific seismic hazard curve was developed for a Wellington site with representative site class IV – Moderately stiff or medium dense soil corresponding to $V_{s30} \approx 275$ m/s. We utilised the hazard curve tool available through the GNS Science web tool (<https://nshm.gns.cri.nz/>). The web tool is based on the 2022 New Zealand National Seismic Hazard Model (NSHM), which underpins SNZ TS 1170.5 and is constructed from a collection of component models for both seismicity-rate and ground-motion characterisation (Gerstenberger et al., 2022). For this example, we select the hazard curve for the spectral acceleration (SA) at 0.2s.

5.3 Structural fragility model

We use a simplified collapse-fragility model developed by Luco et al. (2007), which serves as the basis for MCE_R in ASCE 7-22.

$$f_{\text{Capacity}}(c) = \Phi\left(\frac{\ln c - (\ln SA_{0.7 \text{ or } 1\%}) + 1.28\beta}{\beta}\right) \frac{1}{c\beta} \quad (1)$$

where β (beta) = log-normal-standard deviation; c = collapse capacity, expressed in the same SA used for hazard curve; $SA_{0.7 \text{ or } 1\%}$ = Spectral acceleration with 0.7% or 1% probability of collapse.

The initial log-normal-standard deviation (beta) proposed by Luco et al. is 0.8. However, in ASCE 7-22, 0.6 was used as beta. The recent study by Husley et al. (2022) indicates, for the NZ context, a beta in the range between 0.35 and 0.45, which indicates NZ buildings have strong quality control and predictable behaviour, or collapse occurs in a relatively narrow intensity range compared to US buildings. We run both the model with beta set to 0.4, 0.6, and 0.8 to assess the effect on Ψ_{CALS} .

5.4 Hazard-fragility integration

According to Luco et al. (2007), the spectral acceleration with $x\%$ probability of collapse can be directly computed through numerical integration and an iterative process when the spectral acceleration value, c , results in a 0.7% or 1% probability of collapse, $P(\text{Collapse})$, in 50 years, as illustrated in Figure 2 below.

$$P(\text{Collapse}) = \int_0^{\infty} P(SA > c) f_{\text{Capacity}}(c) dc \quad (2)$$

where $P(SA > c)$ = hazard curve; $f_{\text{Capacity}}(c)$ = fragility curve.

This Luco et al. (2007) formulation is conceptually equivalent to the hazard–capacity framework illustrated in Figure C2.2 of NZS 1170.5 Supplementary Commentary and underpins the development of risk-targeted ground motions in international standards.

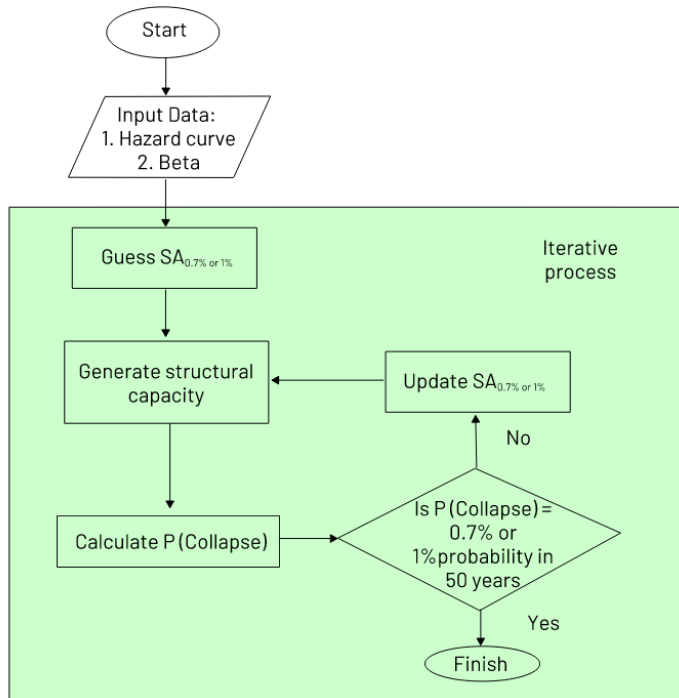


Figure 2: Flow Chart of $SA_{0.7\% \text{ or } 1\%}$

Then Ψ_{CALs} is calculated as

$$\Psi_{\text{CALs}} = \frac{SA_{0.7\% \text{ or } 1\%}}{SA_{\text{ULS}}} \quad (3)$$

5.5 Results

The calculated Ψ_{CALs} is shown in Table 1 and illustrated in Figure 3 for $\beta = 0.6$ and probability of collapse = 0.7%. It should be noted that the SRA or soil non-linearity is not included in this calculation.

Table 1: Summary of calculated Ψ_{CALs} for IL2, 50 years design life, site class IV, and building period of 0.2s

Probability of collapse	beta	SA_{ULS}	$SA_{x\%}$	Ψ_{CALs}
0.7%	0.40	1.79g	3.24g	1.81
0.7%	0.60	1.79g	3.35g	1.87
0.7%	0.80	1.79g	3.74g	2.08
1.0%	0.40	1.79g	2.91g	1.62
1.0%	0.60	1.79g	2.97g	1.66
1.0%	0.80	1.79g	3.27g	1.82



Figure 3: Hazard curve, fragility curve, and hazard-fragility integration for $SA_{0.7\%}$ and $\beta = 0.6$

The results indicated that, for NZ contexts with $P(\text{Collapse}) = 0.7\%$, the Ψ_{CALs} range is 1.8-2.1, whereas for $P(\text{Collapse}) = 1\%$ (similar to ASCE 7), the Ψ_{CALs} range is 1.6-1.8. These values remain above 1.5, as recommended in NZSEE and SESOC guidelines.

6 CONCLUSION

As a practical geotechnical engineer, this paper demonstrates that, by using NSHM 2022 and SNZ TS 1170.5:2025, the Ψ_{CALs} scaling factor (Ψ_{CALs}) is a structural design parameter representing target collapse reliability, rather than a correction for soil behaviour. Although Ψ_{CALs} is not a correction for site response, its magnitude is directly linked to the site-specific hazard curve, which is governed by local seismicity and site characteristics, such as V_{s30} , and by detailed site-response modelling.

The following conclusions are drawn:

- **SRA Limitations:** While site-response analysis (SRA) provides valuable data on ground-motion modification, it cannot be used to justify a reduction in Ψ_{CALs} because the factor is not defined as a function of site response or soil nonlinearity.
- **Risk Consistency:** Integration of Wellington-specific hazard curves with structural fragility models suggests that Ψ_{CALs} values required to meet current life-safety targets may range from 1.6 to 2.1 (for the assumptions considered). This indicates that the nominal factors of 1.3 and 1.5 may be unconservative when evaluated against the updated hazard model. However, further studies are required, especially with development of appropriate fragility curves.
- **Professional Boundaries:** Geotechnical engineers should focus on providing site-specific hazard curves through PSHA. The determination of Ψ_{CALs} remains a structural responsibility, as it is an embedded parameter within the NZBC framework linked to fatality risk.
- **Advanced Methodology:** The use of site-specific non-ergodic PSHA is recommended for complex projects, as it explicitly incorporates local site response into the hazard model and reduces aleatory uncertainty, providing a more accurate basis for risk-targeted design.

Ultimately, treating Ψ_{CALs} as a "fudge factor" to be refined by SRA is conceptually inconsistent with risk-based design principles. Accurate seismic design requires a collaborative approach where geotechnical hazard characterisation and structural fragility analysis are integrated to meet explicit societal risk targets

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