# A Review of Practice-Oriented Methods for Estimating Seismic Demands on Parts and Components

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## ABSTRACT

A variety of methods for the estimation of demands on building parts and components have been proposed in international codes and the literature. These have been reviewed as part of recent research to identify possible improvements that could be made to Section 8 of the New Zealand loadings standard. This paper reviews the outcomes of this research and identifies practice-oriented means of accounting for a range of complex factors affecting demands. These include the impact of floor height and building nonlinearity on floor acceleration demands, the amplification of demands on parts due to amplification of demands at frequencies near the modal frequencies of the structure shaking and the potential benefit of part ductility in reducing part strength requirements. Data obtained from both instrumented buildings and numerical models are used to gauge the ability of practice-oriented methods to account for these phenomena. Finally, the potential impact of adopting an alternative method to predict demands for parts is assessed for a range of case study buildings and component types. It is shown that the alternative method would likely reduce design forces for ductile parts but may prompt higher design forces for any brittle parts that are likely to exhibit similar vibrational periods as the supporting structure.

## 1 INTRODUCTION

Damage to parts and components within buildings that are not part of the primary load resisting system has been repeatedly observed to result in losses from downtime, repair and replacement costs following earthquakes (Baird & Ferner, 2017; Dhakal, 2010; Villaverde, 1997), and can pose a hazard to life safety. Consequently, assessing the seismic demands on these parts for design is a key challenge for engineering practice. Methods for ascertaining the seismic demands on these parts and components in international design standards have been observed to historically perform poorly (Haymes et al., 2020; Uma et al., 2010; Welch & Sullivan, 2017). This appears to be due to simplifications of the dynamics that influence the seismic demands on parts and components for ease of use often, at the detriment of accuracy and specificity.

Recent revisions to the American ASCE/SEI 7-22 (American Society of Civil Engineers, 2021) and the upcoming revisions to European Eurocode 8 (Fajfar & Vukobratović, 2022) were directly informed by recent research work (ATC, 2018; Vukobratović & Fajfar, 2017). The authors of the present work recently published a report (Haymes & Sullivan, 2023) that employs the growing wealth of understanding on the seismic demands on parts and components from local and international research efforts, to recommend revisions to provide a stronger basis for the relevant New Zealand design standard, NZS 1170.5:2004 (Standards New Zealand, 2016). The method presented therein was informed by ongoing consultation with practicing New Zealand structural engineers, and followed a workshop attended by academics and practitioners which identified key practical limitations and opportunities for improvement. The current work presents the method that was developed therein, discusses the basis of these recommendations, and examines the impact of their adoption.

## 2 OVERVIEW OF THE RECOMMENDED DESIGN METHOD

The recommended approach, shown in Figure 1, seeks to maintain the existing NZS 1170.5 framework and minimise alterations where possible to facilitate its adoption by practising New Zealand engineers. The recommended horizontal design earthquake action on a part, $F_{ph}$, is computed using Equation 1:

$$F_{ph} = \frac{C_p(T_p)}{\Omega_p} R_p W_p \leq 5.0\, PGA\, W_p \tag{1}$$

where $\Omega_p$ = the component overstrength factor, whose recommended value is 1.5 unless demonstrated to be greater; $R_p$ = the part risk factor; $W_p$ = the weight of the part; and $PGA$ = the peak ground acceleration. The design response coefficient for parts, $C_p(T_p)$, is determined using Equation 2:

$$C_p(T_p) = PGA \left[\frac{C_{Hi}}{C_{str}}\right]\left[\frac{C_i(T_p)}{C_{ph}}\right] \tag{2}$$

where $C_{Hi}$ = floor height coefficient for level $i$; $C_{str}$ = structural nonlinearity reduction factor; $C_i(T_p)$ = part or component spectral shape coefficient; and $C_{ph}$ = part or component horizontal response factor. Figure 1 illustrates how the recommended approach considers the parameters that influence the design actions on the part or component. The following subsections will outline how the parameters in NZS1170.5 may be modified to better represent behaviours observed in the literature.

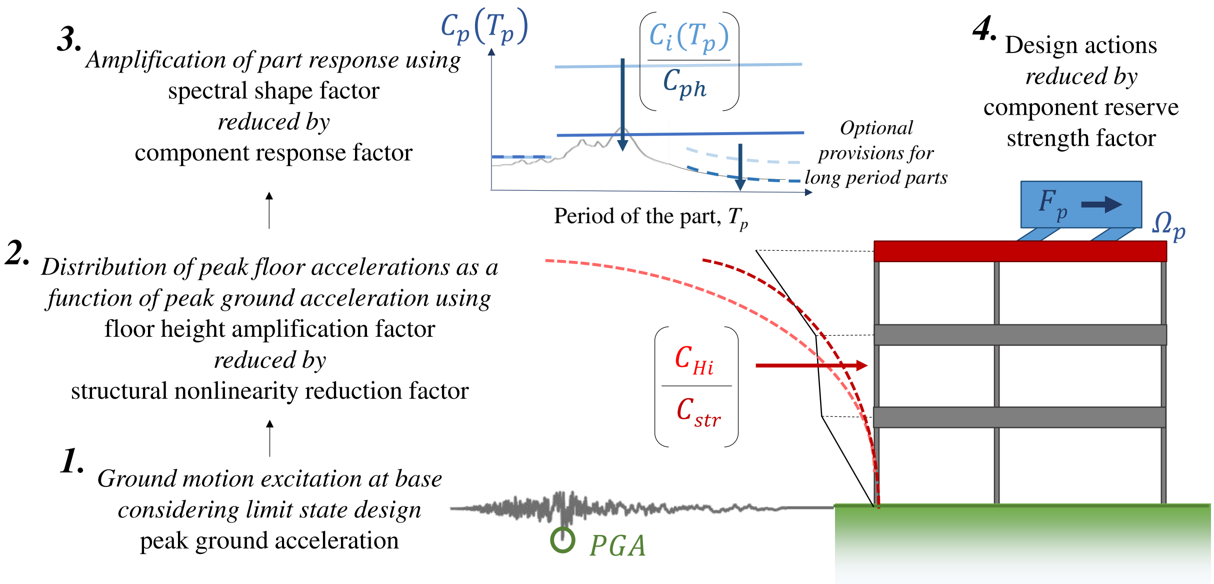



*Figure 1: Summary of the recommended approach for estimating seismic demands on parts and components.*

## 2.1 Effects of Structural Response

### 2.1.1 Floor Height Coefficient

The recommended floor height coefficient at level $i$, $C_{Hi}$, should be determined from Equation 3:

$$C_{Hi} = \begin{cases} 1 + \frac{1}{T_1}\left(\frac{h_i}{h_n}\right) + \left[1 - \left(\frac{0.4}{T_1}\right)^2\right]\left(\frac{h_i}{h_n}\right)^{10} & T_1 > 0.4\ s,\ n_{storeys} > 1 \\ 1 + 2.5\left(\frac{h_i}{h_n}\right) & T_1 \leq 0.4\ s,\ n_{storeys} > 1 \\ \frac{S_{AS}}{PGA} & n_{storeys} = 1 \end{cases} \tag{3}$$

where $h_i$ = height of attachment of the part from the base of the structure; and $h_n$ = height from the base of the structure to the uppermost seismic weight or mass in the structure. $T_1$ is the largest translational period of vibration of the primary structure in the direction being considered, and not to be taken to be less than 0.4 s. If $T_1$ is equal to or less than 0.4 s, or unknown, the floor height coefficient can be calculated using the simple form in the second line of Equation 3. This approach was developed in the ATC-120 report (ATC, 2018) using data from Californian instrumented buildings. The current NZS1170.5 approach for the floor height coefficient appears overly conservative over the lower levels of the structure, as well as for buildings with long periods which were observed in the ATC-120 report to result in less amplification with height.

The current NZS1170.5 approach provides a floor height coefficient of 1.6 for a 3.6 m-tall single-storey building. If directly adopted, the ASCE 7-22 approach would increase this to 3.5, resulting in significantly greater estimates of the peak floor acceleration and, consequently, demands on parts. The maximum peak acceleration response of a single-storey structure is theoretically limited to the design short-period ground spectral acceleration at the considered design intensity, however. The theoretical floor height coefficient may therefore be described as the ratio of the short period spectral acceleration, $S_{AS}$, to the peak ground acceleration, $PGA$, determined considering the local seismic hazard, as given in the third line of Equation 3. $C_{Hi}$ values computed in this way are closer to 1.6 than 3.5 for most sites in New Zealand. This is limited to single-storey structures because, although short buildings also likely to exhibit short periods, structures with two or more structures can exhibit higher modal responses that single-storey structures cannot.

### 2.1.2 Structural Nonlinearity Reduction Factor

The development of nonlinear structural response, through material inelasticity or geometric nonlinearity, may result in beneficial reductions in the demands on parts and components (Adam et al., 2013; Haymes, 2023; Vukobratović & Fajfar, 2017; Welch & Sullivan, 2017) which NZS1170.5 does not currently explicitly account for. The maximum reduction that can be developed is approximated using the maximum structural nonlinearity reduction factor given in Equation 4:

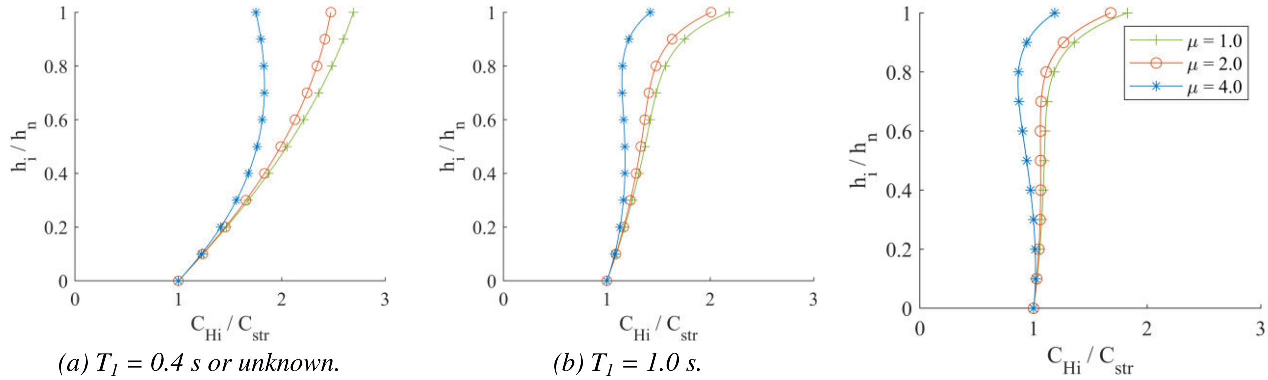
$$C_{str,max} = \sqrt{\mu} \geq 1.3 \tag{4}$$

where $\mu$ = the structural ductility factor. The lower bound value of 1.3 in Equation 4 assumes that all structures will develop the effects of nonlinearity at all considered design intensities, including sources of flexibility and nonlinearity within the structural joints, foundation and soil which may not often be explicitly modelled. Equation 4 is the same as the expression in ASCE 7-22, where reductions from structural nonlinearity are distributed equally with floor height. Empirical correlation of reductions with fundamental structural modal response, however, indicated that this may not be an adequate assumption (Haymes, 2023; Vukobratović & Fajfar, 2017; Welch & Sullivan, 2017). Here, consequently, the structural nonlinearity reduction factor, $C_{str}$ is determined from Equation 5, which raises the relative floor height to a floor height distribution exponent, $e_{str}$, computed using Equation 6:

$$C_{str} = C_{str.max}^{\ e_{str}} \tag{5}$$

$$e_{str} = \left( {}^{h_i}/_{h_n} \right)^{1.5} \tag{6}$$

Figure 2 shows the amplification of demands with floor height estimated using the ratio of the floor height coefficient, $C_{Hi}$, and the structural nonlinearity reduction factor, $C_{str}$, for a range of structural fundamental period and ductility values.




(a) $T_1$ = 0.4 s or unknown. (b) $T_1$ = 1.0 s. (c) $T_1$ = 2.5 s.

*Figure 2: Amplification with floor height using the ratio of the floor height coefficient, $C_{Hi}$, and the structural nonlinearity reduction factor, $C_{str}$, for a range of structural fundamental periods and ductilities.*

## 2.2 Effects of Part or Component Response

### 2.2.1 Part or Component Spectral Shape Factor

The part or component spectral shape coefficient, $C_i(T_p)$, describes the maximum amplification anticipated for a part of component based on elastic dynamic properties with 5% damping, and is given in Table 1.

*Table 1: Values assumed by the part or component spectral shape coefficient, $C_i(T_p)$.*

| Rigid parts or components | Flexible parts or components | |
|---|---|---|
| All levels | At or below ground level | Above ground level |
| 1.0 | $\frac{S_{AS}}{PGA}$ | 4.0 |

The classification of parts as rigid or flexible follows the approach in the ATC-120 report (ATC, 2018) to classify components based upon their likelihood of resonance with the response of the fundamental structural modal response. Many studies have observed significant dynamic amplification at component periods that are close to any structural modal periods (ATC, 2018; Haymes et al., 2020; Uma et al., 2010; Welch & Sullivan, 2017). This classification approach avoids the necessity of the estimation of the periods of the part or structural modal periods. Components deemed sufficiently rigid are expected to directly experience the peak floor acceleration without alteration, consequently having a spectral shape factor of 1.0, half of the current value of 2.0 in NZS1170.5. Classification of parts and components as rigid or flexible may be specified in tables, similar to those provided in ASCE 7-22. The value of 4.0 was recommended for flexible parts by assuming that these parts may be characterised with a damping ratio of 5%, in the absence of reliable testing of components at design intensities (ATC, 2018). The current NZS1170.5 arbitrarily reduces the component spectral shape beyond component periods of 0.75 s, implicitly assuming that the demands associated with the fundamental modal response have significantly reduced due to structural nonlinearity.

This assumption is not appropriate for the serviceability limit state, and is only valid if the fundamental structural period is greater than 1.5 s, which it need not be.

### 2.2.2 Part or Component Response Factor

Permitting the development of nonlinear part response has been observed to result in beneficial reductions in the design strength of the parts, depending on the proximity of the period of the parts to those of the structural modes (Haymes, 2023; Kazantzi et al., 2020). Until the recent release of ASCE 7-22, NZS 1170.5 was one of the only international code approaches that explicitly consider the effects of nonlinear response of the part or component. NZS 1170.5 specifies equal, and often conservative, reductions for all component periods. Table 2 provides values that more accurately describe this behaviour and capitalises on its benefit, based upon observations from instrumented buildings by Haymes (2023).

*Table 2: Recommended values for the part or component response factor, $C_{ph}$.*

| Ductility of the part or component, $\mu_p$ | Rigid parts | Flexible parts | | Long period parts* |
|---|---|---|---|---|
| | All levels | At or below ground | Above ground level | All levels |
| 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| 1.25 | 1.0 | 1.25 | 1.4 | 1.25 |
| 1.5 | 1.0 | 1.5 | 1.85 | 1.5 |
| 2.0 | 1.0 | 2.0 | 2.8 | 2.0 |
| 2.5 or greater | 1.0 | 2.5 | 4.0 | 2.5 |

*A long period part must have a fundamental period, $T_p$, greater than $T_{p,long}$, as defined in Section 2.3.

## 2.3 Long Period Parts or Components

Observations from instrumented buildings found that the peak demands on parts with periods that are much greater than those of the modes of building are correlated closely with those expected at the ground level, irrespective of location in the building. As the period of the part becomes greater, the relative motion from the modal response of the structure influences the response of the part less significantly. Consequently, there is a transition between parts with periods that are near the fundamental structural mode which exhibit responses that are primarily determined from the modal response of the structure, to parts that exhibit responses that are well approximated using the corresponding ground response spectrum. If parts are found to have a period greater than the threshold long period computed using Equation 10, the design response coefficient for long period parts, $C_p(T_p)$, is computed using Equation 9:

$$C_p(T_p) = \frac{S_a(T_p)}{C_{ph}}\left[1 + \frac{1}{\left(\frac{T_p}{T_1} - 1\right)^2}\right] \tag{9}$$

where $S_a(T_p)$ = the spectral acceleration of the ground motion at the period of the part, determined from the seismic hazard at the considered design level intensity, and $C_{ph}$ = the part response factor. The expression within the square brackets accounts for the transition between parts with periods that are near the fundamental structural mode which exhibit responses that are primarily determined from the modal response of the structure, to parts that exhibit responses that are well approximated using the corresponding ground response spectrum. It is recommended that long period parts and components should be defined as those possessing a period, $T_p$, that is greater than the threshold long period $T_{p,long}$, defined using Equation 10:

$$T_{p,long} = T_1(1 + \sqrt{\mu}) \geq 2T_1 \tag{10}$$

where $T_1$ = the largest translational period of vibration of the primary structure in the direction being considered, and $\mu$ = the structural ductility factor. Where the structure is expected to respond elastically, the threshold long period is equal to twice the fundamental structural period. This form accounts for the effects of period elongation due to inelastic softening of the structural system (Haymes, 2023; Sullivan et al., 2013).

## 3 COMPARING THE CURRENT AND RECOMMENDED APPROACHES

An indicative application of the current and recommended approaches is shown in Figure 3, where estimated demands are compared to roof acceleration response spectra, normalised by the peak ground acceleration, generated from time-history analyses of steel moment resisting frame and reinforced concrete wall buildings by Welch & Sullivan (2017). The fundamental periods of these four-, eight-, and twelve-storey buildings, range from 1.2 s to 2.7 s. Analyses were conducted using the FEMA P695 record set (FEMA 2009), incrementally scaling the peak ground accelerations in steps of 0.15 g. Structural ductility values were estimated from the results of each record. Figure 3 shows spectra corresponding to part ductility values, $\mu_p$, of 1.2 and 2.0, and for structural ductility values, $\mu$, within 15% of 1.0 and 4.0. Further information on the models are given in Welch (2016).

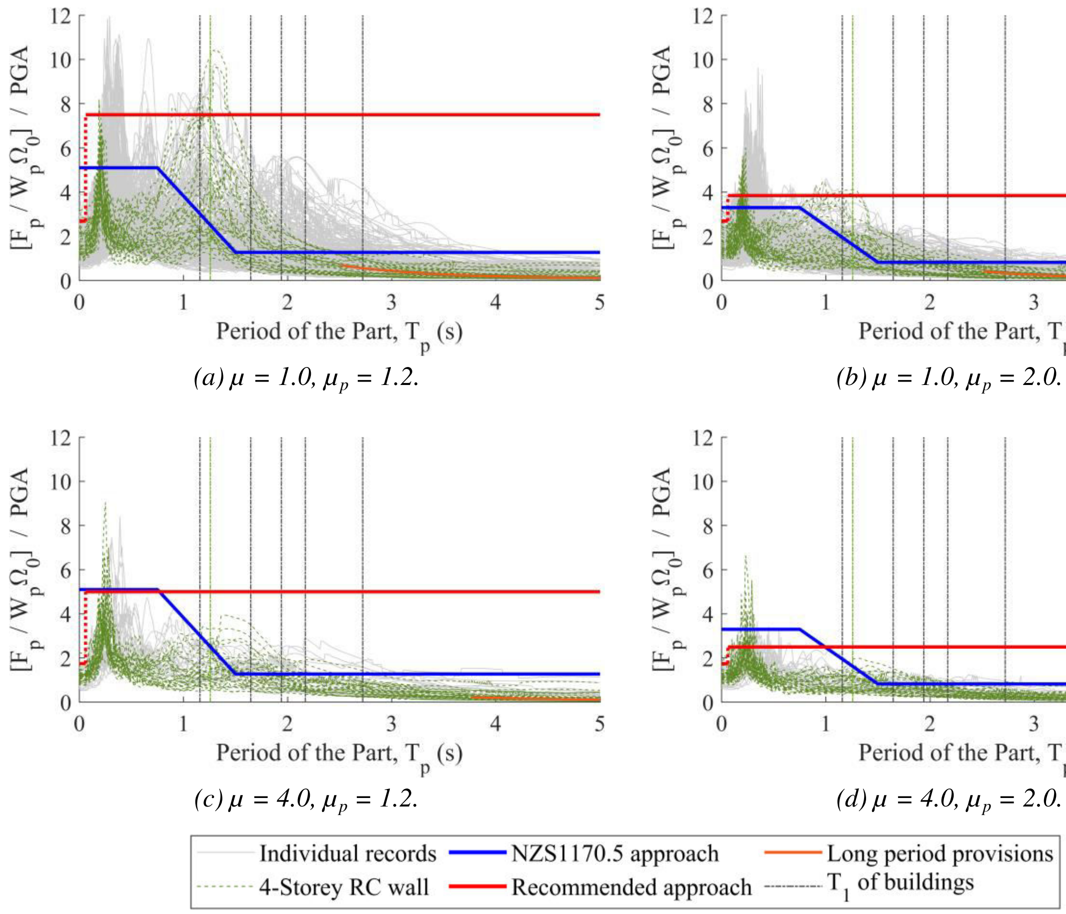



*Figure 3: Roof acceleration spectra from results obtained using numerical modelling of steel moment resisting frames and reinforced concrete wall buildings for a pair of structural and component ductility combinations. Predicted demands using the current NZS1170.5 and recommended approaches are indicated.*

Figure 3 indicates that the NZS 1170.5 approach produces highly conservative estimations of the peak floor accelerations for rigid components with very short periods, particularly if the structure is permitted to develop a significantly nonlinear response. The conservatism reduces with increasing part nonlinearity, as NZS 1170.5 reduces the estimated peak floor acceleration with increasing part ductility, whereas it has been well established that rigid parts do not exhibit this behaviour (Haymes, 2023; Kazantzi et al., 2020; Vukobratović & Fajfar, 2017). The recommended approach, conversely, is able to estimate the peak floor acceleration with greater accuracy. The recommended approach is applied using the floor height coefficient of 3.5, corresponding to the conservative recommendations for unknown fundamental structural periods (line two of Equation 3), which results in a more conservative estimation of the peak floor accelerations than if the fundamental period was specified for these cases (line one of Equation 3). It is anticipated that design loads on rigid parts and components will be lower using the recommended approach in most design applications. Figure 3(a) shows that the current NZS 1170.5 approach is non-conservative for flexible components in structures that develop limited nonlinearity, as expected at the serviceability limit state, particularly if the fundamental structural period is longer than 0.75 s. The recommended approach may provide a more accurate estimate of these demands and does not require estimates of the modal periods of the structure or part. Figure 3 also provides long period part estimations computed for the 4-storey RC wall building using the NZS1170.5 soil class C ground spectral shape factor, which are observed to provide reasonable estimations considering the variation between ground motion frequency content of individual records.

The recent report by the authors (Haymes & Sullivan, 2023) trialled the recommended revisions using illustrative example buildings with loading representative of the 2022 New Zealand seismic hazard model results. The results were compared with those obtained through application of the current NZS 1170.5 parts and components approach and hazard, as well as the current NZS 1170.5 parts and components approach with the updated national seismic hazard. The results suggest that demands on parts that are expected to be to exhibit similar periods to the building period, and those that are characterised by low ductility capacity, would be expected to attract larger design loads. For most parts, however, the results indicate that substantial reductions in design loading may be achieved using the updated approach, particularly for parts and components that are rigid, that develop nonlinear response, or that are mounted on the lower levels of the structure. Detailed comparisons can be found in the aforementioned report (Haymes & Sullivan, 2023).

## 4 CONCLUSIONS

Possible improvements to the approach for the seismic loading on parts and components in NZS 1170.5 are recommended. The recommendations represent growing understanding of parameters that effect this loading, and follows the trend in international code revisions. This robust empirical approach considers the impact of floor height and building nonlinearity on floor acceleration demands, the amplification of demands on flexible parts due to dynamic amplification associated with the response of structural modes, and potential reductions in part strength requirements by permitting nonlinear part response. The recommended approach addresses the overly conservative estimates on rigid part and components, and provides a more accurate estimation of flexible components, including the allowance of lower loads for parts demonstrated to have periods greater than the fundamental period of the supporting structure, without a significant increase in loading compared with the current approach.

## 5 ACKNOWLEDGEMENTS

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