Implications for the Design of Parts from the Updated New Zealand Seismic Loading Standard and Seismic Hazard

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ABSTRACT

New Zealand engineering practitioners will see revisions to the seismic loading standards for non-structural elements (parts and components) in the draft technical specification TS1170.5. This paper considers the implications for the horizontal design force on parts and components by applying the approach to case study structures of different heights in Christchurch and Wellington. The updated standards and current national seismic hazard model (NSHM) loadings are compared with those derived from the previous standard, NZS1170.5:2004 A1 2016, and the previous NSHM. The results suggest that demands on parts that exhibit a vibrational period close to the building modal periods and that are characterised by low ductility capacity would be expected to require larger design strengths. However, for most parts, the results indicate that substantial reductions in design strength requirements may be achieved using the updated approach, particularly for parts and components that are rigid, develop nonlinear response, or are mounted over the lower levels of the structure.

1 INTRODUCTION

Growing local and international knowledge and lessons from recent earthquakes have precipitated changes to the design earthquake loading on non-structural elements (hereafter referred to as "parts" for brevity). This, and the recently updating of the New Zealand National Seismic Hazard model, have driven the development of the upcoming New Zealand National Seismic Hazard Model (NSHM) and the upcoming draft technical specification TS1170.5 that prescribes the procedure for establishing design demands on parts. Here, the parts procedure recently recommended by the authors (Haymes et al., 2024; Haymes & Sullivan, 2023) for inclusion in the draft TS 1170.5 (henceforth simply referred to as "TS 1170.5" or the "TS") is compared with the current design provisions in Section 8 Requirements for Parts and Components of the New Zealand seismic loading standard NZS 1170.5:2004 A1 (Standards New Zealand, 2016) (hereafter also referred to as just “NZS”). This paper uses case studies to provide a direct comparison of the parts procedures of the TS to the NZS by comparing the horizontal force demands on the part normalised by the weight of the part, F_p/W_p. The parameters and behaviours that drive differences in the parts procedures are discussed. For an explanation of the basis of the new parts procedure in the TS, refer to Haymes et al. (2024).

2 OVERVIEW OF CASE STUDIES

Design seismic demands on parts are examined here considering hypothetical case study low-rise (two-storey) and medium-rise (twelve storey) importance level 2 buildings, having fundamental periods of 0.30 s and 1.6 s, respectively. Both structures comprised inter-storey heights of 4.0 m and were considered to exhibit no nonlinear behaviour at the serviceability limit state, SLS, (i.e.: building ductility factor of 1.0) and ductile behaviour at the ultimate limit state (ULS), characterised with a ductility factor of 1.5. The seismic hazard is characterised in the parts procedures using the peak ground acceleration, *PGA*. This was investigated for rock sites (Soil Class A in NZS / and Class I in TS) in both Christchurch, representing a region with small changes in the NSHM; and Wellington, representing a region with significantly increased NSHM demands. Near fault effects were neglected for this study.

3 DESIGN LOADS CONSIDERING CHANGING HAZARD AND PARTS PROCEDURE

3.1 Rigid Parts

Parts may be considered to be sufficiently rigid if they do not exhibit greater peak accelerations than the peak floor acceleration, as is induced in flexible parts which have a period close to a period of the modes of the supporting structure, (i.e.: they do not experience dynamic amplification). Some work proposes that this may be achieved if parts have periods less than 0.06 s (American Society of Civil Engineers, 2021; Applied Technology Council, 2018). Although caution should be taken when classifying rigidity (Kehoe, 2022), recent work by the authors proposed that classifications may be facilitated by tables for common parts (Haymes et al., 2024). The nonlinear capacity of a rigid part will not alter the peak forces as develop, depending instead only on the peak floor acceleration. The TS approach seems to emulate these observations by classifying parts as rigid or flexible, and assigning values the parameters that account for amplification (the part spectral shape factor, $C_i(T_p)$) and reduction (the part response factor, C_{ph}) as appropriate for their expected behaviour. This is not how the NZS approach works, however, instead assigning an amplification of 2.0 for parts with periods less than 0.75 s, and reducing all parts by the same factor independent of their periods.

Tables 1 and 2 present the normalised force demands on rigid parts in the case study buildings situated on Christchurch and Wellington rock sites. In all cases, the demands acting on rigid parts are significantly reduced.

For parts mounted at the ground level, the design procedure in NZS states that they must be designed as if a stand-alone structure using earlier chapters. This is formalised into the parts section of the TS, but the method is essentially unchanged, and the change in demands at SLS reflects the relative change in *PGA*. However, the newly introduced part reserve capacity factor, Ω_p, considers the ratio of the likely strength to the design strength and is taken as 1.5 for the ultimate limit state

(ULS). Consequently, the 113% increase in *PGA* at ULS for the Wellington rock site results in only a 42% increase in design loads for rigid parts mounted at ground level.

Normalised force demands on rigid parts mounted higher in the structure are significantly lower using the TS approach, due to (i) lower amplification of peak floor accelerations with floor height described by the floor height coefficient, C_{Hi} , for the twelve-storey structure, but not the two-storey structure; (ii) the consideration of the beneficial reductions from structural inelasticity reducing peak floor accelerations at ULS; and most significantly (iii) the part or component spectral shape factor, $C_i(T_p)$, being taken as 1.0 (i.e: no dynamic amplification) in the TS approach instead of 2.0. The NZS approach permits this amplification to be reduced using the part response factor, C_{ph}: a parameter that broadly accounts for the reduction in the required strength of nonlinear part response, but which does not occur for rigid parts. Because NZS assumes all parts can achieve some level of ductile response, the practical reduction in design loads is less than shown in Tables 1 and 2 where the part ductility is taken as 1.0. The TS approach permits designers to stiffen their parts to be sufficiently rigid as to use reduced design loads, which is a procedure that is not currently available using the NZS approach.

Table 1. Normalised force demands on rigid parts in the case study buildings situated on a Christchurch rock site.

		Serviceability Limit State			**Ultimate Limit State**		
Building	**Level**	**NZS**	**TS**		**NZS**	**TS**	
12-storey	Roof	0.45 g	0.12 g	(-74%)	1.80 g	0.43 g	(-76%)
	Floor 6	0.45 g	0.07 g	(-84%)	1.80 g	0.26 g	(-85%)
	Ground	0.08 g	0.06 g	(-20%)	0.30 g	0.22 g	(-27%)
2-storey	Roof	0.35 g	0.16 g	(-54%)	1.40 g	0.59 g	(-58%)
	Floor 1	0.25 g	0.12 g	(-51%)	1.00 g	0.45 g	(-55%)
	Ground	0.08 g	0.06 g	(-20%)	0.30 g	0.22 g	(-27%)

Table 2. Normalised force demands on rigid parts in the case study buildings situated on a Wellington rock site.

		Serviceability Limit State			**Ultimate Limit State**		
Building	**Level**	**NZS**	**TS**		**NZS**	**TS**	
12-storey	Roof	0.60 g	0.22 g	(-64%)	2.40 g	1.12 g	(-53%)
	Floor 6	0.60 g	0.13 g	(-78%)	2.40 g	0.68 g	(-72%)
	Ground	0.10 g	0.11 g	(+10%)	0.40 g	0.57 g	(+42%)
2-storey	Roof	0.47 g	0.30 g	(-37%)	1.87 g	1.53 g	(-18%)
	Floor 1	0.33 g	0.23 g	(-32%)	1.33 g	1.16 g	(-13%)
	Ground	0.10 g	0.11 g	(+10%)	0.40 g	0.57 g	(+42%)

3.2 Flexible parts at the Serviceability Limit State

Flexible parts can attract greater demands than rigid parts due to dynamic amplification. At SLS, neither parts or the structural system supporting them are permitted to develop nonlinearity, resulting in greatest amplification of *PGA* demands is expected for parts at this design level. This can be observed in Table 3, which provides the normalised force demands computed for flexible parts at SLS design level intensity for the Christchurch and Wellington rock sites, where the TS approach provides design forces that are governed by the upper bound of 7.5*PGA* (0.45g for Christchurch and 0.83g for Wellington) at the top of both buildings and the mid-height of the two-storey buildings. As this is 25% greater than 6*PGA* prescribed by the NZS approach, the difference at the roof level of the 12-storey structure predominantly reflects the change in the seismic hazard. The increase in the 2-storey structure is driven by the change in the expression for C_{Hi}. The spectral shape factor, *Ci(Tp)*, appears to have been developed in NZS based on an expected shape in strong ground motions that resulted in structural inelastic response, and thus poorly describes the expected shape of floor acceleration response spectra at SLS. The TS approach instead attempts to describe, with adequate allowances, the dynamic amplification anticipated when the period of a part is close to those of the vibrational modes of the supporting structure. The TS approach also permits the use of alternate provisions which result in significantly lower design demands than the TS flexible and NZS approaches, provided that a part can be demonstrated to have a sufficiently long period as to be expected not to exhibit dynamic amplification induced by the modal response of the supporting structure. Further note that to apply the NZS approach, flexible parts mounted at ground level were assumed to have a $C_i(T_p)$, of 2.5 (approximating the ratio of the short period spectral acceleration, S_{as} and *PGA*) and C_{ph} was set equal to the inverse of the part ductility factor.

Table 3: Normalised force demands on flexible parts at SLS design level intensity.

		Christchurch			Wellington		
Building	**Level**	**NZS**	**TS**		**NZS**	**TS**	
12-storey	Roof	0.45 g	0.45 g	(0%)	0.60 g	0.83 g	(+38%)
	Floor 6	0.45 g	0.29 g	(-36%)	0.60 g	0.53 g	(-12%)
	Ground	0.18 g	0.15 g	(-15%)	0.24 g	0.28 g	(+17%)
2-storey	Roof	0.35 g	0.45 g	(+29%)	0.47 g	0.83 g	(+77%)
	Floor 1	0.25 g	0.45 g	(+80%)	0.33 g	0.83 g	(+148%)
	Ground	0.18 g	0.15 g	(-15%)	0.24 g	0.28 g	(+17%)

3.3 Flexible parts at the Ultimate Limit State

Permitting nonlinear behaviour of parts and components allows for the design of the parts and their restraints to lower strengths than is needed to remain elastic under earthquake loading (Applied Technology Council, 2018; Haymes et al., 2024; Kazantzi et al., 2018). This beneficial parameter appears to have been under-utilised in NZS, and is fully incorporated into the TS approach. Designers are likely to be encouraged to use tables that were recommended to be included in the commentary of the TS that provide default classifications and part ductility for many commonly used parts. These were developed to avoid the need for designers to attempt to derive these from first principles or by test, although both of these approaches may likely still be permitted. It should be further noted that the values of part ductility may be higher than designers have previously used, acknowledging that the displacements required to develop reductions in design strength arise generally from the development of even small nonlinear displacements such as bolt-slip. In this context, part ductility may therefore be considered as a measure of nonlinearity that characterises the expected reduction in design strength.

Tables 4 and 5, which provide the normalised force demands on flexible parts with brittle, low and high ductile nonlinear capacities at ULS design level intensity situated on Christchurch and Wellington rock sites, show that while design loads may increase significantly for brittle parts (particularly if the seismic hazard has significantly increased), but may be much lower for parts that exhibit nonlinear capacity. Indeed, the commentary to NZS 1170.5 currently states that almost all parts are expected to have some ability to accommodate nonlinear deformation. The TS approach also allows designers to explicitly account for the expected structural displacement ductility at ULS, which is likely to produce lower demands than the NZS approach.

Table 4: Normalised force demands on flexible parts with brittle, low and high ductile nonlinear capacities at ULS design level intensity situated on a Christchurch rock site.

		Brittle, μ_p = 1.0			**Low ductility, μ_p = 1.5**			**High ductility, μ_p = 2.5**		
Building	**Level**	**NZS**	**TS**		**NZS**	**TS**		**NZS**	**TS**	
12-storey	Roof	1.80 g	1.65 g	(-8%)	1.35 g	0.94 g	(-31%)	0.90 g	0.43 g	(-52%)
	Floor 6	1.80 g	1.05 g	(-41%)	1.35 g	0.57 g	(-58%)	0.90 g	0.26 g	(-71%)
	Ground	0.71 g	0.55 g	(-22%)	0.40 g	0.37 g	(-8%)	0.20 g	0.22 g	(+11%)
2-storey	Roof	1.40 g	1.65 g	(+18%)	1.40 g	1.28 g	(-9%)	1.40 g	0.59 g	(-58%)
	Floor 1	1.00 g	1.65 g	(+65%)	1.00 g	0.98 g	(-2%)	1.00 g	0.45 g	(-55%)
	Ground	0.71 g	0.55 g	(-22%)	0.40 g	0.37 g	(-8%)	0.20 g	0.22 g	(+11%)

Table 5: Normalised force demands on flexible parts with brittle, low and high ductile nonlinear capacities at ULS design level intensity situated on a Wellington rock site.

		Brittle, μ_p = 1.0			**Low ductility, μ_p = 1.5**			**High ductility, μ_p = 2.5**		
Building	**Level**	**NZS**	**TS**		**NZS**	**TS**		**NZS**	**TS**	
12-storey	Roof	2.40 g	4.25 g	(+77%)	1.80 g	2.42 g	(+34%)	1.20 g	1.12 g	(-7%)
	Floor 6	2.40 g	2.71 g	(+13%)	1.80 g	1.47 g	(-19%)	1.20 g	0.68 g	(-43%)
	Ground	0.94 g	1.42 g	(+51%)	0.53 g	0.94 g	(+77%)	0.26 g	0.57 g	(+115%)
2-storey	Roof	1.87 g	4.25 g	(+128%)	1.40 g	3.30 g	(+136%)	0.93 g	1.53 g	(+63%)
	Floor 1	1.33 g	4.25 g	(+219%)	1.00 g	2.51 g	(+151%)	0.67 g	1.16 g	(+74%)
	Ground	0.94 g	1.42 g	(+51%)	0.53 g	0.94 g	(+77%)	0.26 g	0.57 g	(+115%)

4 CONCLUSIONS

New Zealand engineering practitioners may soon need to adopt different design loads for non-structural parts and components as a result to draft revisions to the seismic loading standard and national seismic hazard model. The impact of these changes has been demonstrated numerically in this paper using illustrative case studies, from which the parameters that influence demands on parts were discussed, and the underlying behaviours which govern them were explained. Christchurch and Wellington rock sites were chosen to demonstrate loading for regions with small and large changes to the seismic hazard, respectively. Two- and twelve-storey structures were examined. It may be concluded that increases are expected for flexible parts that have a likelihood of possessing a vibrational period near those of the structural modes and are not permitted to develop nonlinear response. Designers can, however, substantially reduce the design strength requirements by ensuring parts possess sufficiently short or long periods to avoid dynamic amplification, are mounted over the lower levels of the structure, or can accommodate nonlinear deformation. Guidance on the classification rigidity and part ductility may be assumed from the tables, which can facilitate the use of the approach without strenuous engineering effort.

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