A simple method to predict floor response spectral demands on nonstructural components

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ABSTRACT

This paper summarises a newly proposed practice-oriented method to predict elastic floor acceleration response spectra for estimating demands on nonstructural components. The approach builds on previous contributions in the literature, making specific recommendations to explicitly consider floor displacement response spectra, and to account for uncertainty in modal characteristics. The method aims to provide reliable predictions which improve on existing code methods, while maintaining simplicity to facilitate adoption in design and assessment. The method is verified against floor motions recorded in an instrumented building in New Zealand. This work is motivated by recent seismic events which have illustrated the significant costs that can be incurred following damage to secondary and nonstructural components within buildings, even where the structural system has performed well. Such observations have prompted increased attention to the seismic performance of nonstructural components with questions being raised about the accuracy of design floor acceleration response spectra used in practice. A comparison with existing code provisions from NZS1170.5, Eurocode 8, and ASCE 7-16 shows that the proposed approach performs well, particularly if a good estimate of the building's fundamental period of vibration is available. To illustrate its use with New Zealand standards, a design floor acceleration spectrum is computed at the NZS1170.0 and NZS1170.5 serviceability limit state.

1 INTRODUCTION

This paper summarises a recently proposed practice-oriented method for predicting floor acceleration and displacement response spectra (Haymes et al. 2020). The method is based on the concept of modal superposition, and its scope is currently limited to structural systems responding within their elastic range. The method is validated against floor acceleration response data recorded from the UC Physics building under the 2011 M6.3 Lyttelton earthquake. Comparisons are also made to floor response spectrum prediction methods employed by seismic design codes such as NZS1170.5 (Standards New Zealand 2016); Eurocode 8

(European Committee for Standardization 2004); and ASCE 7-16 (American Society of Civil Engineers 2017).

This work is motivated by the recognition that despite decades of improvement in the seismic design of buildings, earthquakes continue to cause significant damage and disruption (Dhakal et al. 2016; Sullivan et al. 2013). The losses and downtime associated with damage to nonstructural components have been observed to be particularly high (Filiatrault and Sullivan 2014). Damage to nonstructural components has at times even attracted scrutiny over the future of otherwise repairable structures (Dhakal et al. 2016).

Reliable seismic design of nonstructural components is impeded by the difficulty in predicting the acceleration and displacement demands imposed on them. Acceleration demands can be estimated using either empirical floor acceleration response spectrum prediction equations or advanced numerical dynamic response simulations. Empirical floor response spectrum prediction methods in current seismic design standards have been shown by a number of studies to be inadequate (Anajafi and Medina 2018; Calvi and Sullivan 2014; Flores et al. 2015), prompting the need to develop improved yet simple prediction methods for use in design and assessment practice.

2 PROPOSED FLOOR ACCELERATION RESPONSE SPECTRUM PREDICTION METHOD

The floor acceleration response spectrum represents the peak pseudo-acceleration demand imposed on a nonstructural element located at a floor as a function of its fundamental period of vibration, T_{NS}. Figure 1 provides an overview of the method for constructing floor acceleration response spectra proposed in this work. The approach uses the mode shapes and periods of the building (Fig. 1, left) to first compute the peak floor acceleration demands for each mode. Subsequently, dynamic amplification factors that depend on the damping ratio of the nonstructural component and the ratio of the nonstructural element fundamental period to each building period are used to establish the contributions of different modes to the total floor acceleration response spectrum (Fig. 1, right). Finally, the predicted floor acceleration response spectrum is computed as the envelope of all of the modal contributions and the ground acceleration response spectrum. To assist in more easily estimating the parameters required, a number of assumptions and simplifications can be made, as further discussed below.

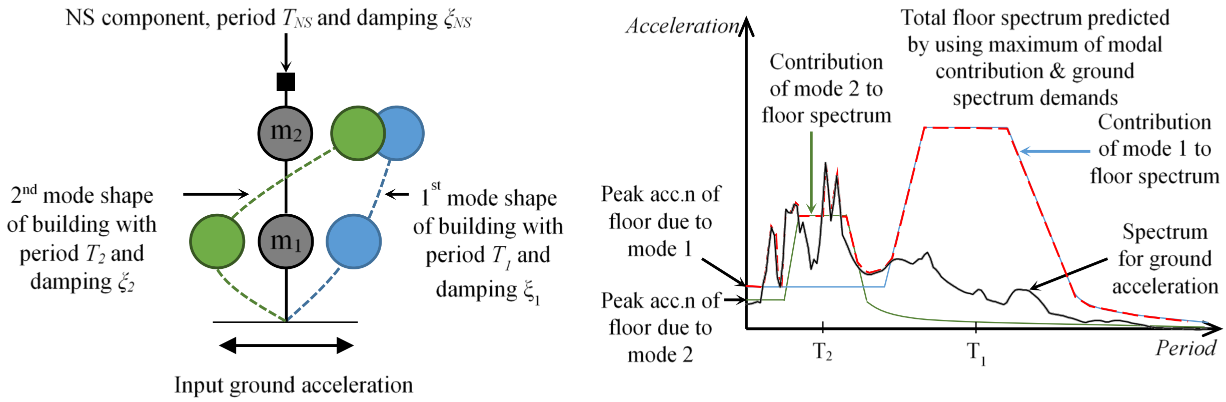

Figure 1: Overview of the proposed method showing mode shapes of a building (left) and a predicted floor acceleration response spectrum (right).

2.1 Modal Contribution Equation

The contribution of mode i, at floor j, to the predicted floor acceleration response spectrum at the period of vibration of the nonstructural component, T_{NS}, is computed using Equation 1:

$$S_{FA,i,j}(T_{NS},\xi_{NS}) = |\Gamma_i \phi_{i,j}| S_{GA}(T_i,\xi_{str}) DAF(r_{T,i}) \quad (1)$$

where Γ_i is the participation factor of mode i; $\phi_{i,j}$ is the coordinate of mode i, at floor j; $S_{GA}(T_i, \xi_{str})$ is the ground acceleration response spectral ordinate corresponding to the period of vibration of mode i, T_i, and the damping ratio of the structure, ξ_{str}, which is typically taken to be 5%; and $DAF(r_{T,i})$ is the dynamic amplification factor (DAF) used to describe the response spectrum amplification at the fundamental modal period of the nonstructural element, T_{NS}, due to building vibrations at mode i.

2.2 Dynamic Amplification Factor

The phenomenon that most governs the shape of the floor acceleration response spectrum is resonance between the modes of vibration of the structure and the period of vibration of the nonstructural component. Dynamic amplification is expected when the fundamental period of vibration of the nonstructural element, T_{NS}, is close to resonance with the period corresponding to any mode of the building, T_i. The Dynamic Amplification Factor, $DAF(r_{T,i})$, which quantifies this amplification, is modelled as a function of the ratio of the nonstructural component's period of vibration (noting that a nonstructural system may have multiple periods of vibration) to the period of vibration of mode i of the building, $r_{T,i}$, given by Equation 2.

$$r_{T,i} = \frac{T_{NS}}{T_i} \tag{2}$$

The Dynamic Amplification Factor, $DAF(r_{T,i})$ is modelled here as a piecewise function, as plotted in Figure 2. At low period ratios below $r_{T,A}$, representing stiff non-structural components, no dynamic amplification is expected and the floor spectrum is the same as the peak floor acceleration. At periods between $r_{T,B}$ and $r_{T,C}$, where the period of the nonstructural component is close to the period corresponding to the building mode, the dynamic amplification reaches its highest value. At periods beyond this peak, representing flexible nonstructural components, the dynamic amplification decays towards zero. The functional form used to model this decay was carefully chosen so as to closely represent the observed shape of floor displacement response spectra computed from the predicted floor acceleration spectra. The proposed equation to compute $DAF(r_{T,i})$ is given in Equation 3.

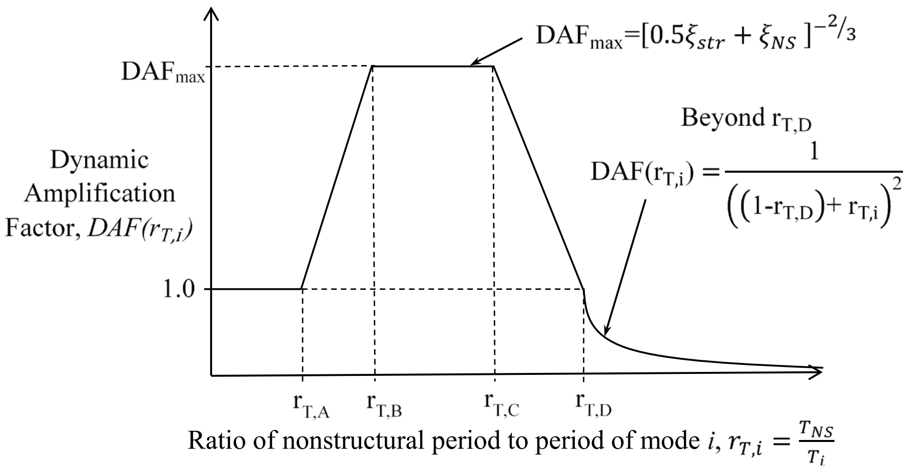

Figure 2: Dynamic Amplification Factor used to compute the contribution of a given mode of vibration of the building to the floor acceleration response spectrum.

$$DAF(r_{T,i}) = \begin{cases} 1 & r_{T,i} \le r_{T,A} \\ \frac{r_{T,i} - r_{T,A}}{r_{T,B} - r_{T,A}}(DAF_{max} - 1) + 1 & r_{T,A} < r_{T,i} < r_{T,B} \\ DAF_{max} & r_{T,B} \le r_{T,i} \le r_{T,C} \\ \frac{r_{T,i} - r_{T,C}}{r_{T,D} - r_{T,D}}(1 - DAF_{max}) + DAF_{max} & r_{T,C} < r_{T,i} < r_{T,D} \\ \frac{1}{[(1 - r_{T,D}) + r_{T,i}]^2} & r_{T,D} \le r_{T,i} \end{cases} \tag{3}$$

The peak dynamic amplification factor, represented by DAF_{max}, is computed as a function of the damping of the nonstructural component, ξ_{NS}, (hereafter called "nonstructural damping") to account for the increased amplification expected in components with low damping. The method proposed here uses the empirically-derived expression of Sullivan et al. (2013), shown in Equation 4.

$$DAF_{max} = [0.5\xi_{str} + \xi_{NS}]^{-2/3} \quad (4)$$

The values of $r_{T,A}$ to $r_{T,D}$ define the width of the DAF peak. These period ranges must be carefully chosen to balance the narrow amplification ranges typically observed in elastic floor acceleration response spectra and the uncertainty in assessing the building's modal periods. It is currently recommended that the values prescribed in ASCE 7-16 be used, where $r_{T,A} = 0.50$, $r_{T,B} = 0.75$, $r_{T,C} = 1.25$, and $r_{T,D} = 2.00$ but future research should consider alternatives.

2.3 Combination of Modal Contributions

The floor acceleration response spectrum at floor level j, $S_{FA,j}(T_{NS}, \xi_{NS})$, is computed as the maximum of the modal response spectrum contributions from all modes, $S_{FA,i,j}(T_{NS}, \xi_{NS})$, computed using Equation 1, and the ground acceleration response spectrum (GRS), $S_{GA}(T_{NS}, \xi_{NS})$, as given by Equation 5:

$$S_{FA,j}(T_{NS}, \xi_{NS}) = \max_i\left[S_{FA,i,j}(T_{NS}, \xi_{NS}),\ S_{GA}(T_{NS}, \xi_{NS})\right] \quad (5)$$

This is illustrated in Figure 1, where each of the modal contributions, the ground acceleration response spectrum, and the combined floor acceleration response spectrum are shown. Consideration of a minimum of three modes in each fundamental direction is recommended. The fourth and fifth modes should, however, be considered for buildings with long fundamental periods (American Society of Civil Engineers 2017; Kehoe and Hachem 2003). Since nonstructural components with periods lower than 0.06 s can usually be considered to be rigid (Kehoe 2014), building modes with periods below 0.06 s do not need to be considered.

The inclusion of the ground response spectrum is motivated by the presence of rigid body modes discussed in Pozzi and Der Kiureghian (2012). This study demonstrated that lower levels of the structure do not filter the ground acceleration response spectrum as upper levels do, thereby retaining the same response spectral shape as the ground. There is some evidence in the floor acceleration response spectra computed from the instrumented building records that even at upper levels the ground acceleration response spectrum demands are still present particularly for low excitation intensities, which may be due to rigid body motion of the building as proposed by Pozzi and Der Kiureghian (2012). For simplicity, the maximum of the ground acceleration response spectrum and the modal contributions are taken for any floor j in the building.

2.4 Ground Acceleration Response Spectra

Two ground acceleration response spectra are required to predict the floor acceleration response spectrum using the proposed method. The first is the design site-specific ground acceleration response spectrum at the damping ratio of the structure, $S_{GA}(T, \xi_{str})$, which is used to determine the peak floor acceleration in each mode used in Equation 1. The second is the design site-specific ground acceleration response spectrum at the damping ratio of nonstructural component, $S_{GA}(T_{NS}, \xi_{NS})$, used to compute the maxima in Equation 5. These spectra can be computed by scaling the standard 5%-damped design spectrum specified in design codes such as NZS1170.5 (Standards New Zealand 2016), by a factor, η. The computation of these spectra by scaling a standard design spectrum at 5% damping is illustrated in Figure 3.

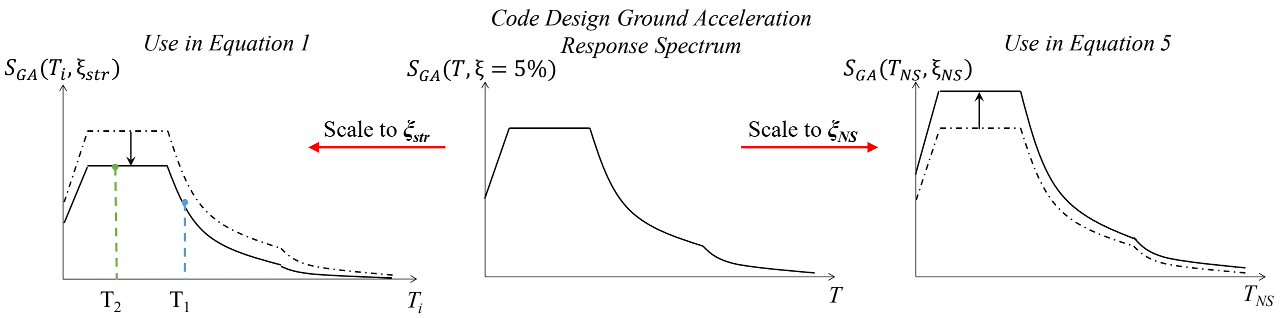

Figure 3: Ground acceleration response spectra scaled to structural and nonstructural damping for use in proposed floor acceleration response spectrum method.

Several empirical expressions for η exist, such as that given in Equation 6 from the guidelines for Seismic Assessment of Existing Buildings produced by NZSEE and MBIE (2017), where ξ is the nonstructural damping ratio of interest, given as a percentage.

$$\eta = \sqrt{\frac{7}{2+\xi}} \tag{6}$$

Although some empirical characterisation of nonstructural damping has been done at low intensities (Watkins et al. 2010), it can generally be difficult to assign a nonstructural damping value to components. Further research into component damping is recommended for greater confidence in the use of floor spectra for component design.

2.5 Conversion to Displacement and Velocity Spectra

Displacement demands on displacement-sensitive nonstructural components can also be computed using elastic floor response spectra. This study explicitly considers the computation of displacement response spectra from the predicted floor acceleration response spectra. At long periods, the floor displacement response spectrum can be approximated as the higher value between the peak floor displacement, calculated from the dominant first mode, and the ground displacement response spectrum. The ASCE DAF shape at T_{NS} exceeding $2.0T_i$ was modified to a decay shape which, when converted to a displacement response spectrum, corresponds to the peak floor displacement.

To convert from a floor acceleration response spectrum at floor j, given in absolute acceleration as $S_{FA,j}$, to a floor displacement response spectrum $S_{FD,j}$, given relative to the floor, Equation 7 is used.

$$S_{FD,j} = \frac{S_{FA,j}{T_{NS}}^2}{4\pi^2} \tag{7}$$

3 VALIDATION AND APPLICATION OF METHOD USING A CASE STUDY

The eight-storey University of Canterbury (UC) Physics building in Christchurch, New Zealand, is used for validation of the proposed method. It is one of three seismically-independent structures that comprise the West building, as seen in Figure 4. The lateral force resisting system in this direction is three reinforced concrete (RC) shear walls evenly spaced along the length of the building. Built in 1961, GeoNet accelerometers recorded the motions of the floors in the 2010/11 Canterbury earthquakes (GeoNet 2019). To assess the performance of the proposed method, the response at the roof in the transverse direction is examined for the M6.3 Lyttelton earthquake where it behaved approximately elastically.

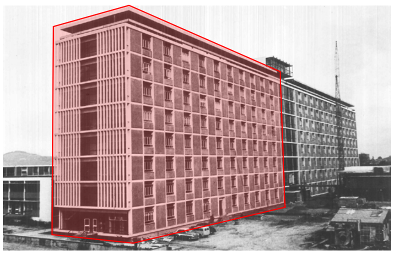

Figure 4: The UC West building with the Physics building highlighted (Reay 1970).

Modal properties should ideally be computed from an eigenvalue analysis when using the proposed method for design. Since this is an existing building though, mode shapes were computed using the average values for shear walls given in the prediction method described in Miranda and Taghavi (2005). The modal periods were inferred from the peaks in the floor acceleration response spectra to illustrate the efficacy of this prediction method when periods are well predicted. ξ_{str} is assumed to be 5%, and therefore, the ground acceleration response spectrum did not need to be scaled to compute the structural modal accelerations $S_{GA}(T_i, \xi_{str})$. These inputs, and the peak floor acceleration of each mode (PFA) are presented in Table 1.

Table 1: Summary of elastic properties

		Mode 1	**Mode 2**	**Mode 3**
	T_i (s)	0.63	0.18	0.10
	$\phi_{i,j}\Gamma_i$ at roof	1.431	-0.434	0.599
Lyttelton M6.3 EQ	$S_{GA}(T_i, \xi_{str})$ (g)	0.196	0.522	0.234
	PFA at roof (g)	0.280	0.227	0.140
Design SLS EQ*	$S_{GA}(T_i, \xi_{str})$ (g)	0.205	0.225	0.225
	PFA at roof (g)	0.294	0.098	0.135

*NZS1170.5 (Standards New Zealand 2016)

Floor acceleration and displacement response spectra were computed at ξ_{NS} of 2% and 5%. The ground acceleration and displacement response spectra, computed from the ground level recording at both ξ_{NS} values, are plotted in Figure 5. The predicted and recorded floor response spectra are compared in Figure 6. The provisions of seismic design codes which do not account for ξ_{NS} (Eurocode 8, NZS1170.5, and ASCE 7-16) are also included in the comparison.

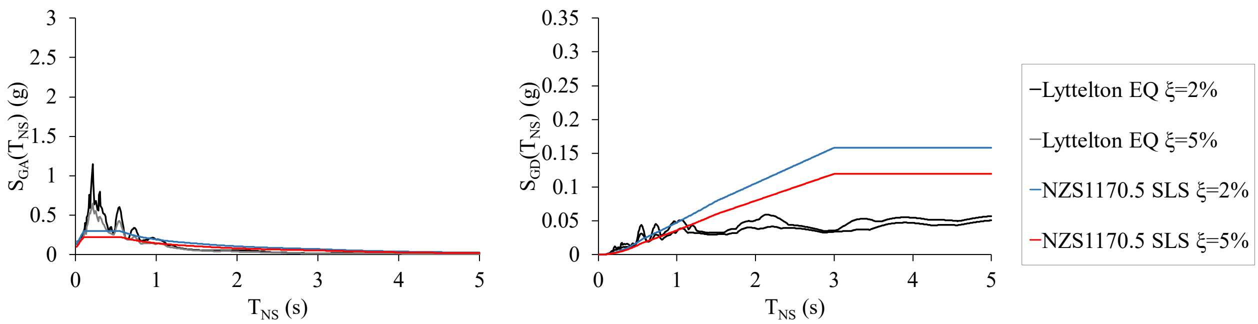

Figure 5: Acceleration (left) and displacement (right) ground response spectra at ξ_{NS}=2% and 5% for the Lyttelton earthquake and the SLS NZS1170.5 site-specific design spectra.

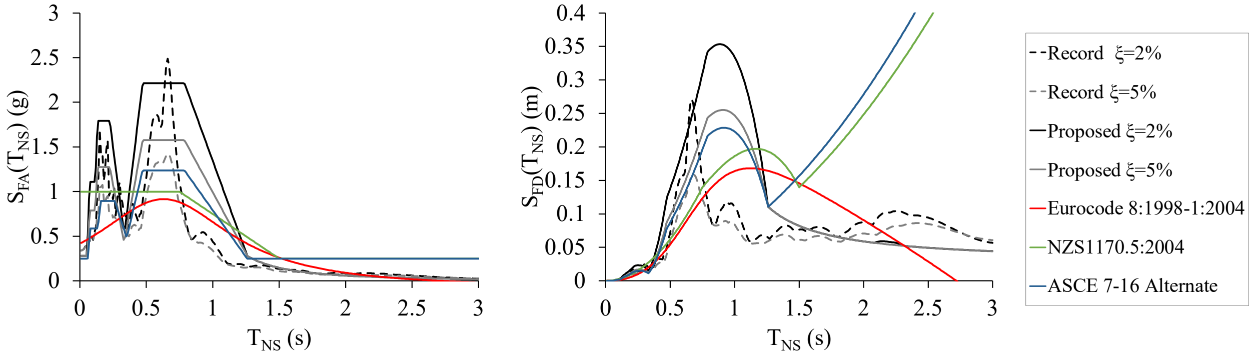

Figure 6: Acceleration (left) and displacement (right) floor response spectra for the UC Physics building Lyttelton 2011 earthquake record in the transverse direction at the roof at ξ_{NS}= 2% and 5%. Predictions are shown for the proposed method, NZS1170.5, Eurocode 8, and the ASCE 7-16 alternate method.

Figure 6 shows that the proposed method predicts the high amplification at the fundamental modal period more accurately than the code methods. The contributions of the higher modes at lower periods are captured by the proposed method, but neglected by Eurocode 8. The peak floor acceleration, $S_{FA,i,j}(T_{NS} = 0,\xi_{NS})$, is more accurately represented by the proposed method than the over-conservative NZS1170.5 approach. At longer periods, the displacement spectra computed from the code acceleration spectrum predictions do not follow expected physical behaviour, while the proposed method is able to provide more reasonable estimates due to the decay shape included in the DAF. The proposed method is further able to capture the increased dynamic amplification at lower values of nonstructural damping whereas the international code methods cannot.

To illustrate the use of the proposed prediction method for design, the approach was also applied to the design ground acceleration spectrum specified at the Serviceability Limit State (SLS) intensity prescribed in NZS1170.5. This was computed for Z = 0.30 for Christchurch with class D soil and scaled to 2% and 5% damping using Equation 6. The acceleration and displacement design ground response spectra are shown in Figure 5. The acceleration and displacement floor response spectra computed using the properties in Table 1 are presented in Figure 7. The floor displacement response spectrum decay curve at T_{NS} greater than the peak fundamental mode amplification region can be observed to change abruptly to the linearly increasing slope at approximately T_{NS} = 1.5 s and 1.8 s for ξ_{NS} = 2% and 5%, respectively. This corresponds to the design ground displacement response spectrum, as seen in Figure 5, which dominates these longer periods.

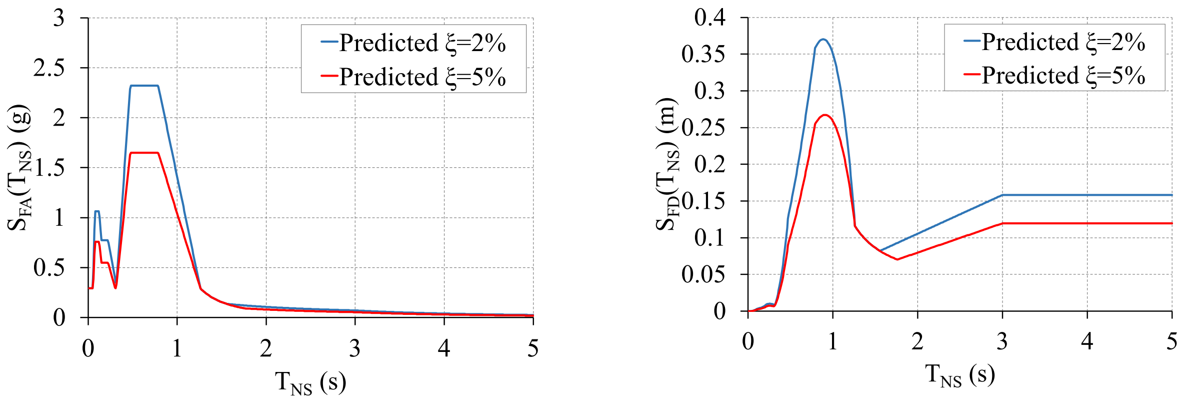

Figure 7: Acceleration (left) and displacement (right) floor response spectra for the UC Physics building using the prediction method with the SLS intensity design site-specific ground acceleration response spectrum in the transverse direction at the roof at 2% and 5% nonstructural damping.

4 UNCERTAINTY IN STRUCTURAL AND NONSTRUCTURAL VARIABLES

Accurate estimation of the fundamental period of vibration of the building, T_1, is important to obtain accurate predictions with any method that uses modal superposition to predict floor acceleration response spectra. There is often, however, significant uncertainty in estimating T_1. An attempt to account for this uncertainty has been made in the proposed prediction method by the increased width of the peak of the DAF function. Figure 6 illustrates this feature, where the peaks in the predicted floor acceleration response spectrum are broader than the recorded peaks.

This paper has not discussed how the period of vibration of nonstructural components may be estimated. Components themselves can be complex, have complex interactions with bracing and supports, and possess a number of periods of vibration. Dynamic tests of components are often limited to low intensity responses and only represent a small fraction of available components (Kehoe 2014). There is, therefore, uncertainty in characterising nonstructural damping and the nonstructural fundamental period. This should be considered when using the proposed method which prescribes floor acceleration response spectra with peaks and troughs over small period intervals. It is cautioned that precise definition of the component period to capture troughs between modes may result in unrealistic predictions of design accelerations due to this uncertainty. There is an opportunity for further work to refine this method to avoid these potential issues.

5 CONCLUSIONS

A simple method for predicting seismic demands on nonstructural components using a floor acceleration response spectrum has been outlined. Appropriate provisions are made for computing realistic floor displacement response spectra as well. The method was shown to work well if modal properties are well estimated, as validated using the motion recorded at the roof of the UC Physics building in the 2011 Lyttelton earthquake. The application of the method in design is demonstrated by computing design floor acceleration and displacement response spectra at the serviceability limit state prescribed by NZS1170.5.

It is cautioned that this method should be used with reasonable estimates of the modal properties of the structure and the period of the nonstructural component, since significant errors can be introduced by errors in these estimates. These errors can, however, be mitigated through the conservative shape and period ratio limits used to define the piecewise dynamic amplification factor (DAF). The application of this method is currently limited to nonstructural components and supporting structures responding in their elastic range. Further work to expand this method for higher intensity events is ongoing. This will enable its use at both Serviceability Limit State and Ultimate Limit State intensities.

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