16[th] World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 2632
Registration Code: S-O1463220621

ILLUSTRATING A NEW POSSIBILITY FOR THE ESTIMATION OF FLOOR SPECTRA IN NONLINEAR MULTI-DEGREE OF FREEDOM SYSTEMS

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Abstract

The adequate consideration of acceleration demands in the seismic design of acceleration-sensitive nonstructural components, although challenging, has shown to be a key area for improvement within the field of earthquake engineering. Moderate and strong seismic events in recent decades have reinforced the importance of these nonstructural systems as a large source of monetary loss, disruption and occupant hazard. However, numerous recent studies have shown that modern design codes and guidelines fall short of incorporating critical parameters necessary to estimate spectral floor acceleration demands. Further, recent trends in performance-based earthquake engineering have moved towards conveying seismic demands in a probabilistic manner; with the median seismic demand to be paired with an estimate of dispersion to allow better design decisions to be made in terms of building importance, occupancy and other performance objectives. This paper describes a novel procedure to combine important factors affecting spectral floor acceleration response in terms of both central tendency and uncertainty from ground motion variability (record-to-record dispersion) while maintaining a reasonable level of simplicity in implementation.

The study focuses on the influence of both structural and nonstructural damping ratios on the amplification of floor response spectra demands as well as the effect of nonlinear response in the primary structure. The fundamental frequencies and mode shapes are considered via a modal superposition method to incorporate modal characteristics of individual structures. The proposed technique for estimating floor response spectra targets the median floor spectra for given modal and damping characteristics as well as the expected ductility demand of the main structure for a given input spectrum. The consideration of nonlinear demands and damping ratios is incorporated through empirical relationships derived from linear single degree of freedom systems and nonlinear multiple degree of freedom (MDOF) systems, analyzed with a large suite of ground motions at various intensities. Distinctions are made in the procedure with respect to the type of structural system based on the preliminary findings of the study. The approach is applied to a reinforced concrete cantilever wall and a steel moment-resisting frame building as case studies. The proper consideration of higher mode response of the primary structure and nonstructural damping ratio are shown to have the greatest influence in central tendency across all periods. The good correlation between predicted results and those obtained via nonlinear time-history analyses suggests that the new procedure could represent a useful development for the assessment of nonstructural components within a performance-based earthquake engineering framework.

Keywords: non-structural elements; nonstructural components; floor acceleration spectra; nonlinear MDOF systems

1. Introduction

The damage of acceleration-sensitive nonstructural components (NSCs) in earthquakes can be a major (and even the largest) source of economic and social disruption from these events as shown by documentation spanning decades; including the 2010 Maule, Chile event [1]. The mitigation of the risk associated with NSCs is met with a three-faceted and on-going challenge, namely the need to: i) ascertain how structures containing these elements will affect them during seismic response (*e.g.* loading); ii) understand the seismic behavior of the elements themselves; and iii) implement improved (or intended) design and construction practice for NSCs. The first two points are represented by the on-going experimental and analytical studies conducted in order to increase the current level of knowledge of both structural and nonstructural behavior during seismic loading. The third point, however, is not so well defined. On the surface, this is would be assumed to be covered by building codes and standards, yet there are still numerous issues involved with the construction process that will need to evolve for improved design considerations to be effective [2].

Many of these issues revolve around the multi-disciplinary and fragmented nature of the construction process; where large levels of project integration would be required for "proper" consideration of seismic design of NSCs [3]. In addition to integration difficulties, traditional engineering roles are likely inadequate to handle increased workload when considering that procurement is undeniably controlled by cost-competiveness; where owners are likely uninformed about seismic risk and are mainly concerned with the bottom line. This is not a new concept and is, arguably, the basis for modern performance-based earthquake engineering (PBEE); where incentives for upgraded (more expensive) structural designs are typically justified through conveyance of consequences resulting from damaged NSCs. Recent proposals for enhanced project integration goals and advanced tools to achieve them can be found in the literature [2-4], yet any major changes to the construction industry related to NSCs may only occur slowly. In the interim, it is conceivable that PBEE concepts and methods will be needed in order to provide incentives to decision makers (even to achieve minimum design requirements [2]) and that any tools used to convey them should be simplistic in order to accommodate the likely lack of available resources.

From a structural engineering standpoint, the aforementioned difficulties in allocating and organizing resources for the seismic design of NSCs has been reflected by numerous simplifications in the definition of their loading. The earliest simplification was, arguably, the avoidance of direct analysis of NSCs using some form of floor acceleration response spectra (FRS); using the so-called cascade approach [3]. Notwithstanding the drastic reduction in complexity this offers compared to explicit modeling of the primary and secondary systems and the utility of the information provided, the method remains computationally demanding for the design of NSCs in typical built facilities [5]. Conversely, many recent studies (such as [6]) have shown that modern seismic codes for NSC demands lack the specificity to understand likely acceleration demands for explicit cases; information that could assist in the refined assessment of NSCs within a PBEE framework.

The current study aims to provide a means of estimating acceleration demands on NSCs through an approximation of the results obtained using the cascade approach to create FRS. The proposed approach has not yet been simplified for implementation within building codes, but could still be useful to practitioners interested in gaining a better estimate of FRS demands; serving as a liaison between computationally expensive methods and standardized values. The approach targets valuable feedback that can be updated based on the level of knowledge of both the primary structure and the nonstructural component.

2. Development of a Modal Superposition Approach for Floor Spectra Estimation

The proposed method uses the basis of previous research efforts ([6] to [8]) in order to estimate floor response spectra (FRS) for MDOF buildings responding in the nonlinear range. The development of the method focuses on two distinct aspects affecting spectral floor acceleration demands, namely: i) capturing the influence of damping ratio of both the primary structure and the nonstructural component on the elastic dynamic amplification (*e.g.* ratio of NSC acceleration to floor acceleration at the resonance condition of a coupled primary-secondary system responding elastically) and; ii) the influence of nonlinear building response on floor response spectra for MDOF buildings; distinguishing between different structural systems. The proposed approach relies on the concepts of modal superposition in order to adequately estimate spectral floor acceleration demands in MDOF buildings. As shown in Fig. 1, the main concepts behind the modal superposition approach are: estimating spectral acceleration demands; using modal properties to estimate modal floor accelerations; computing modal contributions to the FRS; and finally combining modal contributions for the estimated spectral floor acceleration (SFA) demands. The development of the proposed approach is outlined in the following subsections.

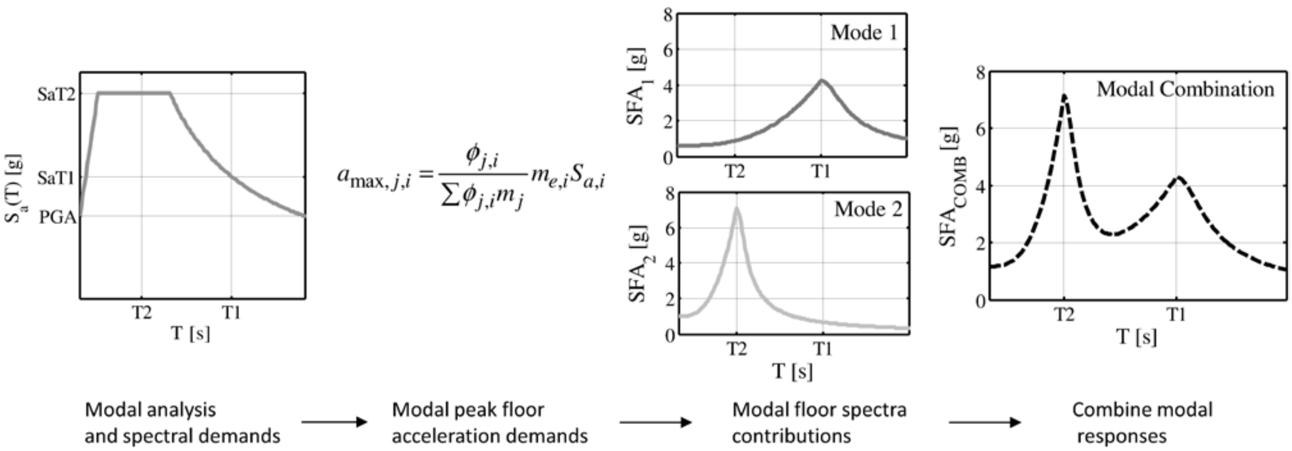
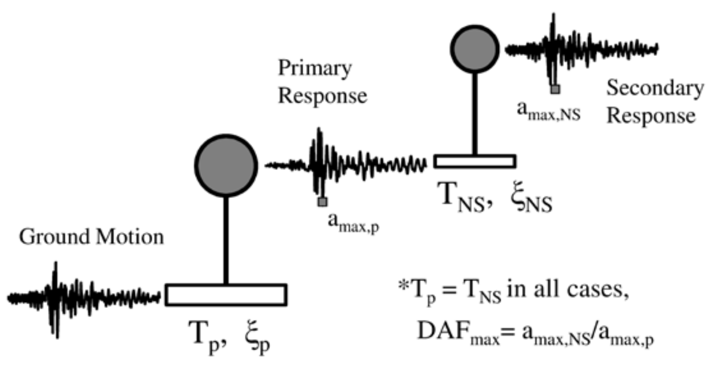


Fig. 1– Illustration of modal superposition for estimating floor response spectra

2.1 Dynamic Amplification of Coupled Primary-Secondary Elastic SDOF Systems

The investigation of estimating peak dynamic amplification (DAF_{max}) was conducted by considering a large number of elastic single degree of freedom (SDOF) systems at various damping levels. The concept of DAF_{max} for SDOF systems is illustrated along with the variations of primary (ξ_p) and nonstructural (ξ_{NS}) damping ratios and other information about the SDOF study in Fig. 2.



a)

Record Set: FEMA P695 far-field [9]

- 22 horizontal ground motion recordings
- 44 accelerograms total

Nonstructural Damping (ξ_{NS}): 0.5%, 2%, 5%, 10%

Primary Damping (ξ_p): 1%, 3%, 5%

Period Range ($T_p = T_{NS}$): 0.1s to 4.0s at 0.1s intervals

Total number of data points = 21120

b)

Fig. 2– Details of SDOF study for dynamic amplification: a) definition of DAF_{max}; b) Input parameters

The results of the SDOF study were used to define a relationship to quantify DAF_{max} across a range of periods, as well as a range of combinations of ξ_p and ξ_{NS}. Targeting the median amplification at each period, the equation development initially considered the functional form of $DAF_{max}=\xi_{NS}^{b}$ proposed by [6]. The functional form was extended to $DAF_{max}=(a\xi_p + \xi_{NS})^b$ to consider ξ_p as shown in Eq. (1). Similar to the findings of [7], a decrease in DAF_{max} was witnessed at periods below the initial corner period T_B defining the onset of the constant acceleration region. This reduction was incorporated using a simple linear reduction at periods below T_B (taken as 0.3s in this study) as shown in Eq. (2); where T is the spectral period of interest.

$$DAF_{\max} = (a\xi_p + \xi_{NS})^b = \left(0.47\xi_p + \xi_{NS}\right)^{-0.661} \approx \left(0.5\xi_p + \xi_{NS}\right)^{-0.667} \tag{1}$$

$$DAF_{\max,\mathrm{T<T_B}} = DAF_{\max}\left(0.55 + 0.45\left(\frac{T}{T_B}\right)\right) \tag{2}$$

A sample of the DAF_{max} expression is compared with dynamic analyses for a single combination of ξ_p and ξ_{NS} in Fig. 3a, while the comparison of the function for all damping ratios and periods $\geq T_B$ (refer Fig. 2b) is compared to the median of dynamic time-history results in Fig. 3b (see [10] for more information).

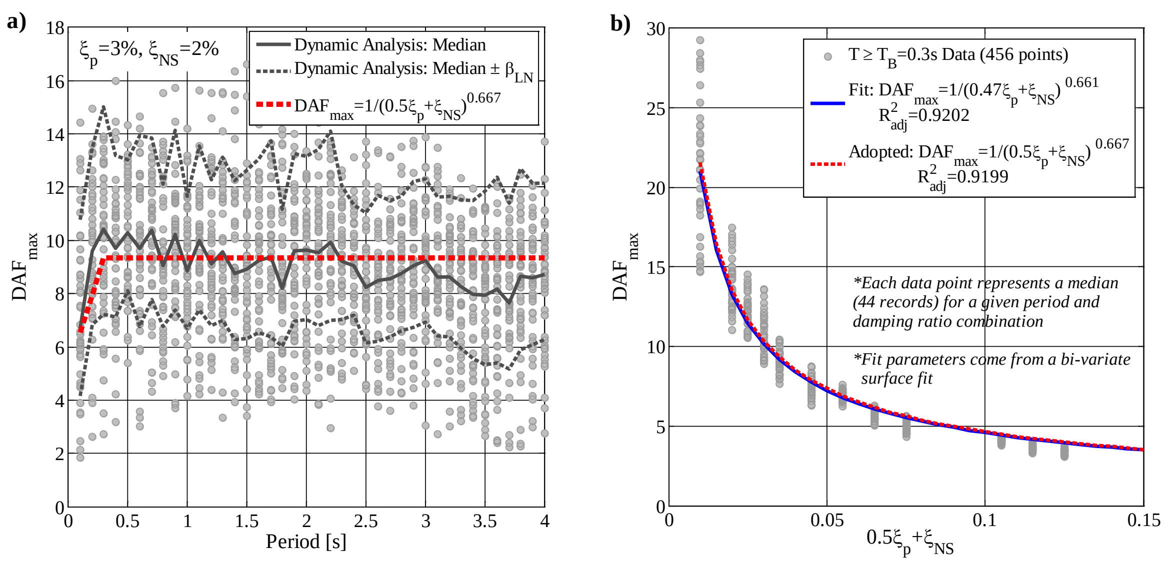


Fig. 3 – Adopted DAF_{max} function compared with median dynamic analysis results: a) Single damping ratio combination; b) All damping ratio combinations across all periods $\geq T_B$ = 0.3s, with best fit and adopted equations indicated

2.2 Incorporating Nonlinear Structural Response into the Modal Superposition Approach

The effect of nonlinear response on spectral floor acceleration (SFA) demands is not straightforward; with numerous variables involving the level of ductility demand, height of the building, damping ratios, modal correlations, hysteretic type of the primary structure and location along the height of the building, to name a few. Due to these complexities, a closed-form solution is not likely to be elegant; and certainly not practical for the design or assessment of nonstructural components that may already receive minimal time and resource allocation. Given this, the current study seeks to quantify the effects of nonlinear building response via empirical methods; neglecting numerous complexities with motives of viable implementation.

The work of [10] considered a set of six steel moment-resisting frame (MRF) buildings and three reinforced concrete (RC) cantilever wall buildings for the initial investigation; all consisting of 4, 8 or 12 stories. The same record set used for the SDOF study described in Section 2.1 [9] was applied at various intensity levels (median PGA of 0.15g to 1.2g at 0.15g intervals). One novel aspect of the approach was that each individual record was monitored for system displacement ductility demand μ in order to perform regression analysis to capture the effects of nonlinear response on SFA demands. The ductility demands were estimated using displacement ductility at the effective height for wall buildings and using a work-based approach (*i.e.* to combine story level demands) in combination with a recently proposed relationship for estimating the yield drift of steel MRF buildings [11]. More details of select case study buildings are provided in Section 3 (refer to [10] for complete information).

Dynamic time-history analyses were conducted for each case study building considering elastic and nonlinear analysis. Floor acceleration response spectra were computed at quarter points up the height of the building at nonstructural damping ratios ξ_{NS} of 0.5%, 2%, 5% and 10% of critical. The roof spectra results were monitored at the first three fundamental periods of the case study buildings. The concept of nonlinear FRS reduction factors was introduced for MDOF systems, defined as the ratio of the floor spectral ordinate from elastic analysis to the corresponding nonlinear case. An illustration of the nonlinear reduction definition is shown in Fig. 4a. Values were adjusted to 1.0 for an intensity that produced a μ estimate closest to unity (actual range was 0.95 to 1.1) in order to eliminate the large scatter in DAF_{max} demands for individual records (refer Fig. 3a). These adjusted results were used for regression analysis for μ ranging from 1.0 to 5.0 using the form $R_i = \mu^{\alpha}$; where R_i is the reduction at mode i and α is the regression variable. Sample regression fits for the first two modes of each type of building with ξ_{NS} of 2% are shown in Fig. 4b.

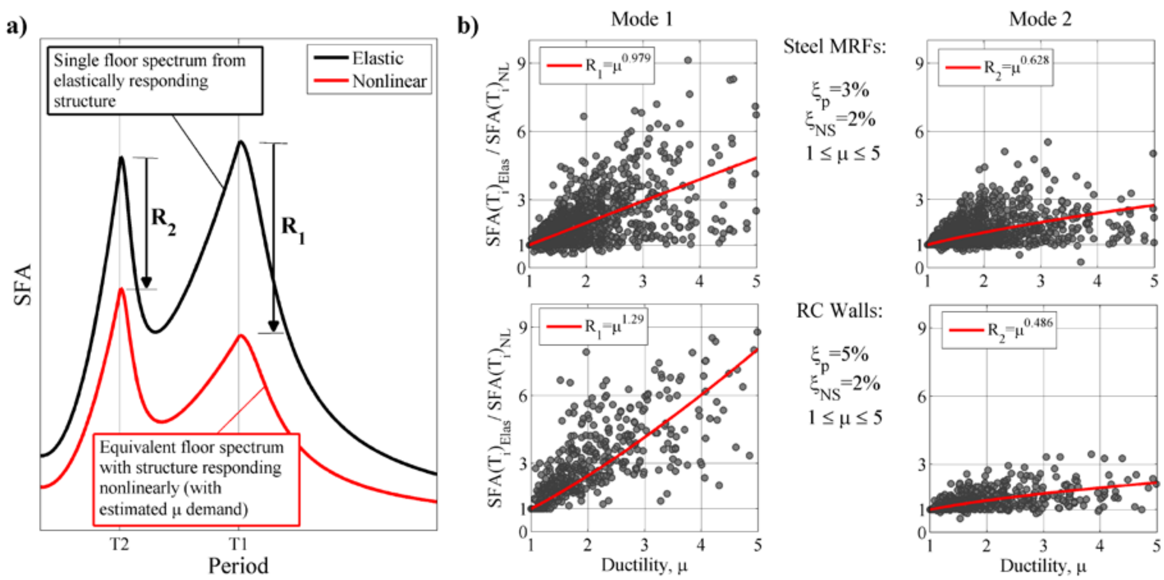


Fig. 4 – a) Definition of floor response spectrum (FRS) nonlinear reduction factors; b) Regression results for steel MRF buildings (above) and RC walls (below) for the first two modes at 2% nonstructural damping

The results of regression analysis with all considered values of ξ_{NS} showed a larger reduction of FRS demands with decreasing nonstructural damping. As result of this, the behavior witnessed at ξ_{NS} of 2% and 5% was given a higher weighting when proposing input parameters for the method; with slight conservatism implied for very low nonstructural damping values. The reduction at the fundamental period assumes a reduction proportional to the expected ductility demand μ at the intensity of interest for steel MRF buildings (*i.e.* MDOF frames with bilinear hysteresis). This is deemed reasonable in order to simply estimate ductility demand using a variety of methods, including the relative intensity that relates the first mode yield acceleration $S_{a,1y}$ to the elastic first mode elastic demand $S_a(T_1)$ (equal displacement rule) that may be approximated with very little information known about the structure. For RC cantilever walls, typically attributed a stiffness degrading model (*e.g.* Takeda or Q-model) an additional reduction is incorporated based on the findings of [10] and [12], noting that this additional reduction was also found to be sensitive to the level of ξ_{NS}; with more reduction as ξ_{NS} decreases. The reduction factors at the fundamental period R_1 are shown in Eq. (3):

$$R_1 = R_{i=1} \approx \mu \vee \frac{S_a(T_1)}{S_{a,1y}} \quad SteelMRF \quad ; \quad R_1 = R_{i=1} \approx \mu^{1.25} \vee \left(\frac{S_a(T_1)}{S_{a,1y}}\right)^{1.25} \quad RCWall \tag{3}$$

where $\vee$ signifies "or" and $S_a(T_1)/S_{a,1y}$ is the relative intensity. The reduction of higher modes uses a single reduction factor R_{HM} for all modes i >1 and only distinguishes between structural type as shown in Eq. (4).

$$R_{HM} = R_{i>1} \approx \mu^{0.6} \vee \left(\frac{S_a(T_1)}{S_{a,1y}}\right)^{0.6} \quad SteelMRF \quad ; \quad R_{HM} = R_{i>1} \approx \mu^{0.4} \vee \left(\frac{S_a(T_1)}{S_{a,1y}}\right)^{0.4} \quad RCWall \tag{4}$$

The use of a single reduction factor for higher modes is deemed sufficiently accurate based on witnessed performance of the method for a limited number of case study investigations. More complete information regarding variations of the use of higher mode reduction can be found in [10]. One additional consideration in accounting for nonlinear demands is in terms of effective periods. For steel MRFs, the inclusion of effective periods is neglected; supported by the findings of [10] and other studies [13]. Conversely, the inclusion of effective periods for RC walls considers the elongation of both the first $T_{1,eff}$ and second $T_{2,eff}$ mode periods (assuming a 2D planar structure) according to Eqs. (5) and (6):

$$T_{1,eff} = T_1 \sqrt{\frac{\mu}{\left(1 + r\left(\mu - 1\right)\right)}} \tag{5}$$

$$T_{2,eff} = T_2 \left(1 + 0.5\left(\frac{\mu}{\mu_{pin}}\right)\right) \quad for \quad 1.0 < \mu \le \mu_{pin} \tag{6}$$

where T_i is the elastic period, μ is the expected ductility demand and r is the post-yield hardening factor (if applicable). The term in Eq. (5) is a relatively standard form (amongst many available) to estimate $T_{1,eff}$. However, the relationship in Eq. (6) for $T_{2,eff}$ relies on an approximation using continuum beam mechanics where the maximum second mode period elongation is related to that of a fully pinned continuous cantilever which is assumed to be reached at a ductility of μ_{pin} (5.0 for cantilever walls) based on the previous work of [14]. Notably, Eq. (6) does not elongate the second mode for ductility demand less than 1.0 and the maximum elongation ($T_{2,eff}$ = 1.5T_2) is returned for any values of μ greater than μ_{pin}.

2.3 Combining Modal Responses

In order to produce floor response spectra estimates, the results from the previous subsections must be incorporated within a modal superposition approach. Using modal information, elastic spectral demands and an estimate of ductility-based reduction factors (see Eqs. 3 and 4), the peak modal floor accelerations $a_{max,j,i}$ of each mode i at every floor j can be found according to Eq. (6):

$$a_{\max,j,i} = \frac{\phi_{j,i}}{\sum \phi_{j,i} m_j} m_{e,i} \cdot \left(\frac{S_a\left(T_i\right)}{R_i}\right) = \phi_{j,i}\Gamma_i \cdot \left(\frac{S_a\left(T_i\right)}{R_i}\right) \quad ; \quad R_i \ge 1 \tag{6}$$

where $\phi_{j,i}$ is the mode shape of mode i at floor j, $m_{e,i}$ is the effective modal mass of mode i and m_j is the story mass at floor j (Note Γ_i is the modal participation factor of mode i). The terms $S_a(T_i)$ and R_i are the modal elastic spectral accelerations and FRS reduction factors, respectively. Taking the peak modal floor acceleration as an anchor point (*i.e.* at T_i = 0s), the modal floor spectrum contribution can be constructed using the three terms of Eq. (7); representing pre-, at- and post-resonance regions:

$$\begin{aligned} a_{m,j,i}\left(T < T_i\right) &= \left(T/T_i\right)^2 \left[a_{\max,j,i}\left(DAF_{\max} - 1\right)\right] + a_{\max,j,i} \\ a_{m,j,i}\left(T_i \le T \le T_{i,eff}\right) &= a_{\max,j,i} DAF_{\max} \\ a_{m,j,i}\left(T > T_{i,eff}\right) &= a_{\max,j,i} DAF = a_{\max,j,i}\left[\left(1 - \frac{T}{T_{i,eff}}\right)^2 + \left(0.5\xi_p + \xi_{NS}\right)\right]^{-0.667} \end{aligned} \tag{7}$$

where $a_{m,j,i}(T)$ is the modal spectral acceleration of mode i for floor j. T is the FRS period of interest, while T_i and $T_{i,eff}$ are the elastic and effective periods (if applicable) of mode i, respectively. The term DAF_{max} is calculated with Eqs. (1) or (2) depending on the location of T_i with respect to T_B and the DAF term in the third part of Eq. (7) must also be scaled by 0.55+0.45(T_i/T_B) when $T_i<T_B$. Once all modal contributions are estimated for a given floor j, the total spectral floor acceleration (SFA) can be obtained by taking the square-root-sum-of-the-squares (SRSS) of the modal contributions across all periods of interest according to Eq. (8):

$$SFA_j\left(T\right)_{SRSS} = \sqrt{\sum_{i=1}^{nm} a_{m,j,i}\left(T\right)^2} \tag{8}$$

where *nm* is the number of modes considered. It is proposed based on the work of [10] that three modes is sufficient for RC wall buildings and that steel MRF buildings should include the lesser of the number of stories and four. Notably, the use of SRSS is subject to limitations, including a lack of consideration of rigid mode response near the base level of the building. To overcome this, the work of [7] suggested taking the envelope of $S_{a,GM}$ (median ground motion spectral acceleration at ξ_p) and the result of Eq. (8) for floor levels less than H_i/H_n = 0.5; with H_i/H_n representing the normalized elevation of level *i* with respect to roof level. A restriction of this approximation is adopted for the current method, with the use of $S_{a,GM}$ only when $\xi_p \geq \xi_{NS}$; based on error analysis conducted within [10]. The final combination relationship is shown in Eq. (9).

$$SFA_j\left(T\right) = \begin{cases} \max\left(SFA_j\left(T\right)_{SRSS}, S_{a,GM}\left(T,\xi_p\right)\right) & for \quad H_i/H_n < 0.5, \quad \xi_p \geq \xi_{NS} \\ SFA_j\left(T\right)_{SRSS} & for \quad H_i/H_n \geq 0.5, \quad \xi_p \geq \xi_{NS} \end{cases}$$
$$SFA_j\left(T\right) = SFA_j\left(T\right)_{SRSS} \quad for \quad \xi_p < \xi_{NS} \tag{9}$$

3. Comparison of the Proposed Approach with Median NLTH Analysis

The case study buildings selected for illustration of the proposed method consist of a steel MRF building and a RC cantilever wall building; each having 8 stories. The steel MRF has a first story height of 4.5m with the remaining stories at 3.5m; the RC wall building has a constant story height of 3m. Planar frame models are used within the program Ruaumoko [15]. The steel MRF is modeled as a three bay (at 7.0m) lumped plasticity frame assuming expected material properties and a bilinear *M-φ* hysteresis with 2% strain hardening. The wall building is an equivalent cantilever representing 4 identical walls that make up the lateral force resisting system in the direction under consideration. The equivalent wall model assumes cracked elastic properties according to the yield curvature and has a single plastic hinge at the base exhibiting Takeda-thin behavior with 0.5% strain hardening. The models assume a Tangent stiffness proportional Rayleigh damping model updated according to the secant stiffness with the first two modes assigned 3% and 5% damping for the steel MRF and RC wall, respectively. A brief summary of the two selected buildings is provided in Table 1. Complete information for all case study buildings can be found within [10]. The first mode yield acceleration $S_{a,1y}$ was estimated from a force-based pushover analysis according to the first mode shape.

Table 1 – Brief summary of properties of 8 story building models selected for illustration

Type	PGA_{Design} [g][*]	Elastic Modal Periods			Effective Modal Mass (% of total)			$S_{a,1y}$ [g][†]
		T_1 [s]	T_2 [s]	T_3 [s]	$m_{e,1}$ [kNs^2/m]	$m_{e,2}$[kNs^2/m]	$m_{e,3}$ [kNs^2/m]	
Steel MRF	0.46	1.94	0.67	0.38	1167 (84%)	143 (10%)	44 (3%)	0.246
RC Wall	0.29	1.65	0.27	0.1	3146 (66%)	981 (20%)	340 (7%)	0.215

[*]Eurocode 8 Type I spectrum adjusted for soil C [†]First mode yield acceleration

The approach outlined in Section 2 was applied to the two case study buildings assuming ξ_p as 3% for the steel MRF and 5% for the RC wall. Since the same time history analyses used for calibration of the method are used for comparison, the implementations presented in this section are the median results from individual record estimates of floor spectra (*i.e.* individual $S_a(T_i)$ and μ estimates). This was assumed to be a more rigorous test of the method; use of the median spectral demands and ductility estimates would be a more realistic implementation. A sample of results at the highest intensity considered and for 2% ξ_{NS} is given in Fig. 5 for quarter points up the height of the building. The figure shows very good results along the entire height despite the significant variations in spectral floor acceleration (SFA) from lower to upper stories.

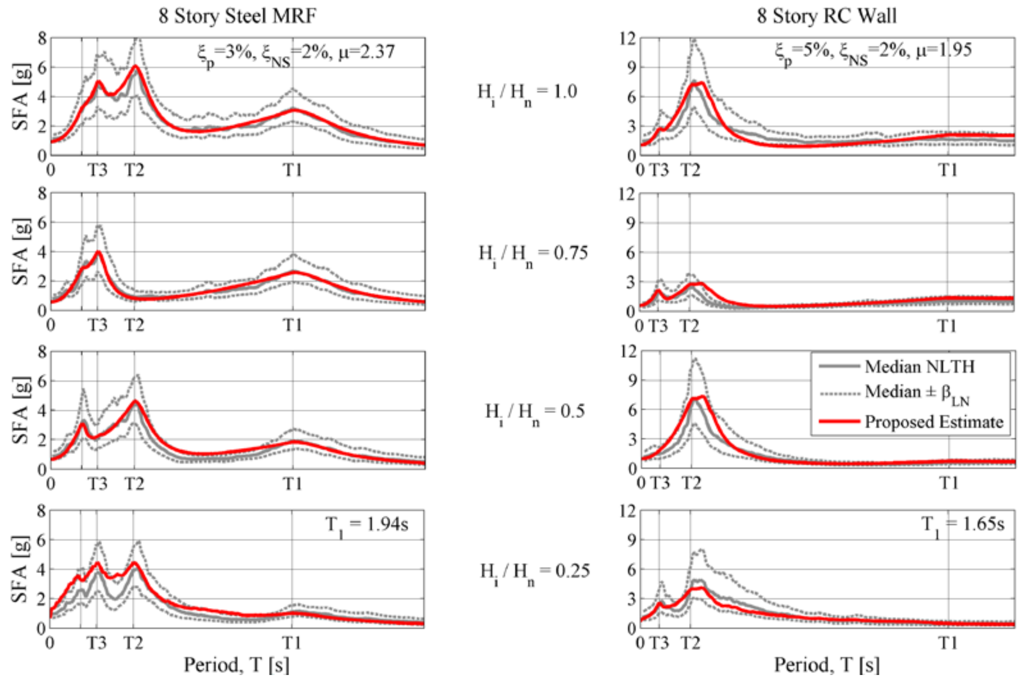


Fig. 5 – Comparison of proposed method with NLTH analysis at quarter points along the elevation with 2% nonstructural damping: 8 story steel MRF (left), 8 story RC wall (right). Median ductility demands are annotated

The ability for the method to account for the effect of ξ_{NS} is shown in Fig. 6 with the roof spectra at the highest intensity considered for both case study buildings. The results are also compared with standard nonstructural component demands for three different seismic codes from around the world [16-18]; noting that all importance factors and component reduction factors were set to 1.0. Fig. 6 clearly shows the importance of ξ_{NS} for the peak floor spectra response at the natural modes of the structure; in particular the higher mode region. Further, the proposed approach can give valuable feedback when comparing to existing code equations that do not consider the structural and nonstructural systems explicitly. The roof spectra results of the proposed method across a range of ductility demands at 5% ξ_{NS} are presented in Fig. 7. The figure illustrates that the method can adequately capture the effects of both increasing intensity and nonlinear demands; including the elastic range of response. The results presented illustrate that the proposed method can give similar information on acceleration demands as the floor response spectra created using more computationally expensive analysis methods.

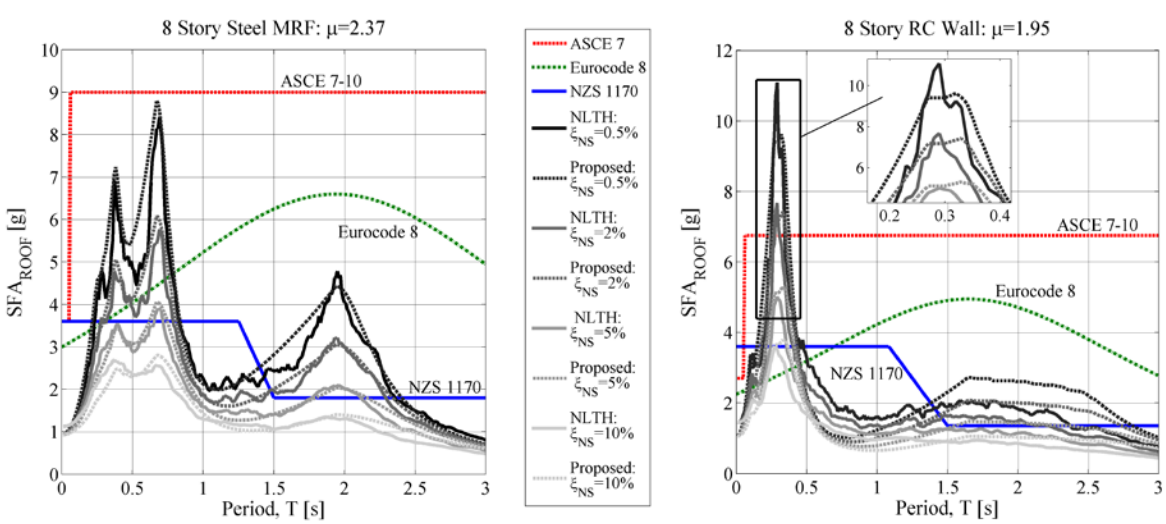


Fig. 6 – Comparison of proposed method with NLTH analysis at roof level for various values of nonstructural damping: 8 story steel MRF (left), 8 story RC wall (right). Current code approximations are annotated

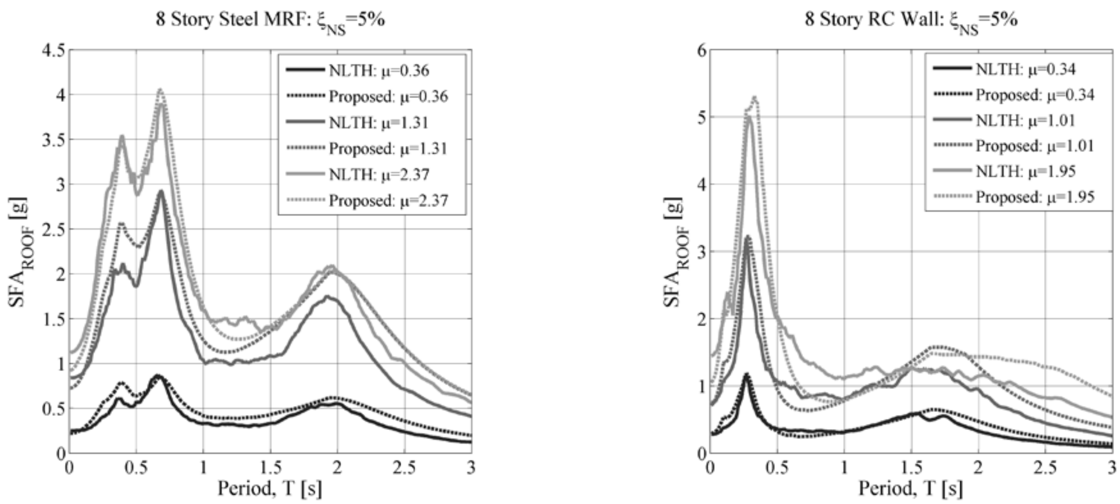


Fig. 7 – Comparison of proposed method with NLTH analysis at varying levels of ductility demand with 5% nonstructural damping: 8 story steel MRF (left), 8 story RC wall (right)

4. Record to Record Uncertainty in the Estimation of Floor Response Spectra

The incorporation of record to record uncertainty within performance-based earthquake engineering (PBEE) approaches commonly utilizes the lognormal standard deviation, or dispersion, β_{LN} of ground motion parameters and structural responses in order to define the aleatory uncertainty associated with variations in actual seismic loading. Further, the work of [19] has pointed out that variation in floor spectra demands are reasonably represented by a lognormal distribution. The current study proposes that the record to record uncertainty may be incorporated into the proposed method by simply using the uncertainty in the ground motion acceleration spectra of interest β_{GM}. This is a rather obvious starting point, yet it is not without justification. The results of Fig. 8 show the floor spectra responses at different ductility demands for the steel MRF building with the comparison of corresponding lognormal dispersion and that of the input ground motion (dashed red line). The figure shows relatively consistent dispersion values and that nonlinear response decreases dispersion near the fundamental periods; also supported by the findings of [19]. Similar results are witnessed for the RC wall building shown in Fig. 9; with the main exception being a larger dispersion in elastic response near the fundamental period.

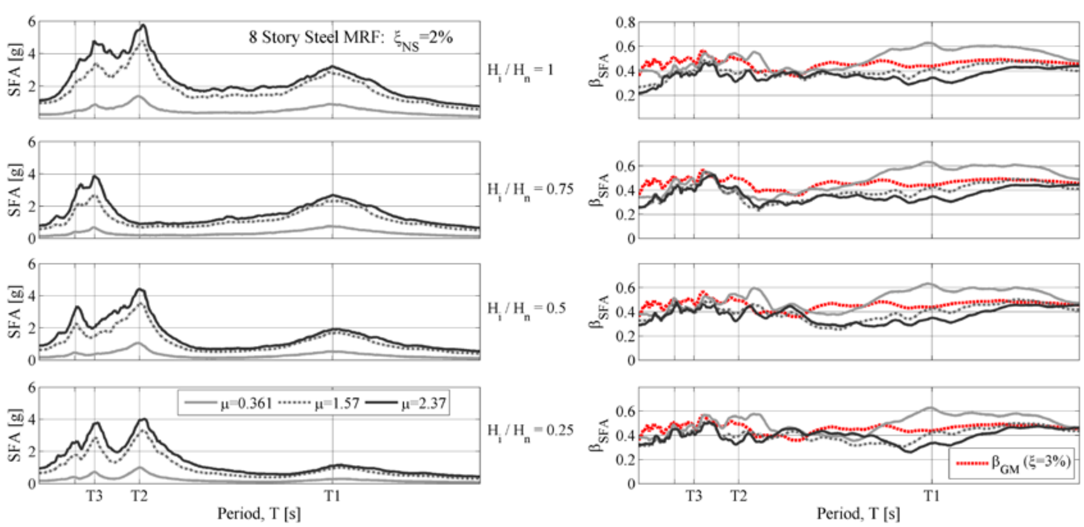


Fig. 8 – Median floor spectra from NLTH analysis for the 8 story steel MRF at various ductility demands 2% ξ_{NS} (left), corresponding lognormal dispersion compared with that of input ground motions (right)

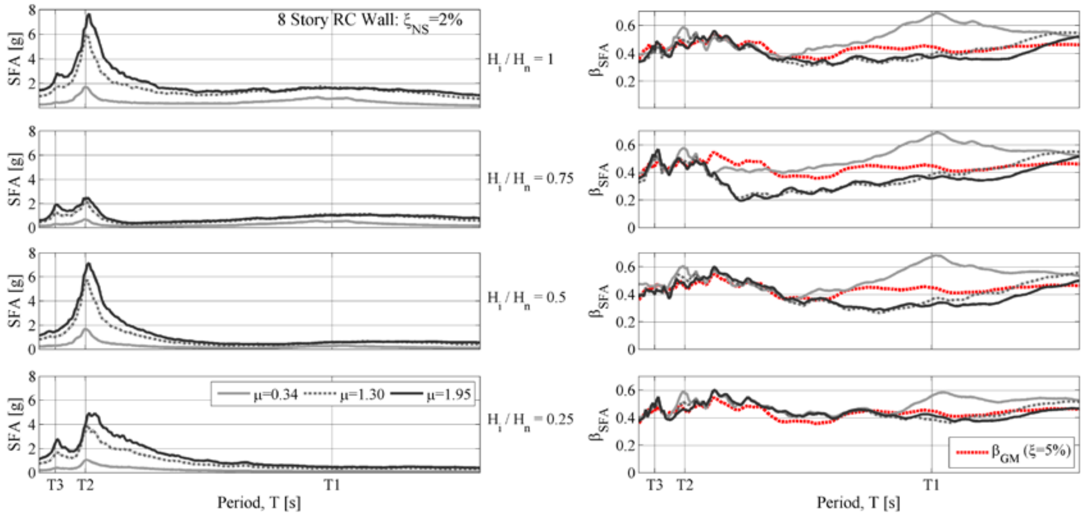


Fig. 9 – Median floor spectra from NLTH analysis for the 8 story RC wall at various ductility demands 2% ξ_{NS} (left), corresponding lognormal dispersion compared with that of input ground motions (right)

An example implementation is illustrated for the 8 story steel MRF building in Fig. 10. The estimate results utilize the median ground motion spectra and the ductility demand estimated by the relative intensity; representing a more realistic implementation of the procedure. Fig. 10 illustrates a very good agreement with NLTH analyses both in terms of dispersion and central tendency. A similar implementation is presented for the RC wall building in Fig. 11. It must be noted that as the difference between ξ_p and ξ_{NS} increases, the applicability of using the ground motion dispersion is likely to reduce. However, the extent of these implications is part of ongoing research.

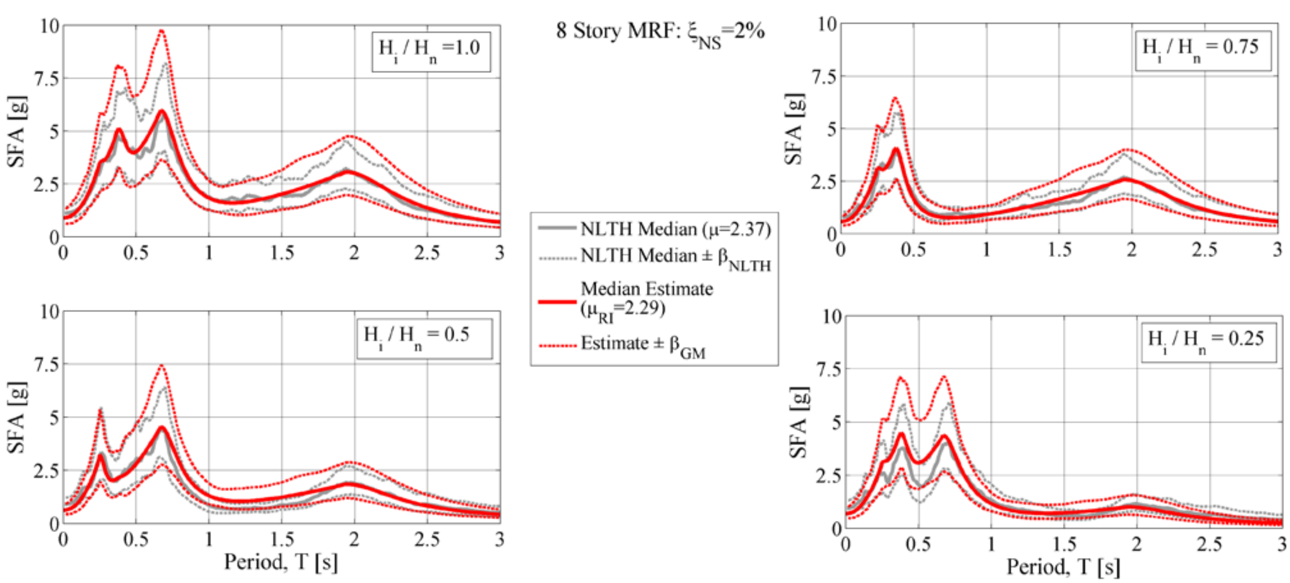


Fig. 10 – Estimates of median and dispersion compared with NLTH analysis using the dispersion of the input ground motion for the 8 story steel MRF building at 2% nonstructural damping

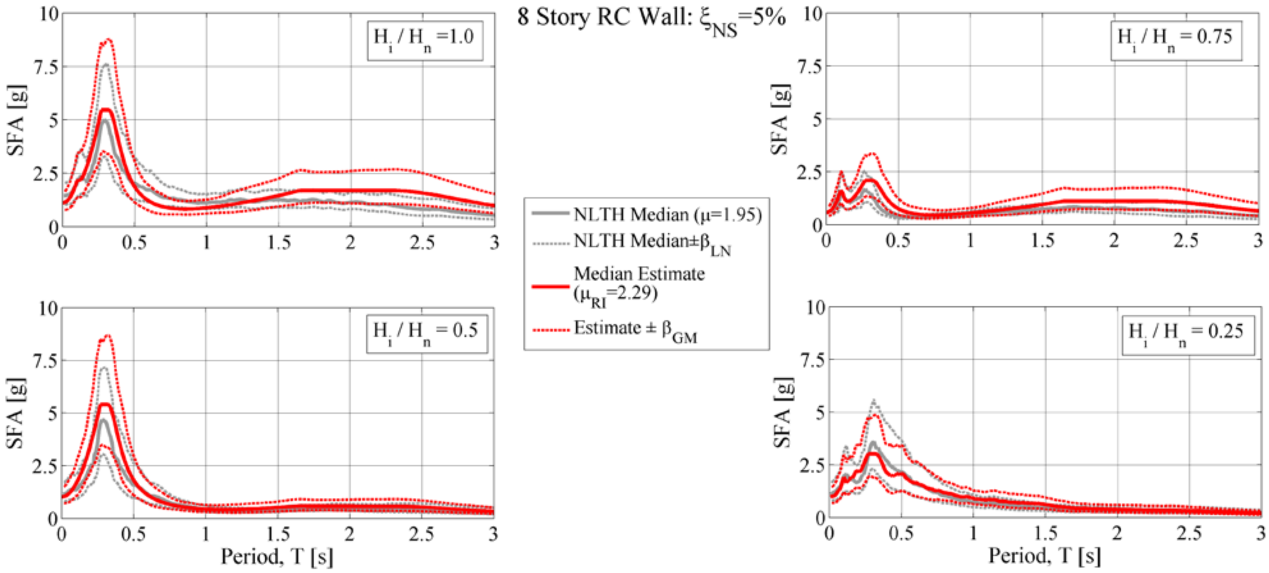


Fig. 11 – Estimates of dispersion compared with NLTH analysis using the dispersion of the input ground motion for the 8 story RC wall building at 5% nonstructural damping

One other important source of uncertainty in the estimation of floor spectra demands is the uncertainty in modal properties. Given that information on floor spectra demands can be sought, conceivably, at any part of the design process, the amount of available information about the building could vary drastically. Further, as typical structural models include only primary structural elements (*i.e.* bare frame models), the consideration of uncertainty in dynamic properties (*i.e.* periods) has relevance even if an explicit structural model is available.

Previous work by [20] has shown that the modal properties of the first three modes of planar frame or wall structures can be reasonably estimated provided that an estimate of the fundamental period is made. Further, numerous studies have proposed empirical relationships to estimate the fundamental period of structures based on actual measurements; one such set of relationships are those of [21]. For these reasons, investigation into the uncertainty in modal properties should be the subject of ongoing research, yet it is deemed important to address that existing resources show promise for implementing the current procedure under circumstances of widely varying knowledge about the structure under question.

5. Concluding Remarks

A procedure to estimate floor acceleration response spectra of MDOF buildings was presented. The method shows good results with the ability to incorporate different levels of knowledge about the primary and nonstructural systems including: the type of structural system, the modal properties and level of expected ductility demand in the primary structure, the damping ratios of both the structural and nonstructural systems, and the location of the nonstructural system within the building. The procedure was also shown to be able to reproduce reasonable dispersion bounds using the uncertainty in a target record set.

Considering the reduced level of effort compared to explicit calculation of floor response spectra using the cascade approach, the current procedure can give similar feedback with comparatively little effort. Although not intended as a direct replacement for code oriented equations, the proposed procedure can allow for a better understanding of likely acceleration demands on nonstructural elements; where current code equations may provide loading estimates that could be highly conservative or non-conservative depending on the situation.

The current method relies on fundamentals of structural dynamics, yet is also largely based on empirical relationships and simplifying assumptions to aid implementation. As with any empirically based method, the current procedure is still limited in terms of validation and investigative efforts to define bounds of applicability. Future work is certainly merited, yet the research efforts presented herein are a reasonable starting point and clearly show that the proposed method is a solid framework as a new option for estimating floor spectra.

6. Acknowledgements

The funding provided by IUSS Pavia as part of the first author's PhD scholarship is gratefully acknowledged.

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