# STATISTICAL MODELS FOR A PROPOSED ACCELERATION-RESPONSE MODIFICATION FACTOR FOR NONSTRUCTURAL COMPONENTS ATTACHED TO INELASTIC STRUCTURES

**Ragunath Sankaranarayanan**[1] **and Ricardo A. Medina**[2]

[1] *Structural Engineer, AREVA NP Inc., 7207 IBM Drive, Charlotte, NC 28262, USA*
[2] *Assistant Professor, Dept. of Civil Engineering, University of New Hampshire, Durham, NH, USA*
*Email: ragu.sankara@areva.com, ricardo.a.medina@unh.edu*

## ABSTRACT :

This paper summarizes ongoing research efforts to quantify a proposed acceleration-response modification factor ($R_{acc}$) useful to estimate the seismic demands imposed on acceleration-sensitive nonstructural components attached to inelastic multistory structures. The proposed $R_{acc}$ factor can be used similar to response modification factors for primary structures to scale elastic Floor Response Spectra (FRS) to obtain estimates of inelastic FRS. In this context, the terms ‘elastic’ and ‘inelastic’ refer to the primary structure, for only linear elastic components are addressed in this paper. The $R_{acc}$ factor is a function of the location of the component, its damping ratio and period of vibration, as well as the fundamental period of vibration and the level of inelasticity of the primary structure. In some cases, inelasticity in the primary structure amplifies the acceleration demands corresponding to elastic primary structures ($R_{acc} < 1$), particularly for components located at the bottom floors with periods smaller than half the fundamental period of the primary structure. This amplification of peak component acceleration demands depends strongly on the distribution of inelastic behavior along the height of the structure. However, in most cases, $R_{acc}$ is greater than one, particularly when the period of the component coincides with the modal periods of the primary structure. Statistical models to estimate $R_{acc}$ values are proposed and evaluated in this paper based on inelastic dynamic analyses with multistory structures exposed to 40 ground motions.

**KEYWORDS:** Nonstructural components, acceleration response modification factor, peak acceleration demands, floor response spectra, multistory structures, acceleration-sensitive components.

## 1. INTRODUCTION

Current U.S. building code requirements for the seismic design of NonStructural Components (NSCs) are based on the 2003 NEHRP Provisions (BSSC, 2003). These design guidelines are supported primarily by past experience, intuition, and engineering judgment rather than by experimental and analytical results (Filiatrault et al., 2004). In recent times, many researchers have attempted to characterize the seismic behavior of NSCs and their attachments both analytically and experimentally so that effective mitigation measures can be developed. For example, the NSF-sponsored Network for Earthquake Engineering Simulation Research (NEESR) program has awarded two different projects: *NEESR-GC: Simulation of the Seismic Performance of Nonstructural Systems and NEESR-SG: Experimental Determination of Performance of Drift-Sensitive Nonstructural Systems Under Seismic Loading* to evaluate and quantify the response of nonstructural components and systems. In addition, testing protocols have been recently developed to determine the seismic performance characteristics of NSCs (ATC, 2007). Other studies have dealt with evaluating the seismic performance of NSCs and quantifying their fragilities (Filiatrault et al., 2004; Retamales et al., 2006). The majority of past research focused on establishing elastic Floor Response Spectra (FRS). Only isolated studies have been conducted with inelastic supporting structures. Some of these studies used a similar approach to the one utilized in this paper, e.g., Kawakatsu et al. (1979), Lin and Mahin (1985), Sewell et al. (1986), Singh et al. (1993), Villaverde (2006), Singh et al. (2006). However, most studies used different measures for quantifying the nonlinearity of the building (e.g., displacement ductility ratio). The focus of this paper is on the influence that inelastic MDOF structural behavior has on the response of acceleration-sensitive NSCs. More specifically, this paper summarizes ongoing research to quantify a proposed acceleration-response modification factor ($R_{acc}$) useful to estimate the seismic demands imposed on acceleration-sensitive NSCs attached to inelastic multistory structures. The proposed $R_{acc}$ factor can be used similar to response modification factors for primary structures to scale elastic FRS to obtain estimates of inelastic FRS (Sankaranarayanan and Medina, 2007). The terms ‘elastic’ and ‘inelastic’ refer to the primary structure, for only linear elastic NSCs are evaluated.

The study by Singh et al. (1993) defined a response reduction factor or R-factor which is the ratio between the elastic and inelastic absolute accelerations of the subsystem. The conclusion was that for normal equipment, the use of R-factors provides a practical and simple approach to include the effect of yielding in the calculation of forces on NSCs. This observation applies to cases in which yielding decreases floor response spectrum ordinates. The study by Sewell et al. (1986) attempted to understand the factors influencing the equipment response mounted on nonlinear structures and also the peak component acceleration amplification that can occur in some regions of FRS. The application of their results to typical regular buildings of the type considered in this study is limited because of two primary reasons: (a) the reference shear beam (stick) structural model used was representative of a typical, fixed-base, stiff Nuclear Power Plant structure; (b) the size of the ground motion sample considered was too small to establish any firm correlations between input motions and the results. More recent research that proposed simplified methodologies for estimating seismic design forces for NSCs acknowledged the need for in-depth studies to quantify the $R_{acc}$ factor or similar factors as a function of different properties of the supporting structure and component/equipment (Singh et al., 2006; Villaverde, 2006).

A recent paper by Sankaranarayanan and Medina (2007) quantified the influence of various parameters such as height of the supporting structure, location of NSC, damping ratio of NSC, and level of inelasticity of the building on the $R_{acc}$ factor. The large number of building and ground motion parameters used in this study allowed the estimation of $R_{acc}$ values and their associated uncertainties. It was concluded that the $R_{acc}$ factor was primarily a function of the location of the component, its damping ratio and period of vibration ($T_C$), as well as the fundamental period of vibration ($T_{B1}$) and the level of inelasticity of the primary structure. In this paper, statistical models for $R_{acc}$ were developed and evaluated based on inelastic dynamic analyses with multistory frame structures exposed to 40 ground motions. The frame structures had periods from 0.3 to 2.4 s. and number of stories from 3 to 18. It was concluded that in most cases, inelasticity in the primary structure reduces peak component acceleration demands. However, in several cases, inelasticity in the primary structure amplifies the acceleration demands corresponding to elastic primary structures ($R_{acc} < 1$), particularly for components located at the bottom floors with fundamental periods smaller than half the fundamental period of the primary structure (i.e., short-period region). One of the possible factors for this amplification is the modal interaction or internal resonance of the modes of the nonlinear system that causes $R_{acc}$ values in the higher mode regions to decrease (Nayfeh and Mook, 1979). This amplification of peak acceleration demands also depends strongly on the distribution of inelastic behavior along the height of the structure.

## 2. ACCELERATION RESPONSE MODIFICATION FACTOR ($R_{acc}$)

The acceleration response modification factor, $R_{acc}$, is defined as the ratio of the peak component acceleration demand of a NSC attached to an elastic building to the peak component acceleration demand experienced by the NSC when it is attached to an inelastic building. This paper deals exclusively with linear elastic NSC that can be modeled by a Single-Degree-Of-Freedom (SDOF) system. These results do not consider the nonlinearity of the equipment and is valid for light components (i.e., those with a mass of less than about 1% of the mass of the primary structure) that do not offer dynamic feedback to the building. A typical plot of the parameter $R_{acc}$ along with the elastic and inelastic floor response spectrum is presented in Fig. 1 for a NSC attached to the fourth floor level (Relative Height, $RH$ = 0.33) of a 9-story moment-resisting frame structure exposed to a recorded ground motion. The fundamental period of the building, $T_{B1}$, is equal to 0.9 s and its degree of inelastic behavior is defined by the parameter, $RI$, which is analogous to the ductility-dependent response modification factor. Thus, $RI$ = 0.25 implies elastic behavior and $RI$ = 4.0 inelastic behavior. The damping ratio of the component is 5%. Rayleigh damping equal to 5% of critical is also assumed for the 1st and 3rd modal periods of the building. As seen in Fig. 1, the floor response spectrum can be divided into 3 different regions: Short-Period (High-Frequency) Region, Fundamental Period Region, and Long-Period Region. These regions are defined by the ratio of the period of the component to the fundamental period of the building, $T_C / T_{B1}$. The ordinates of the graph represent $R_{acc}$ factor, as well as the peak component acceleration, $S_{aC}$. In each one of the 3 regions, the maximum $R_{acc}$ values are denoted as $(R_{acc\text{-}HF})_{\max}$, $R_{acc1}$, $(R_{acc\text{-}LF})_{\max}$ while the minimum values are labeled $(R_{acc\text{-}HF})_{\min}$, $(R_{acc1})_{\min}$, $(R_{acc\text{-}LF})_{\min}$. $R_{acc} < 1$ represents an increase in peak component acceleration demands due to inelastic behavior of the primary structure and $R_{acc} > 1$ a decrease in peak component acceleration demands.

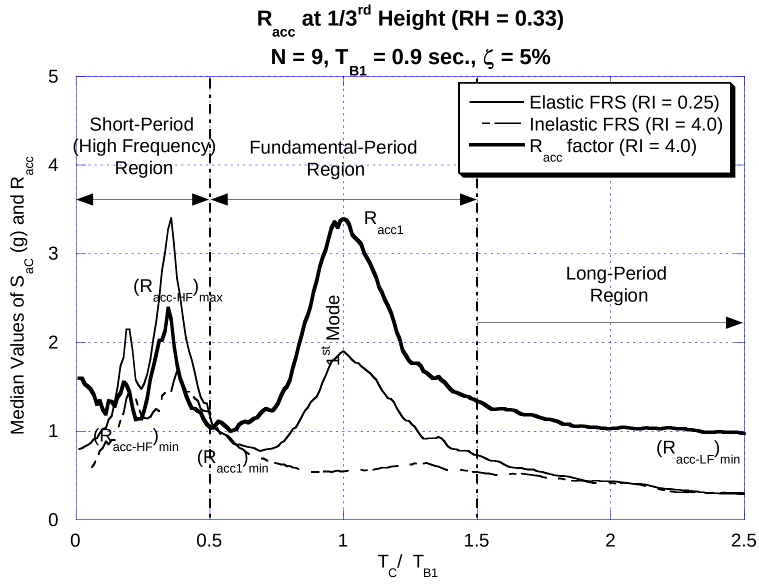

Figure 1. Acceleration response modification factor in three different FRS regions (after Sankaranarayanan and Medina, 2007).

The most important factors that influence the amplification of peak component acceleration demands caused by inelastic behavior of the primary structure are the location of the component in the structure, the damping ratio of the component, the level of inelastic behavior of the primary structure, and the spread of inelasticity in the primary structure. For instance, Fig. 2 illustrates the dependence of $(R_{acc})_{min}$ on the location of the component in the building, as well as the component damping ratio. Median $R_{acc}$ values are calculated based on inelastic time history responses with 40 recorded ground motions. The maximum increase in peak component acceleration demands is observed when the component damping ratio is small ($\zeta < 1\%$) and when the component is located at the bottom of the structure. Values of $R_{acc}$ less than one are typically found in regions that correspond to component periods that lie between the modal periods of the primary structure while maximum $R_{acc}$ values are observed when the component period matches one of the modal periods of the primary structure.

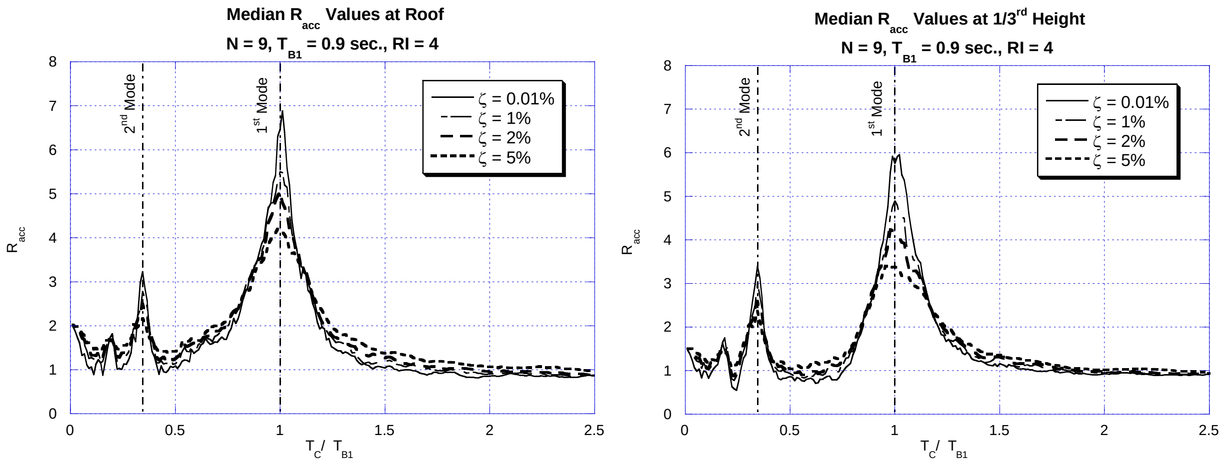

Figure 2. Effect of component damping ratio on acceleration response modification factors, $N = 9$, $T_{B1} = 0.9$ s., $RI = 4$, (a) at roof, and (b) at bottom-third location (after Sankaranarayanan and Medina, 2007).

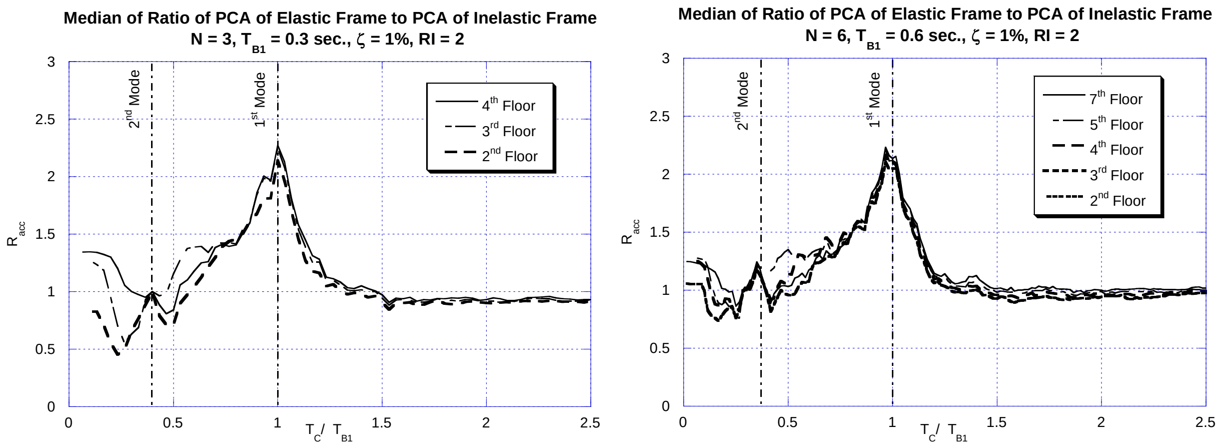

Figure 3. Acceleration response modification factor for 3- and 6-story frame structures that exhibit a weak-first story, $RI = 2$, component damping ratio = 1%.

The increase in peak component acceleration demands due to inelasticity of the primary structure is more pronounced when inelastic behavior is concentrated in a story or in a few adjacent stories in a building. For instance, if a moment-resisting frame exhibits a weak-story mechanism, $R_{acc}$ plots resemble those shown in Fig. 3. In this case it can be observed that $R_{acc} < 1$ can be found in most of the short-period region and at various floor levels when the component damping ratio is 1% and the building is mildly inelastic (i.e., $RI = 2$). The spread of inelasticity in the structure has a significant influence on the peak strength demands experienced by acceleration-sensitive NSCs. This is of particular importance for buildings that are prone to develop story mechanisms and those that are designed to experience a concentration of inelastic behavior at the base. For structures designed to experience inelastic behavior along its height, the increase in peak component acceleration demands is not as significant, especially when the component damping ratio is greater than 1%. The $R_{acc}$ parameters that represent amplification of FRS are more important from a design point of view as they make the current practice of calculating component seismic design forces based on elastic structures inadequate.

This is especially true in the short-period region. The scope of this paper is limited to the development of statistical models to estimate $R_{acc}$ values that signify a decrease in peak component acceleration demands. These models are more relevant for seismic performance assessment studies, which require the reliable quantification of seismic demands. Statistical models to estimate $R_{acc}$ values that correspond to an increase in peak component acceleration demands are part of a parallel study conducted by the second author.

## 3. STATISTICAL MODELS FOR $(R_{acc})_{max}$

This section deals with the development of equations to predict median $(R_{acc})_{max}$ values in the 3 floor response spectrum regions identified in Fig. 1. The decrease of inelastic FRS peaks in each of these floor response spectral regions can be represented by a single $R_{acc}$ value. The $R_{acc1}$ value exemplifies the entire fundamental-period region. In the short-period region, the $(R_{acc\text{-}HF})_{max}$ is characteristic of all the peaks. $(R_{acc\text{-}HF})_{max}$ represents the inelastic floor response spectrum envelope value in the short-period region as long as the peak component acceleration demand for the elastic building is used to estimate the inelastic floor response spectrum envelope value.

### 3.1. Long-Period Region: $(R_{acc\text{-}LF})_{max}$

In this region, a value of $(R_{acc\text{-}LF})_{max} = 1.0$ is proposed. $R_{acc} < 1.0$ can be found for only a few cases with small component damping ratios. When component damping ratios are greater than 1%, median $R_{acc}$ values are close to one.

### 3.2 Fundamental-Period Region: $R_{acc1}$

Inelastic FRS in the fundamental period are governed by the $R_{acc1}$ factor. Sankaranarayanan and Medina (2007) showed that the $R_{acc1}$ factor depends on location (i.e., relative height, $RH$), relative intensity of ground motion ($RI$), and NSC damping ratio ($\zeta$). The following statistical model was developed to estimate $R_{acc1}$ :

$$R_{acc1}(RH, RI, \zeta) = a\, RI^{b}, \text{ where } 2 \leq RI \leq 4 \qquad (3.1)$$

The results from a linear regression analysis with the natural logarithm of the values are provided in Table 1 where the coefficients $a$ and $b$ are regression parameters that depend on the values of $RH$ and $\zeta$ .The correlation coefficients for the simple statistical model are greater than 0.7, and they indicate relatively strong positive correlation between $R_{acc1}$ and $RI$ values. The standard errors of the estimate, which are a measure of the accuracy of predictions made with the regression line, are of the order of 0.3. Thus, the simple model used in the regression is capable of accurately predicting median $R_{acc1}$ values.

Table 1. Regression coefficients for equation 3.1.

| *RH* | $\zeta$ | *a* | *b* | Correlation Coefficient | Standard Error |
|---|---|---|---|---|---|
| 1.0 | 0.0001 | 1.079 | 1.335 | 0.738 | 0.347 |
| 1.0 | 0.01 | 1.015 | 1.259 | 0.752 | 0.314 |
| 1.0 | 0.02 | 0.981 | 1.208 | 0.758 | 0.296 |
| 1.0 | 0.05 | 0.932 | 1.105 | 0.763 | 0.264 |
| 0.5 | 0.0001 | 1.107 | 1.410 | 0.750 | 0.354 |
| 0.5 | 0.01 | 1.037 | 1.326 | 0.756 | 0.327 |
| 0.5 | 0.02 | 0.991 | 1.278 | 0.759 | 0.312 |
| 0.5 | 0.05 | 0.943 | 1.161 | 0.761 | 0.280 |
| 0.33 | 0.0001 | 1.229 | 1.257 | 0.740 | 0.326 |
| 0.33 | 0.01 | 1.134 | 1.167 | 0.744 | 0.298 |
| 0.33 | 0.02 | 1.079 | 1.113 | 0.746 | 0.282 |
| 0.33 | 0.05 | 1.024 | 0.976 | 0.737 | 0.253 |

### 3.3. Short-Period Region: $(R_{acc\text{-}HF})_{max}$

$R_{acc}$ values in the short-period region can be defined by a single parameter, namely, $(R_{acc\text{-}HF})_{max}$. This parameter takes into effect the decrease in inelastic FRS around all the higher modes of the supporting structure. Unlike the $R_{acc1}$ values, this parameter is dependent on $T_{B1}$. Hence, the following model is proposed:

$$(R_{acc\text{-}HF})_{max}\,(RH, RI, T_{B1}, \zeta) = c\,RI^{d}\,T_{B1}^{\;e}\text{ , where } 2 \le RI \le 4\text{ ; and } 0.6 \le T_{B1} \le 2.4\text{ s.} \tag{3.2}$$

Values for regression coefficients $c$, $d$, and $e$ are provided in Table 2. The statistical model for $(R_{acc\text{-}HF})_{max}$ is less accurate than the one proposed to predict $R_{acc1}$ (i.e., correlation coefficients of the order of 0.65 and standard errors close to 0.35). This is understandable because of the dependence of $(R_{acc\text{-}HF})_{max}$ on $T_{B1}$ and the fact that this parameter represents the entire short-period region. However, equation 3.2 is considered to be sufficiently accurate to reasonably predict the decrease in peak component acceleration demands present in this region.

Table 2. Regression coefficients for equation 3.2.

| $RH$ | $\zeta$ | $c$ | $d$ | $e$ | Correlation Coefficient | Standard Error |
|---|---|---|---|---|---|---|
| 1.0 | 0.0001 | 1.147 | 1.018 | 0.552 | 0.666 | 0.413 |
| 1.0 | 0.01 | 1.051 | 0.931 | 0.467 | 0.664 | 0.369 |
| 1.0 | 0.02 | 1.012 | 0.890 | 0.432 | 0.658 | 0.355 |
| 1.0 | 0.05 | 0.982 | 0.809 | 0.364 | 0.635 | 0.318 |
| 0.5 | 0.0001 | 1.181 | 0.929 | 0.556 | 0.665 | 0.393 |
| 0.5 | 0.01 | 1.060 | 0.873 | 0.483 | 0.660 | 0.363 |
| 0.5 | 0.02 | 1.020 | 0.832 | 0.465 | 0.653 | 0.353 |
| 0.5 | 0.05 | 1.010 | 0.732 | 0.393 | 0.615 | 0.320 |
| 0.33 | 0.0001 | 1.215 | 0.984 | 0.635 | 0.651 | 0.447 |
| 0.33 | 0.01 | 1.079 | 0.906 | 0.556 | 0.644 | 0.409 |
| 0.33 | 0.02 | 1.034 | 0.853 | 0.539 | 0.640 | 0.395 |
| 0.33 | 0.05 | 0.960 | 0.787 | 0.498 | 0.623 | 0.360 |

### 3.4 Dispersion of $R_{acc}$ values

The statistical models explained above are useful for predicting median $R_{acc}$ values. The dispersion of $R_{acc}$ increases with the value of component damping ratio and the fundamental period of the primary structure. The standard deviation of the natural logarithm of both $R_{acc1}$ and $(R_{acc\text{-}HF})_{max}$ varies from 0.1 to 0.45.

## 4. CORRELATION OF STATISTICAL MODELS WITH RESULTS FOR A MULTI-BAY FRAME

The single-bay, generic frame models used in this study are not geared for rigorous demand prediction for a specific building. They are meant to represent the global seismic behavior of multi-bay frames. An actual building model will have unique characteristics that might deviate from the assumptions used in the development of generic models. The objective of this section is to correlate the $R_{acc}$ factors obtained for elastic NSCs mounted on an inelastic multi-bay reinforced-concrete frame structure (Van Nuys building model (Krawinkler, 2005)) with those obtained using equations 3.1 and 3.2. The Van Nuys building model exhibits cyclic strength and stiffness deterioration at the component level.

It can be seen from Fig. 4(a) that the shape of the $R_{acc}$ curves corresponding to the Van Nuys building model is consistent with the shape obtained in this study using the family of generic frames. In addition, the statistical models proposed in this study provide slightly conservative estimates of $R_{acc1}$ and slightly unconservative estimates of $(R_{acc\text{-}HF})_{max}$ (see Fig. 4(b)). However, in most cases, the proposed statistical models predicted values that were within 20% of the actual ones. These percentage differences are not considered to be significant given the relatively large uncertainty present in the estimation of $R_{acc}$ as discussed in Section 3.4.

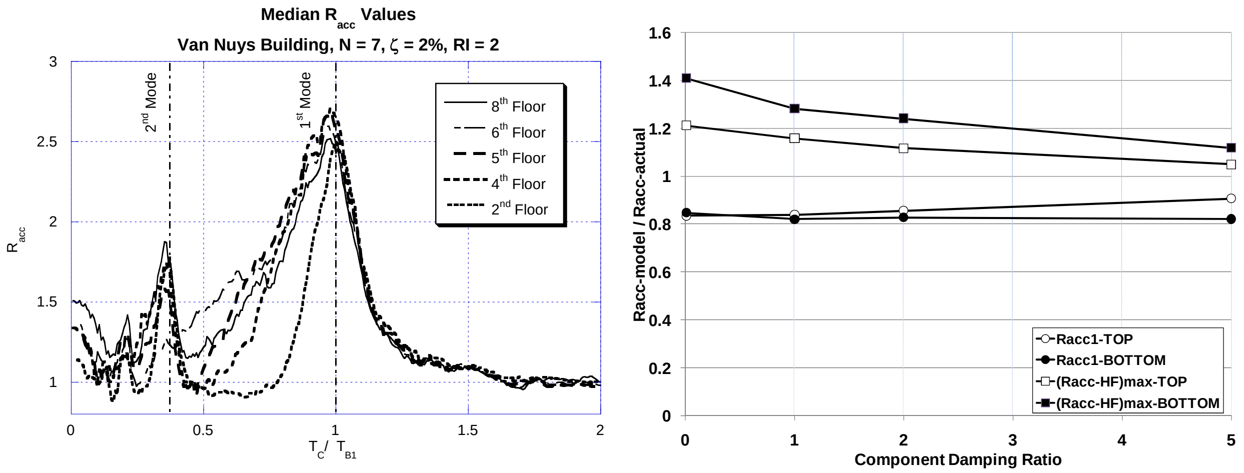

Figure 4. (a) Median $R_{acc}$ values for Van Nuys building model, $RI = 2$; (b) ratio of $R_{acc}$ based on proposed statistical models to median $R_{acc}$ value for Van Nuys building model at top ($RH = 1.0$) and bottom ($RH = 0.3$).

## 5. CONCLUSIONS

The results from this study demonstrate that the prediction of the peak response of acceleration-sensitive nonstructural components attached to inelastic buildings is a challenging task. The basic approach undertaken in this study was based on estimating a proposed acceleration response modification factor, $R_{acc}$, to obtain peak acceleration demands of components mounted on inelastic structures based on their response when they are mounted on elastic structures. This paper is limited to the quantification of $R_{acc}$ values that signify a decrease in peak component acceleration demands. Floor response spectra were developed and 3 main regions were identified as a function of the ratio of the period of the component to the fundamental period of the building: short-period region, fundamental-period region, long-period region. It was noted that inelasticity in the primary structure has the potential to either increase or decrease the peak component acceleration demands. Whether inelasticity is beneficial or detrimental to the component and/or its attachment depends upon how the period of the component compares to the period of the primary structure, the damping ratio of the component, and the location of the component in the building.

Statistical models to predict acceleration response modification factors for components with periods in tune with the modal periods of the primary structure were proposed in this study. It was demonstrated that in the fundamental-period region, the reduction in peak acceleration demands due to inelasticity of the primary structure is a function of the level of inelastic behavior of the primary structure, the location of the component, and the damping ratio of the component. In the short-period region, the decrease in peak component acceleration demands is also a function of the fundamental period of the primary building. Thus, these statistical models are primarily applicable to seismic performance evaluation studies with components with periods close to the modal periods of the primary building. An evaluation of the proposed statistical models with results obtained by simulating the seismic response of a multi-bay, 7-story reinforced-concrete building structure showed that the models were able to predict the acceleration response modification factors within 20% in most cases. This difference is not considered significant given the uncertainties present in the quantification of acceleration response modification factors for multistory structures.

The main goal of the paper was to provide guidance for the development of models to predict floor response spectra for inelastic buildings. Estimates of the coefficients that form part of the proposed models need to be improved and statistical models for $R_{acc}$ factors that account for the increase in peak acceleration demands due to inelasticity of the primary structure need to be developed. However, the underlying principles behind the overall properties of the acceleration response modification factor and its dependence on critical ground motion and structural response parameters have been identified. The interpretation of the results from this study needs to be made within the conditions described for the structural models and ground motions used.

## ACKNOWLEDGMENTS

This study has been partially supported by the National Science Foundation through grant # NSF 0753684. This financial support is gratefully acknowledged.

## REFERENCES

ATC (2007). Interim Testing Protocols for Determining the Seismic Performance Characteristics of Structural and Nonstructural Components (FEMA 461). Federal Emergency Management Agency, Washington DC.

BSSC (2003). The 2003 NEHRP Recommended Provisions For New Buildings And Other Structures Parts 1 and 2: Provisions and Commentary (FEMA 450). Federal Emergency Management Agency, Washington DC.

Filiatrault, A., R. Tremblay and S. Kuan (2004). Generation of floor accelerations for seismic testing of operational and functional building components. *Canadian Journal of Civil Engineering* **31:4**, 646-663.

Kawakatsu, T., K. Kitade, T. Takemori, Y. Kuwabara and Y. Ogiwara (1979). Floor Response Spectra Considering Elasto-Plastic Behavior of Nuclear Facilities. *5th International Conference on Structural Mechanics in Reactor Technology*, Berlin, Germany.

Krawinkler, H. (2005). Van Nuys Hotel Building Testbed Report: Exercising Seismic Performance Assessment. PEER Report 2005/11. Pacific Earthquake Engineering Research Center, Berkeley, CA.

Lin, J. and S. A. Mahin (1985). Seismic response of light subsystems on inelastic structures. *Journal of Structural Engineering-ASCE* **111:2**, 400-417.

Nayfeh, A. H. and Mook, D.T. (1979). Nonlinear Oscillations. Wiley, New York.

Retamales, R., G. Mosqueda, A. Filiatrault and A. M. Reinhorn (2006). Experimental Study on the Seismic Behavior of Nonstructural Components Subjected to Full-Scale Floor Motions. *8th U.S. National Conference on Earthquake Engineering*. Paper No. 1359, San Francisco, CA.

Sankaranarayanan, R., and Medina, R.A. (2007). Acceleration response modification factors for nonstructural components attached to inelastic moment-resisting frame structures. *Earthquake Engineering and Structural Dynamics* **36:14**, 2189-2210.

Sewell, R. T., C. A. Cornell, G. R. Toro and R. K. McGuire (1986). A study of factors influencing floor response spectra in nonlinear multi-degree-of-freedom structures. JABEEC Report No. 82. Department of Civil and Environmental Engineering, Stanford University, Palo Alto, CA.

Singh, M. P., L. E. Suarez, E. E. Matheu and G. O. Maldonado (1993). Simplified Procedures for Seismic Design of Nonstructural Components and Assessment of Current Code Provisions. NCEER-93-0013. National Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, NY.

Singh, M. P., L. M. Moreschi, L. E. Suarez and E. E. Matheu (2006). Seismic design forces. II: Flexible nonstructural components. *Journal of Structural Engineering-ASCE* **132:10**, 1533-1542.

Villaverde, R. (2006). Simple method to estimate the seismic nonlinear response of nonstructural components in buildings. *Engineering Structures* **28:8**, 1209-1221.