

IN-STRUCTURE SEISMIC ACCELERATION RESPONSE SPECTRA GENERATED FROM NONLINEAR VS. LINEAR BUILDING RESPONSE ANALYSIS

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ABSTRACT

Linear and nonlinear analyses of the response of a fictitious reinforced concrete building with a HCLPF capacity of 0.65g PGA to the ground motion of the 1994 magnitude 6.7 Northridge earthquake were performed to study differences in the in-structure accelerations depending on which of the two modeling assumptions were used. Analyses were performed for three levels of ground motion intensity by scaling the actual ground motion records to 0.75g, 0.5g and 0.2g PGA. The results indicated quite marked differences in the in-structure response spectra, with the values from the nonlinear analyses being lower, even at the low 0.2g PGA level for the relatively high capacity structure. In particular, the spectral peaks and typically, but not always, also the zero period acceleration values from the nonlinear analyses, were lower than those obtained from the linear analyses. The results of the work provide insights to the magnitude and the nature of the differences between in-structure spectra generated from linear vs. nonlinear response analyses and help in assessing when nonlinear analysis may be beneficial and more appropriate. The results and conclusions will aid in the process to enhance the procedures for seismic response analysis and equipment seismic qualification in nuclear power plant structures.

INTRODUCTION

Typically, seismic qualification of equipment mounted in nuclear power plant buildings is performed based on the in-structure acceleration response spectra at the location of the equipment. In the past, the in-structure acceleration response spectra have customarily been computed from the acceleration time histories stored for the desired locations throughout a building during a linear building time history response analysis. With the evolving requirements for seismic evaluations, especially with the need to perform seismic evaluations for higher levels of ground motions in response to the Fukushima event and the Generic Safety Issue GI-199 [US NRC (2005)], there is a growing need for nonlinear building response analysis and generation of in-structure acceleration response spectra using the responses from nonlinear response analyses.

With the advances in computer hardware and software, performing nonlinear response analyses of NPP structures has not only become feasible, but significant experience has been gained in such analyses in various engineering disciplines. However, to date, the NPP industry practice is to perform the seismic response analyses using linear modeling. The main objective of the work discussed in this paper was to gain insights about the differences between in-structure spectra generated from nonlinear vs. linear response analyses.

MODELING

A finite element model of a fictitious reinforced concrete building, shown in Figure 1, was set up using LS-DYNA [LSTC (2013)] software for the purposes of this study. The building plan is 22m square and the height is 18m. The building has four shear wall “cores” as seen in the slice at the bottom of

Figure 1. The shear walls are 0.3m thick and the slabs are 0.4m thick. The columns are 0.3m square. The main lateral load resisting shear walls have a reinforcement ratio of 1% both horizontally and vertically. Material properties are provided in Table 1.

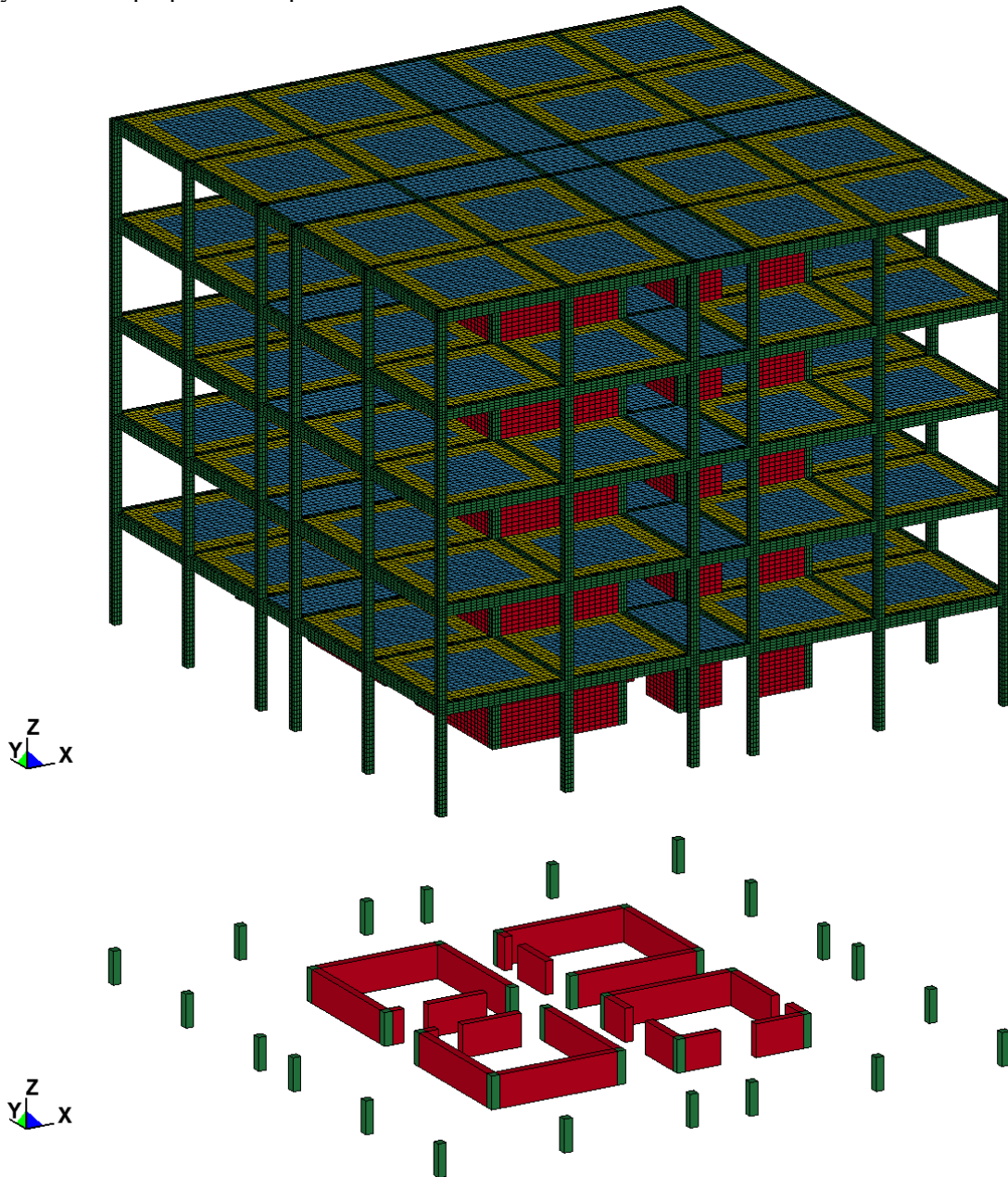


Figure 1: 3D view of solid model (top) and the base of structure (bottom)

Table 1: Material properties

Material	Modulus of Elasticity [MPa]	Poisson Ratio	Yield Stress [MPa]	Compressive Strength [MPa]	Tensile Strength [MPa]
Concrete C25/30	31,000	0.17	-	25.00	2.60
Reinforcing Steel	200,000	0.30	500	-	-

The Winfrith concrete material model (*MAT_085) in 8-noded solid elements with smeared reinforcement representation was used to model the reinforced concrete walls, slabs and columns. The walls and columns had three and slabs had four elements through the thickness. Four elements through the thickness were used for the slabs as vertical, out-of-plane accelerations were tracked at the selected mid-panel slab nodes; and typically, for single point integrated elements, four elements through the thickness are considered minimum. As flexural response was not of interest for the walls and columns, three elements through the thickness were used in this study – mainly to control run times.

EARTHQUAKE LOADING

Actual records from the Northridge 1994 magnitude 6.7 earthquake were utilized for the ground motion definition. The accelerograms and 5% damped acceleration response spectra from station IIEA097 USC (1994), with an epicentral distance of 10.8 km, are shown in Figure 2.

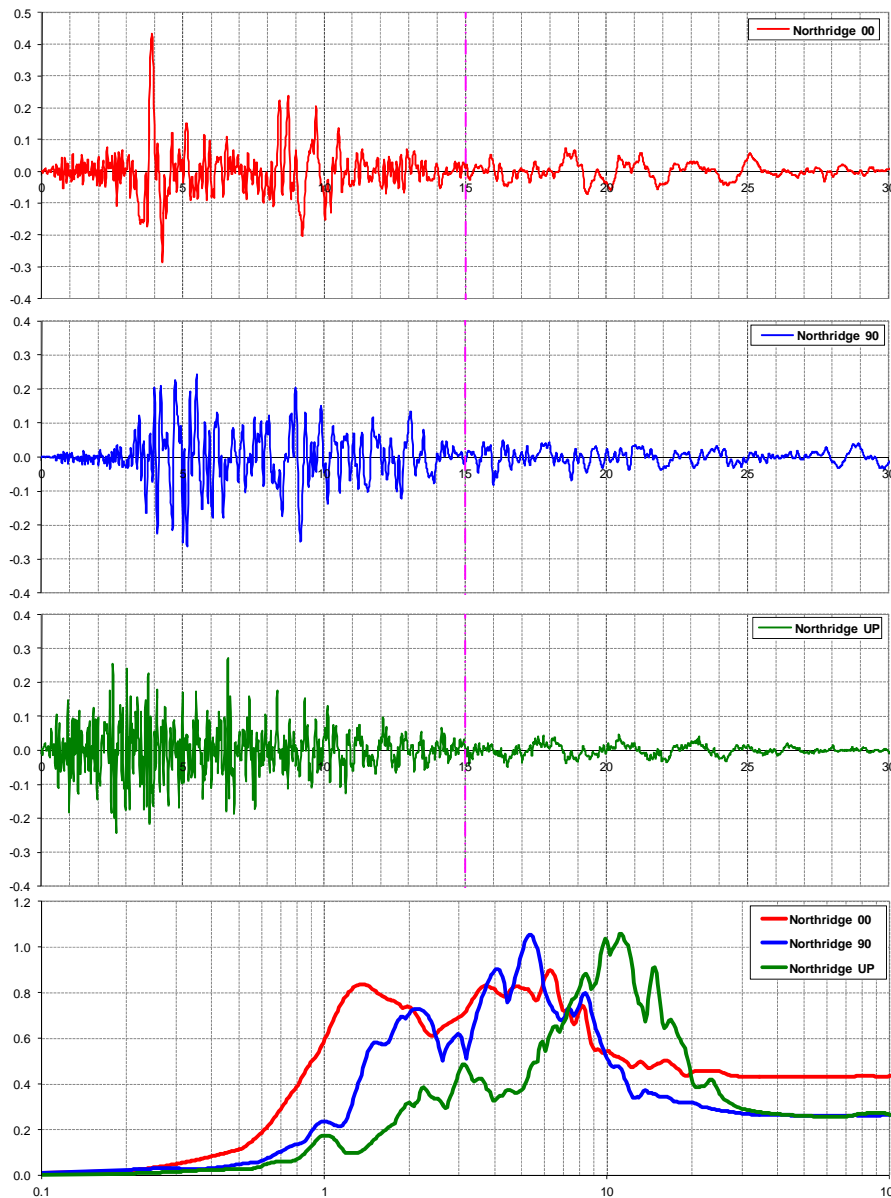


Figure 2: Ground motion: time histories [units are sec and g] and 5% damped acceleration response spectra [units are Hz and g]

ANALYSES AND RESULTS

HCLPF Capacity of the Building

Based on a response spectrum analysis with a linear version of the model, using a spectral shape provided by the USGS website for the vicinity of the Northridge earthquake epicenter, the HCLPF capacity [EPRI (1991, 1994, 2009)] was estimated as 0.65g PGA – governed by the shear wall capacity at the base.

Modal Frequencies

Modal analysis indicated the first modes, sway in X and Y-directions, having frequencies of about 6Hz.

Earthquake Time History Analysis

Time history analyses were performed and in-structure acceleration spectra computed utilizing the three ground motion acceleration histories of Figure 2 for six cases with the time histories scaled as follows:

1. The ground motion PGA scaled to 0.2g - linear analysis
2. Same as Case 1, except nonlinear analysis
3. The ground motion PGA scaled to 0.5g - linear analysis
4. Same as Case 3, except nonlinear analysis
5. The ground motion PGA scaled to 0.75g - linear analysis
6. Same as Case 5, except nonlinear analysis

Based on the material properties and element size, the explicit time integration algorithm in LS-DYNA set the time step based on the inherent stability condition to 2.2E-05. A constant 5% damping over the 5 Hz to 70 Hz frequency range was specified (*DAMPING_FREQUENCY_RANGE).

To control run times, the transient runs were performed only through the first 15 seconds of the excitation as it was evident that the lower level of excitation during the last 15 seconds did not affect the computed in-structure acceleration response spectra.

During the transient runs, displacements and accelerations were stored at the two nodes indicated in Figure 3.

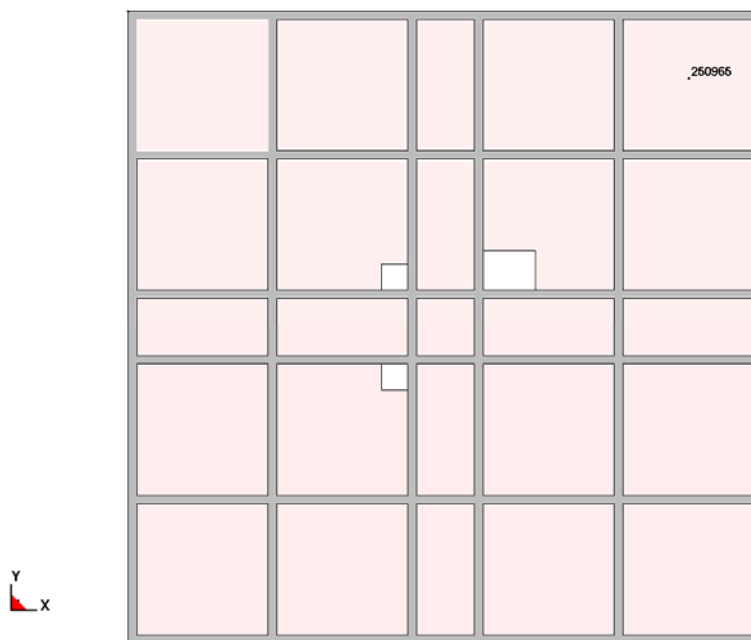


Figure 3: Location of the nodes on the 1st floor (node 250965) and the same location in plan on 4th floor (node 647925) where response spectra were computed

In all the following time history and spectra plots, the continuous curves are from linear building response runs and the dashed curves are from nonlinear building response runs.

Figure 4 shows relative displacement time histories in the X, Y, and Z-directions for the node on the fourth floor for Case 4; 0.50g PGA. As expected, the displacement peaks from the nonlinear run have higher magnitudes than the peaks from the linear run. The Z-curve from the nonlinear run also indicates slight permanent deformation – drop.



Figure 4: Relative displacements comparison – 0.50g, 4th floor (Node 647925) [units are sec and m]

Figure 5 illustrates deformation and cracking at the time of peak X-deflection – the plot has a displacement scale factor equal to 50; significant cracking in the lower level shear walls is evident. The slab deflections are larger at the top floors.

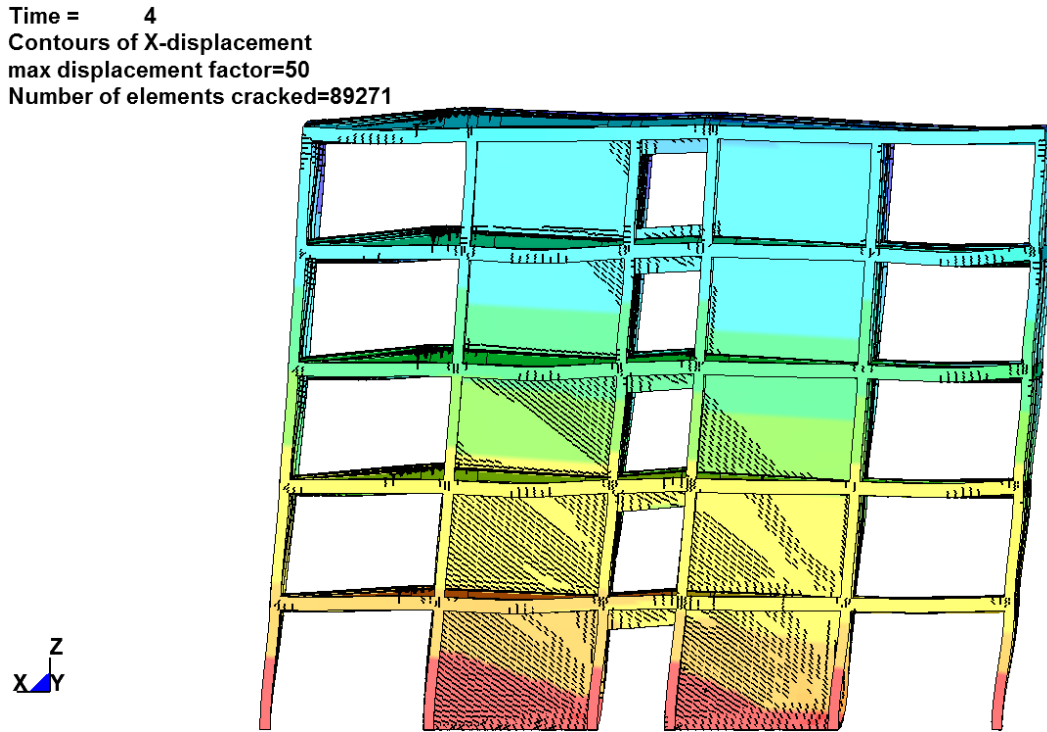


Figure 5: Cracking at the time of max horizontal displacements – displacement plot scale factor = 50
– 0.50g PGA

Figure 6, Figure 7, and Figure 8 show 5% damped in-structure acceleration response spectra in X, Y, and Z-directions for the ground motion of Figure 2 scaled to 0.20g, 0.50g, and 0.75g PGA, respectively; the node on the first floor on the left, the node on the fourth floor on the right. Differences, most obviously the drop of the spectra peaks from the nonlinear building response run vs. the linear building response run, are well evident already at the 0.20g PGA level; for a building with a HCLPF capacity of 0.65g PGA; and more evident at the higher PGA levels.

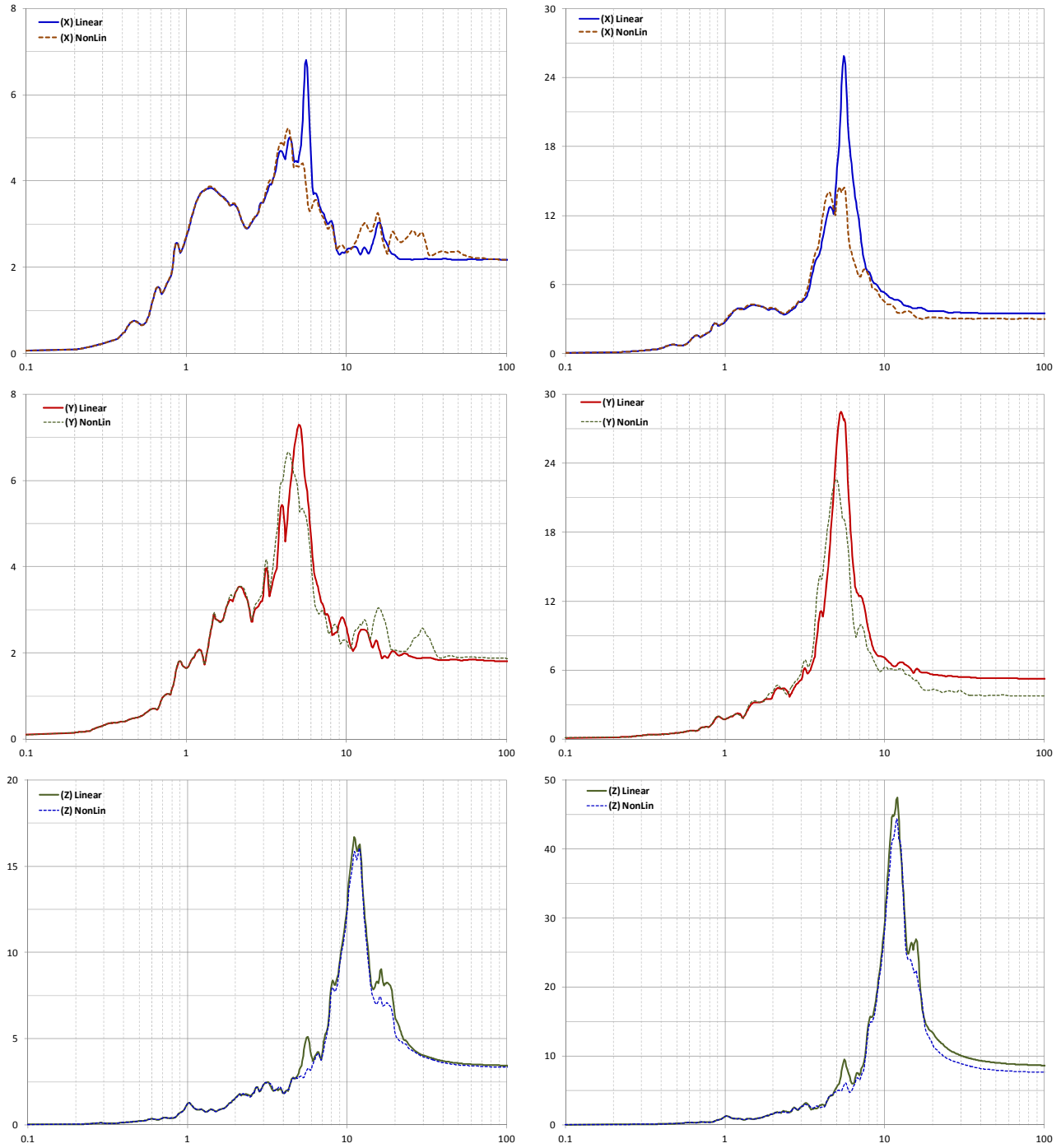


Figure 6: Response Spectra – Comparison – 0.20g, 1st floor (left) and 4th floor (right) [units are Hz and m/sec²]

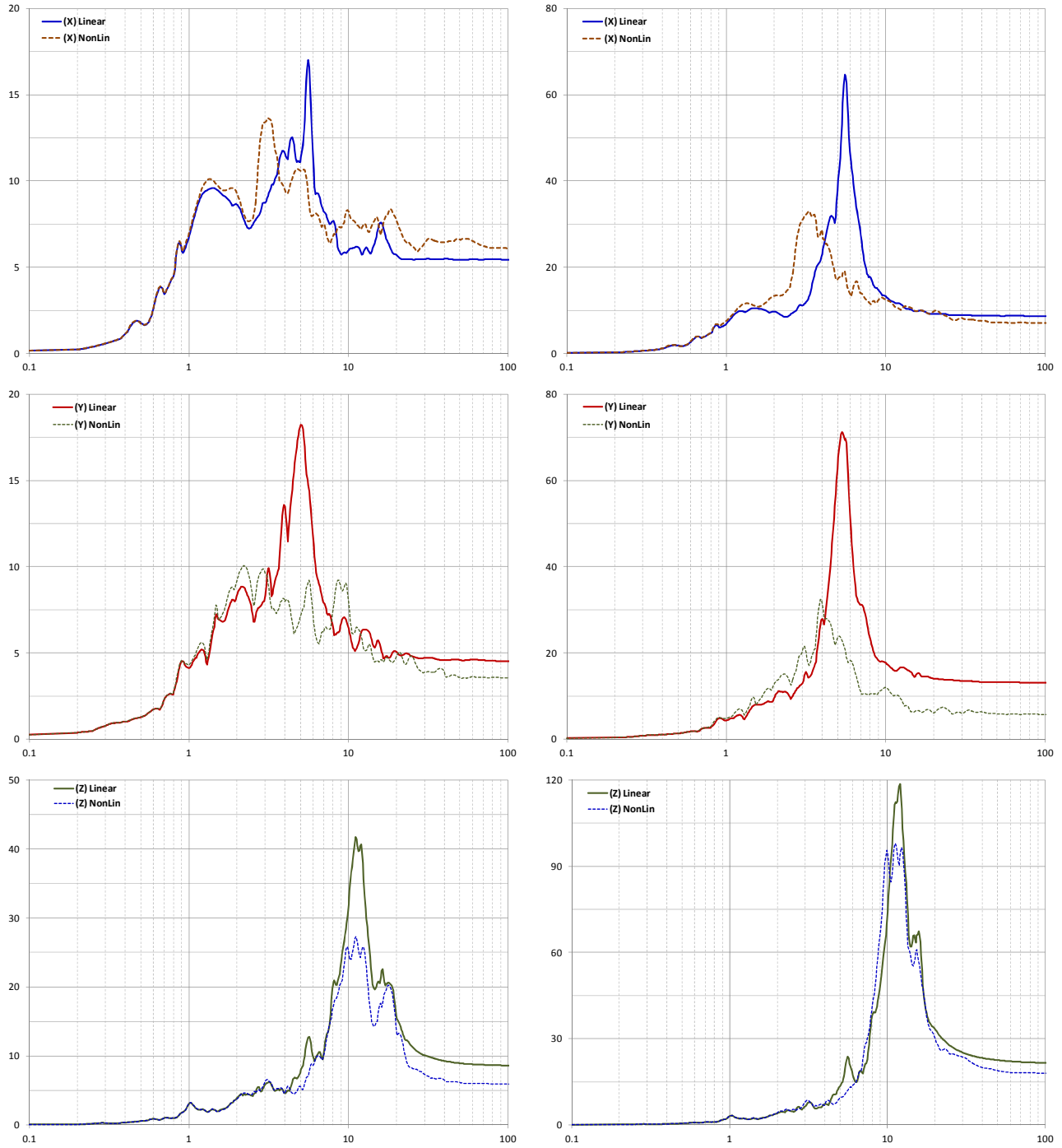


Figure 7: Response Spectra – Comparison – 0.50g, 1st floor (left) and 4th floor (right) [units are Hz and m/sec²]

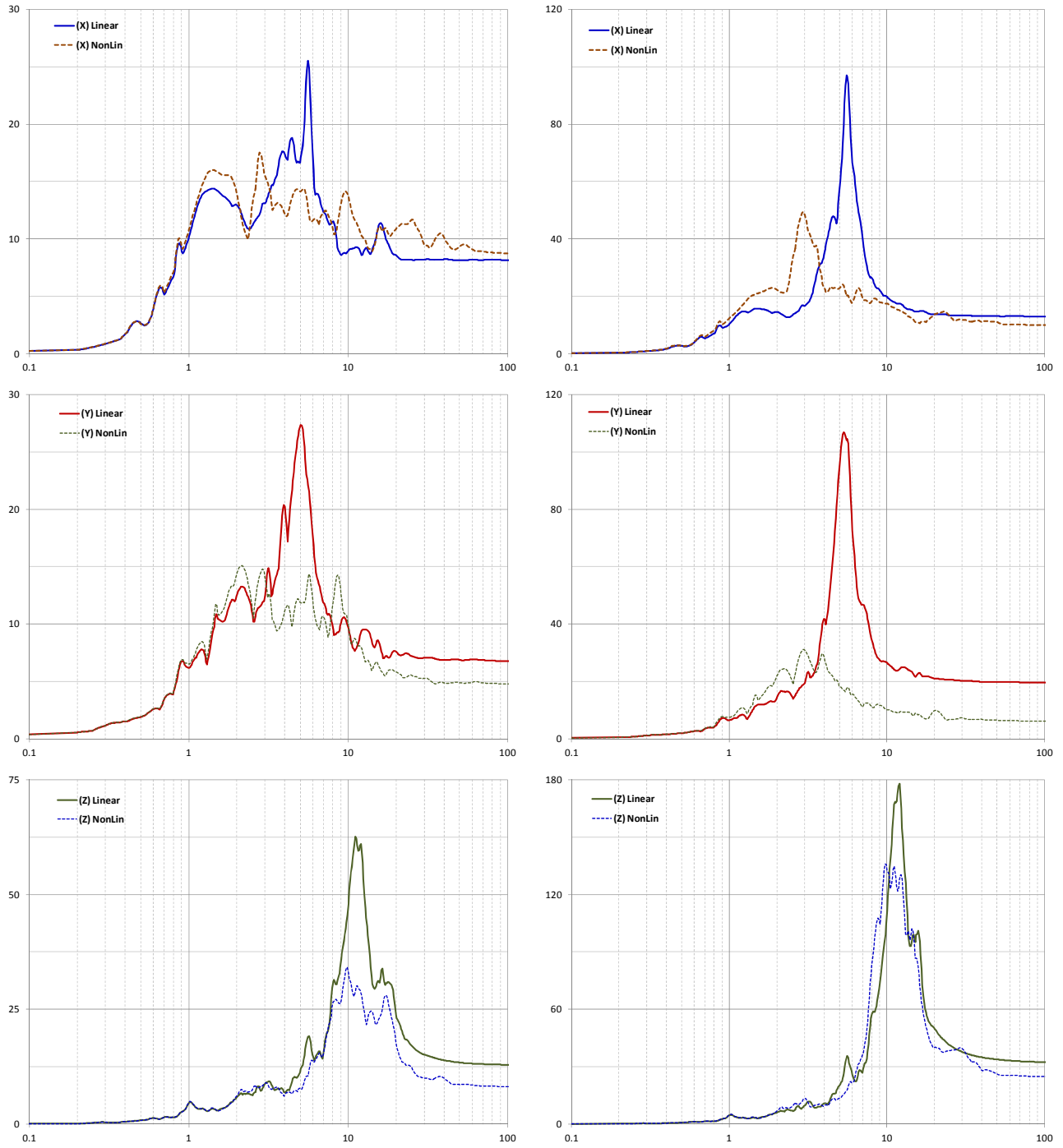


Figure 8: Response Spectra – Comparison – 0.75g, 1st floor (left) and 4th floor (right) [units are Hz and m/sec²]

CONCLUSION

Linear and nonlinear analyses of the response of a fictitious reinforced concrete building, with a HCLPF capacity of 0.65g PGA, to the ground motion of the 1994 magnitude 6.7 Northridge earthquake were performed to study differences in the in-structure accelerations depending on the method used. Analyses were carried out for three levels of ground motion intensity by scaling the actual ground motion records to 0.75g, 0.5g and 0.2g PGA. The results indicated quite marked differences in the in-structure response spectra; the values from the nonlinear analyses being lower, even at the low 0.2g PGA level for the relatively high capacity structure. Particularly, the spectral peaks and typically, but not always, also the zero period acceleration values from the nonlinear analyses were lower than those obtained from the linear analyses.

The results of the work provide insights as to the magnitude and the nature of differences between in-structure spectra generated from nonlinear vs. linear response analyses and provides indication when nonlinear analysis may be beneficial and more appropriate. The results and conclusions will aid in the process to enhance the procedures for the seismic response analysis and equipment seismic qualification in NPP structures.

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