EXPERIMENTAL STUDY ON THE SEISMIC BEHAVIOR OF NONSTRUCTURAL COMPONENTS SUBJECTED TO FULL-SCALE FLOOR MOTIONS

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ABSTRACT

Observations during past earthquakes have demonstrated the seismic vulnerability of nonstructural components and equipment with their expensive recovery and/or replacement costs. With the exception of the nuclear industry, the limited data collected form past earthquakes are not sufficient to completely characterize the seismic behavior of nonstructural components and develop effective mitigation measures. To address these limitations, the University at Buffalo's (UB-NEES) facility is commissioning a dedicated Nonstructural Component Simulator (UB-NCS) composed of a two-level testing frame capable of simultaneously subjecting both displacement-sensitive and acceleration-sensitive nonstructural components to realistic full scale floor motions expected in typical multi-story buildings. In order to generate the necessary data to evaluate the seismic performance of nonstructural components and quantify their experimental seismic fragility, a dynamic testing protocol capable of replicating expected average absolute floor accelerations and inter-story drifts has been developed. This paper summarizes the criteria considered for the selection of the required UB-NCS servo-hydraulic equipment, the main characteristics of the UB-NCS testing facility and the criteria considered to generate an adequate dynamic testing protocol.

Introduction

With the development of performance-based earthquake engineering, harmonization of the performance levels between structural and nonstructural components becomes vital. Even if the structural components of a building achieve an immediate occupancy performance level after a seismic event, failure of architectural, mechanical, or electrical components of the building can lower the performance level of the entire building system. This reduction in performance caused by the vulnerability of nonstructural components has been observed in several buildings during

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the recent 2001 Nisqually earthquake in the Seattle-Tacoma area (Filiatrault et al. 2001) and during several other earthquakes that have occurred in the last 40 years. According to Miranda et al. (2003), the contents and nonstructural components in office, hotel and hospital buildings compose about 82%, 87% and 92% of the total monetary investment in a building, respectively. Clearly the investment in nonstructural components and building contents is far greater than that of structural components and framing. Therefore, it is not surprising that in many past earthquakes, losses from damage to nonstructural building components exceeded losses from structural damage. Furthermore, failure of nonstructural building components could become safety hazards or could affect the safe movement of occupants evacuating or rescue workers entering buildings.

In comparison to structural components and systems, there is still relatively limited information on the seismic design of nonstructural components. Basic research work in this area has been sparse, and the available codes and guidelines (FEMA 1994, 2000, Canadian Standard Association 2002) are usually, for the most parts, based on past experiences, engineering judgment and intuition, rather than on objective experimental and analytical results. Often, design engineers are forced to start almost from square one after each earthquake event: observe what went wrong and try to prevent repetitions. This is a consequence of the empirical nature of current seismic regulations and guidelines for nonstructural components.

In order to reproduce in real-time the full-scale multi-axial seismic floor motions that are required to properly assess the seismic performance of nonstructural components, the University at Buffalo's (UB-NEES) facility is commissioning a dedicated Nonstructural Component Simulator (UB-NCS). The UB-NCS is a modular and versatile two-level platform for real-time experimental performance evaluation of displacement- and acceleration-sensitive nonstructural components and equipment. The UB-NCS can provide the dynamic stroke necessary to replicate full-scale displacements, velocities and accelerations at the upper levels of multi-story buildings during earthquake shaking. The input motions can be obtained from recorded floor motions of buildings during past earthquakes or the simulated numerical response of a building to a given earthquake record. In order to more broadly assess the seismic vulnerability of nonstructural components independent of building or earthquake record, a general testing protocol is proposed. Experimental testing protocols for displacement sensitive (racking protocol) and acceleration sensitive (shake table protocol) nonstructural components have been separately developed (ATC 2005), whereas the NCS seeks to simultaneously subject test specimens to expected absolute floor acceleration and inter-story drifts.

University at Buffalo's Non-Structural Component Simulator (UB-NCS)

The main requirements for performing real-time seismic testing of nonstructural components resides in the ability of the servo-hydraulic equipment to reproduce the multi-directional absolute floor motions at various levels of building structures excited by earthquake ground motions. In order to assess these equipment requirements, floor motions recorded in four instrumented buildings during major earthquakes in California were considered. One of these buildings was shaken by the 1989 Loma Prieta earthquake, while the other three were shaken by the 1994 Northridge earthquake. Table 1 summarizes the peak responses measured and estimated at the roof level of these instrumented buildings.

From Table 1, it can be seen that the kinematical equipment requirements to envelope the roof responses of the four instrumented buildings are a peak acceleration of ± 1.5g, a peak velocity of ± 82.7 in./s, and a peak-to-peak stroke of 80 in. Considering these peak demand parameters, the UB-NCS testing frame is activated by four identical high performance dynamic actuators capable of subjecting nonstructural components and equipment up to 3g horizontal accelerations, 100 in./s velocities and $\pm$ 40 in. displacements for specimens up to 6.9 kips per level. Each actuator has a reversal load capacity of 22 kips, a displacement stroke of 80 in. and a mid-stroke length of approximately 15 ft. Fig. 1 shows the uni-axial and bi-axial testing configurations for the UB-NCS. Vertical accelerations can also be included in an experiment by mounting the testing frame on one of the existing earthquake simulators (shake tables) at the UB-NEES facility.

Table 1. Peak seismic responses at roof level.

Building	Building Description and Location	Measured Peak Roof Acceleration (g)	Estimated Fundamental Period T (s)	Estimated Peak Roof Velocity (in/sec)	Estimated Peak Roof Displac. (in)
Pacific Park Plaza	30-story, 95 m height. Concrete shear walls and moment resisting frames. Emeryville, CA.	0.37 (Loma Prieta)	2.69	61.0	26.4
Olive View Medical Center	6-story. Concrete moment resisting frames and steel plate shear walls. Sylmar, CA.	1.50 (Northridge)	0.33	30.3	1.57
7-story R/C building	Moment resisting frames in perimeter and flat plates and columns in the interior. Van Nuys, CA.	0.58 (Northridge)	1.98	70.5	22.4
13-story R/C building	Non-ductile moment resisting concrete frames with concrete shear walls in basements. Sherman Oaks, CA	0.45 (Northridge)	3.00	82.7	39.4

The testing frame is composed of two square 12.5 ft platforms with an inter-story height of 14 ft. Beams are typically constructed from HSS8x6x1/2” hollow tube sections and the columns are made of HSS8x8x1/2” (Fig. 2). The platform is a 2x2 ft grid with tie-down holes spaced at 1 ft. Additionally, four centrally located cruciform shapes are removable to provide four 3.5x3.5 ft square openings which can accommodate tall equipment that may span more than one level. Universal joints, used at the column-platform connections, allow for the unrestricted bi-directional motion of the frame. The frame can be braced in one direction for unidirectional testing. Figures 3 and 4 show the predicted deformed shape of the NCS and a comparison between the nominal and the experimentally obtained performance curves from 2 of the 4 actuators that have already been tested.

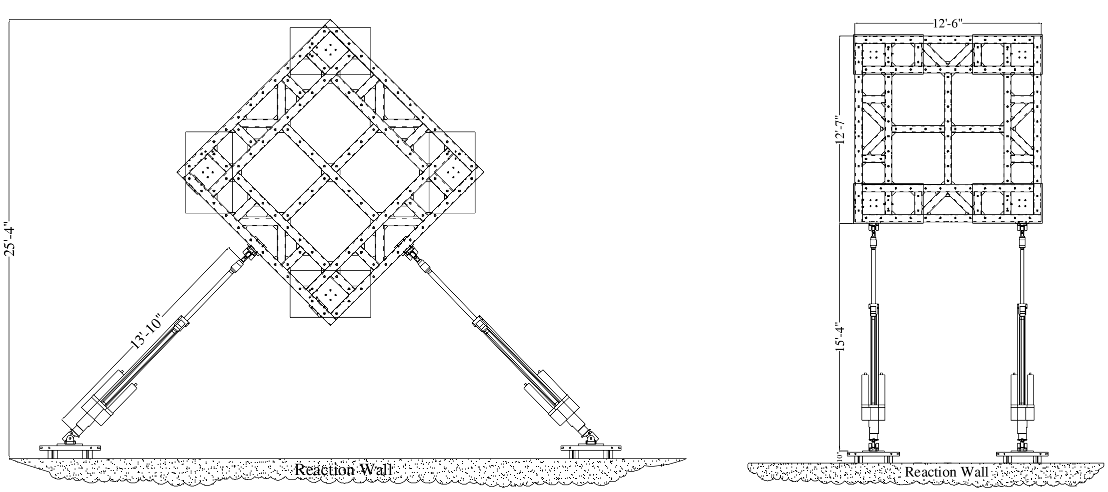

Figure 1. Plan view of bi-axial and uni-axial testing configurations.

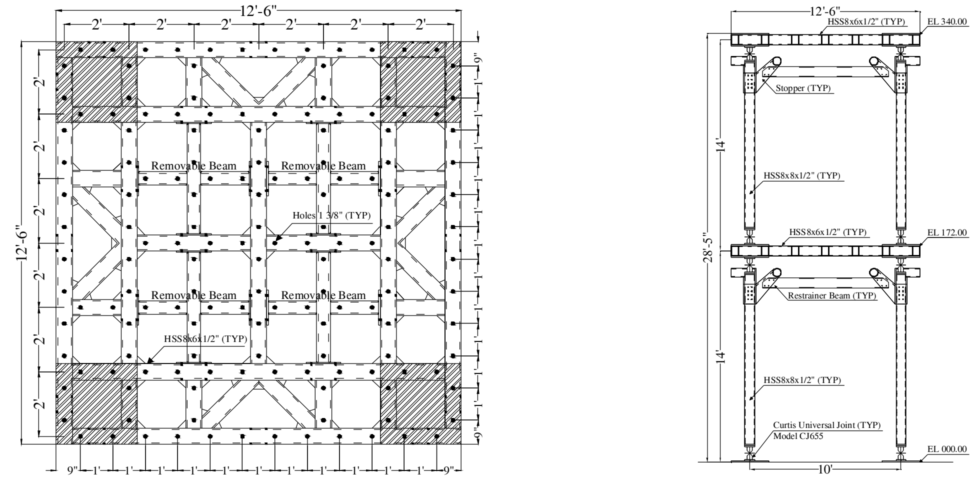

Figure 2. Plan view and elevation of UB-NCS.

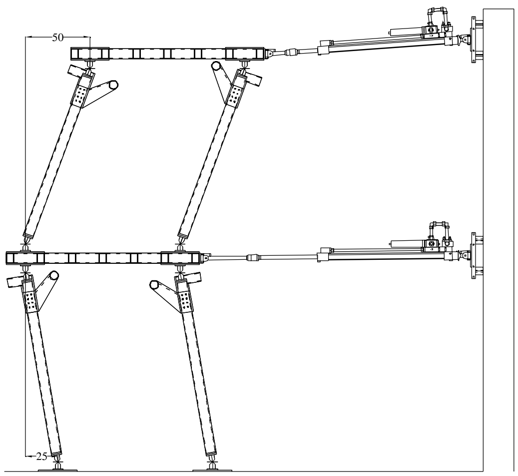

Figure 3. Deformed shape of UB-NCS (32% inter-story drift at top NCS level).

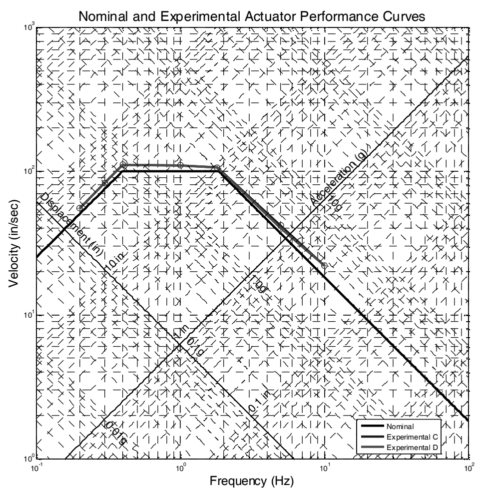

Figure 4. Actuator performance curves.

Development of a Dynamic Testing Protocol

A dynamic testing protocol has been developed to be used with the NCS. The main objective is to obtain the necessary experimental data for evaluating the seismic performance and generating the fragility curves of displacement- and acceleration-sensitive nonstructural components. The proposed protocol simultaneously subjects test specimens to expected absolute floor accelerations and inter-story drifts. Here, the protocol is developed for a bin of synthetic earthquakes associated to a seismic hazard (SH) with a probability of exceedance (PE) of 2% in 50 years. The proposed protocol can be scaled to obtain other SH levels using, for example, the approach recommended in FEMA 356 [e.g., Eqs.1-2 and 1-3 (FEMA 2000)].

The proposed protocol is obtained from statistical analysis of the response of a set of buildings with elastic periods varying between 0.16 and 2.6 sec and story levels ranging between 2 and 10, subjected to a bin of earthquake ground motions. The resulting protocols are calibrated to replicate average seismic demands expected on nonstructural components placed inside these typical buildings. Variables including number of cycles and time required to reach maximum displacement, inter-story drift and acceleration amplitudes are studied. Evolution with time of these parameters is statistically analyzed using "Rainflow" cycle counting algorithm (ASTM, 1997) to simplify the sequence of displacement, inter-story drifts and floor acceleration amplitudes into a set of simpler symmetric reversal amplitudes.

Main characteristics of the proposed protocol

The purpose of the testing protocol is to derive two displacement histories, one for the bottom level and one for the top level of the NCS. The time evolution of the envelope of displacement amplitudes at the bottom level of the NCS is derived from statistical analysis of the floor displacement responses. The peak amplitude of the displacement protocol at this level is selected to obtain the average acceleration demands expected in a typical multi-story building. The displacement protocol for the top level of the NCS is obtained from the displacement protocol proposed for the bottom level of the NCS combined with the expected time evolution of inter-story drifts. Thus the bottom level excitation is selected primarily to match the floor accelerations while the response of the top level is selected to impose the proper inter-story drifts between the two levels. Three alternatives for the time evolution of the excitation frequency are considered: constant, linear and parabolic frequency variation. The total duration of the loading protocol is selected to be the average bracketed duration of the earthquake records considered.

Structural model

Nine linear models of classical multi-story shear buildings (Fig. 5) are considered in this analysis, each with a fixed number of floor levels ranging from 2 to 10. A range of natural periods is selected for these structures as a function of number of levels and possible seismic resistant system using the approximate formulas in FEMA 356. Five sub-models are considered for each structure with fixed number of stories to cover the expected period range. The inter-story stiffness assigned at each level is varied while the floor mass (the same in each floor) remains constant. In the analysis, a variation of the typical story height h_n between 8 and 14 ft is considered. Fig. 6 shows the envelope of expected periods (associated to different structural systems) as function of the number of levels in the building, and the periods of the 45 structural models considered in the analysis.

Selection of earthquake records

One set of synthetic ground motion records, obtained from the Multidisciplinary Center for Earthquake Engineering Research (MCEER) database, and developed as part of the MCEER Demonstration Hospital Project, have been considered for the development of this testing protocol. The records were estimated using the Specific Barrier Model method (SBM) developed by Papageorgiou and Aki (1983) and calibrated using a stochastic modeling approach by Halldorsson and Papageorgiou (2004). In particular, the records considered are associated to a SH with a PE of 2% in 50 years for the city of Northridge, CA. The averaged values of important parameters for the bin of records are listed in Table 2. The averaged bracketed duration shown in Table 2 was calculated considering threshold accelerations equal to 0.025PGA. Fig. 7 and Fig. 8 show a comparison between displacement and acceleration histories corresponding to the bin of earthquakes considered in the analysis. Fig. 9 and Fig. 10 show the displacement and the absolute acceleration response spectra, respectively. Mean spectral values are highlighted.

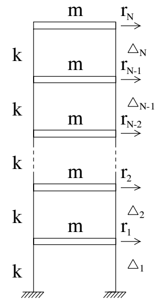

Figure 5. Multi-story building model.

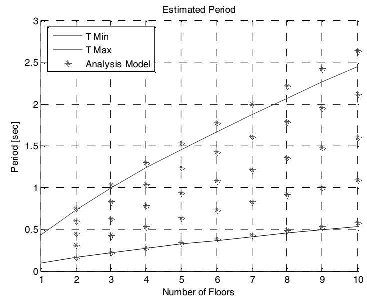

Figure 6. Variation of period with number of floors (FEMA 356).

Table 2. Averaged properties for the bin of synthetic earthquake records.

Parameter	Average Value
Bracketed Duration (sec)	19.0
PGD (in)	21.0
PGV (in/sec)	40.8
PGA (g)	0.95

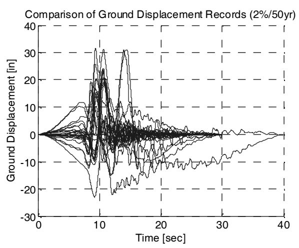

Figure 7. Ground Displacements.

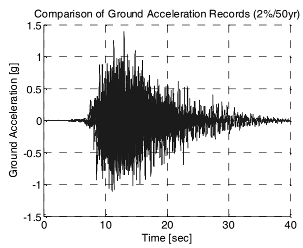

Figure 8. Ground Acceleration.

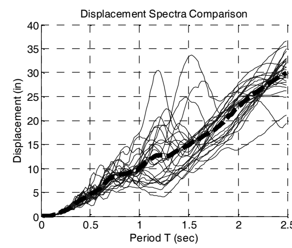

Figure 9. Spectral displacement.

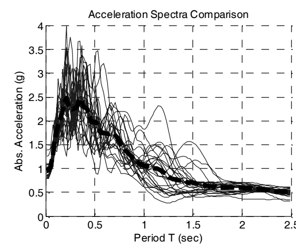

Figure 10. Spectral acceleration.

Procedure for protocol estimation

For each of the 45 structural models considered, linear response history analysis is performed using all earthquake record in the bin. Floor displacement, inter-story drift and absolute floor acceleration histories are recorded for all levels and excursion amplitude histories (positive and negative) are extracted. From each excursion amplitude history (floor displacements, inter-story drifts and absolute accelerations), pre-peak excursion amplitudes are separated and independently analyzed. It is assumed that post-peak excursions do not generate additional damage in nonstructural components. At this stage, information such as peak excursion amplitude and time required to reach the peak amplitude are saved. Rainflow counting algorithm is used to calculate range amplitudes and cycle mean values for the response quantities (ASTM, 1997). Total rainflow amplitudes r are calculated from the summation of range amplitudes r_{RA} and cycle mean values r_{MV}, as follows:

$$r = \frac{r_{RA} + r_{MV}}{2} \tag{1}$$

Finally, total rainflow amplitudes (for floor displacements, inter-story drifts and absolute floor accelerations) r are sorted in decreasing order. Amplitudes less than the 2 percent of the maximum amplitudes are neglected since it is assumed that small excursions do not produce additional damage in nonstructural components (Krawinkler et al., 2002).

Following with the analysis, all the results obtained (for all floor responses) are averaged in order to obtain an estimation of the mean evolution of demand amplitudes and average peak amplitudes. A displacement protocol $r_B(t)$ for the bottom level of the NCS is obtained from a smoothing spline curve $p(t)$ fit to the averaged rainflow floor displacement amplitude evolution.

The cyclic frequency of the signal varies with time according to one of the criteria presented in the following section. The displacement protocol for the bottom level is given by:

$$r_B(t) = \lambda p(t) Sin[\omega(t)t] \tag{2}$$

where λ corresponds to a calibration factor used to adjust the peak acceleration at the bottom level of the NCS to the averaged peak absolute floor acceleration calculated for the whole set of structural models.

A similar procedure is performed to get an adequate inter-story drift protocol. The inter-story drift protocol $\Delta(t)$ is obtained from a smoothing spline curve $p_\Delta(t)$ fit to the averaged rainflow inter-story drift amplitude evolution and a harmonic function with a time varying frequency according to one of the criteria presented in the following section.

$$\Delta(t) = p_\Delta(t) Sin[\omega(t)t] \tag{3}$$

Finally, the displacement protocol $r_T(t)$ for the top level is obtained from the summation of the displacement protocol estimated for the bottom level and the inter-story drift protocol:

$$r_T(t) = r_B(t) + \Delta(t) \tag{4}$$

Time evolution of excitation frequency

Three alternatives for the time evolution of the test frequency are evaluated: constant, linear and parabolic variations. Initially, and starting from Fig. 6, a probability density function for the fundamental structural period is estimated. Using this probability density function, expectation and standard deviation of structural period are calculated. The calculation of these values yields an expectation $E_T(T)$=1.04 sec and a standard deviation $\sigma_T = 0.52$ sec. Starting from these values, a maximum (ω_{Max}), expected ($\overline{\omega}$) and minimum (ω_{Min}) test frequency are proposed. ω_{Min} and ω_{Max} are obtained from the expected structural period plus and minus 1.75 standard deviations (σ), respectively. Doing so, the obtained values are $\overline{\omega}$ = 6.02 rad/sec, ω_{Min} = 3.22 rad/sec and ω_{Max} = 46.9 rad/sec.

As mentioned before, three alternatives are considered for the evolution of the loading frequency with time. The first and simplest case considers a protocol with constant frequency $\omega(t) = \overline{\omega}$. The second case considers a linear variation for the frequency, given by Eq. (5). Finally, the third case considers a parabolic time variation of the excitation frequency, obtained from Eq. (6):

$$\omega(t) = \omega_{Max} - \frac{\omega_{Max} - \omega_{Min}}{\overline{t}_T} t \tag{5}$$

$$\omega(t) = \frac{\omega_{Max} - \omega_{Min}}{\overline{t}_T^{\,2}} t^2 + 2\frac{\omega_{Min} - \omega_{Max}}{\overline{t}_T} t + \omega_{Max} \tag{6}$$

In Eqs. (5) and (6), $\bar{t}_T$ corresponds to the summation of the average of bracketed durations $\bar{t}_b$ of the synthetic earthquake records considered in this analysis and the time t_{Ramp} required by the initial ramp function used to start the loading process. A value t_{Ramp} = 1.5 sec is considered.

Proposed Protocol

Following the steps described in the previous sections, displacement protocols for the bottom and top level of the NCS are developed. Three possible functions for the variation of frequency with time are considered. According to the statistical analysis done, the average bracketed duration $\bar{t}_b$ of the bin of earthquake ground motions is 19.0 sec. This value is considered as the duration of the central portion of the proposed testing protocol. The central portions are symmetric with respect to $t_{Ramp} + \bar{t}_b/2$. According to the numerical analysis, the average absolute floor acceleration and the average inter-story drift expected in a typical building are 1.51g and 2.61 in, respectively. The average peak absolute floor velocity is 76.5 in/sec.

Fig. 11 shows an example of the procedure performed to obtain the averaged inter-story drift amplitude evolution corresponding to all models analyzed. A similar procedure is used to estimate the time evolution of the average floor displacement amplitudes. Fig. 12 and Fig. 13 show the proposed displacement protocols for the linear and parabolic time variation of frequency. Fig. 14 shows a comparison of the absolute acceleration and relative displacement spectrum for a linear SDOF system excited by the protocol with parabolic frequency variation and by the floor accelerations obtained from the analysis of the set of building models. Fig. 15 through Fig. 17 show the inter-story drift protocol history for the 3 alternatives of frequency time variation considered. In these figures it can be seen that the maximum inter-story drift is approximately 1.67% of the NCS inter-story height (estimated considering h=13'). The magnitude of the inter-story drift obtained using this test protocol (1.67%) is sufficiently large to damage displacement sensitive non-structural components. For example, damage in gypsum partition walls initiates at inter-story drifts of 0.82% (Restrepo et al., 2005).

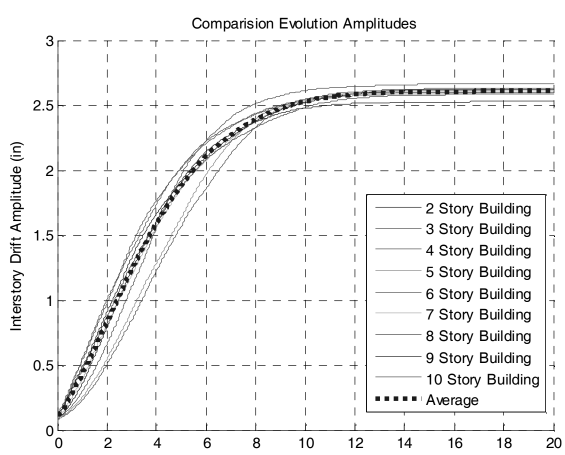

Figure 11. Average inter-story drift amplitude evolution for all model and sub-model.

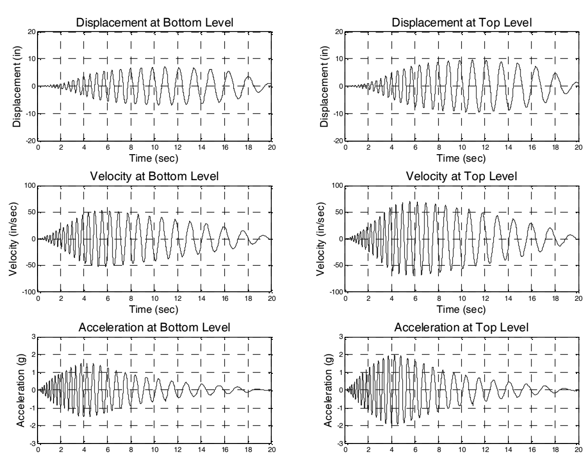

Figure 12. Proposed protocol Case 2: Linear frequency variation.

From the inspection of the loading protocols considered, the parabolic time variation of the loading frequency appears to be the best alternative. In this case, the bottom NCS floor acceleration and NCS inter-story drift histories match the expected mean seismic demand values of 1.51g and 2.61 in, respectively. The parabolic frequency variation also results in the least error in reproducing the expected absolute floor velocities (76.5 in/sec) by 12.6%. Further, preliminary studies comparing the response of nonstructural components to the protocol and actual floor motions indicate that the protocol conservatively imposes the expected floor motions demands on nonstructural components. However, as indicated by Fig. 14, the protocol demands are less than the maximum demands computed for all building models and ground motions considered.

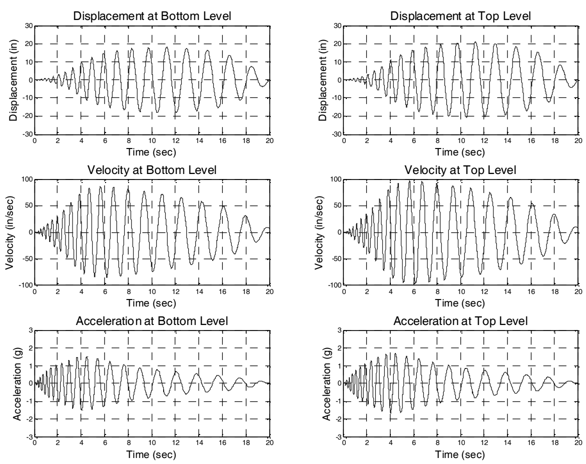

Figure 13. Proposed protocol Case 3: Parabolic frequency variation.

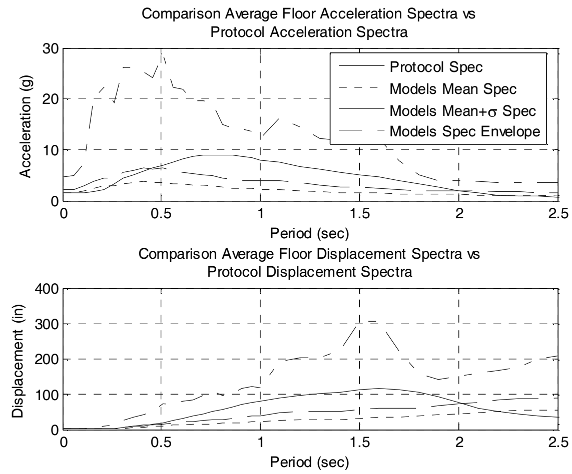

Figure 14. Comparison of response spectra for protocol Case 3 and computed floor motions.

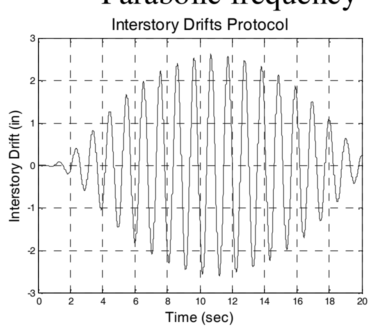

Figure 15. Inter-story drift protocol, constant frequency.

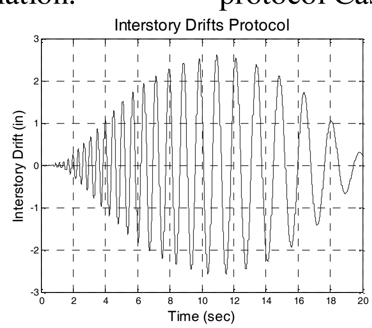

Figure 16. Inter-story drift protocol, linear frequency.

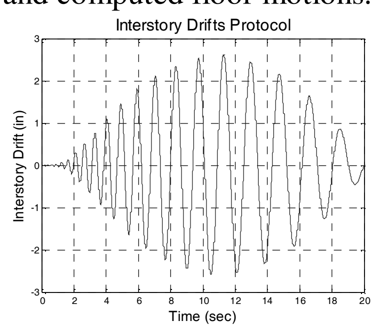

Figure 17. Inter-story drift protocol, parabolic frequency.

Conclusions

At the present, experimental testing facilities do not have sufficient capability for investigating the performance of nonstructural components during a seismic event. This deficiency is serious considering that the investment in nonstructural components and building contents is far greater compared to structural components and framing. Not surprisingly, losses from damage to nonstructural building components exceeded losses from structural damage in several recent earthquakes. To address this need, the University at Buffalo's (UB-NEES) facility is commissioning a dedicated Nonstructural Component Simulator (UB-NCS) which can subject non-structural components to realistic full-scale horizontal and/or vertical floor motions.

A dynamic testing protocol, which allows for evaluating the seismic performance and fragility of both displacement-sensitive and acceleration-sensitive nonstructural components has been developed. The proposed protocol is capable of replicating the expected average absolute floor accelerations, inter-story drifts and absolute floor velocities obtained from the time response analysis of a set of linear multi-story buildings. Current studies are examining the dynamic response of nonstructural components subjected to the developed loading protocols and a comparison of their response when subjected to actual floor accelerations. Protocols associated to seismic hazard levels other than the 2% in 50 years considered in this analysis can be obtained by extrapolating the proposed protocol using, for example, an approach similar to the recommended procedure in FEMA 356.

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