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Evaluation of Ground Motion Scaling Methods for Nonlinear Analysis of Structural Systems

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Abstract: This paper provides an overview on NSF Grant # CMMI-0928662 entitled "Evaluation of Ground Motion Scaling Methods for Nonlinear Analysis of Structural Systems." The overall objective of the project, which started in August 2009, is to assess and develop ground motion selection and scaling methods considering a wide range of structural properties, seismic hazard conditions, and hazard levels. The project tasks to date have focused on the design and construction of a small-scale reusable and reconfigurable six-story frame test specimen, test setup, measurement of the dynamic and static structural characteristics of the specimen, development of analytical models, and preliminary shake table testing. The design and construction of a nonlinear beam-column connection for the test structure is currently ongoing. The remainder of the project will involve further shake-table testing and model analysis to investigate how different site parameters and structure characteristics affect the accuracy and efficiency of different ground motion scaling methods.

1. Introduction: As performance-based considerations become pre-requisite in the seismic design and evaluation of building structures, the use of nonlinear response history analysis (RHA) has gained utmost importance (e.g., for tall or irregular structures, innovative structural systems and materials, and/or structures on soft soil). This rigorous analysis method requires, as input, a suite of ground motion records that have been selected and modified (i.e., scaled) appropriately to make them compatible with the site-specific hazard level(s) considered [e.g., Maximum Considered Earthquake (MCE) level, Design Basis Earthquake (DBE) level]. The selection and scaling of ground motion histories have a large impact on the results from the nonlinear analysis of structural systems. In fact, the ground motion record itself is undeniably the most important variable (more than the analytical model parameters) governing the outcome and amount of uncertainty from seismic design involving nonlinear RHA. Ironically, this is also the single parameter with the least guidance provided in current building codes, resulting in the use of mostly subjective choices in design.

Procedures for selecting and scaling ground motion records for a site-specific seismic hazard are broadly described in building codes and have been the subject of considerable research in recent years. The ground motion selection and scaling procedures in IBC (ICBO 2006) and CBC (ICBO 2007) are based on ASCE 7-05 (ASCE 2006). According to ASCE 7-05, the average 5%-damped linear-elastic acceleration response spectrum for the set of scaled records should not be less than the design spectrum over the period range from $0.2T_1$ to $1.5T_1$, where T_1 is the fundamental period of the structure being designed. The design value of an engineering demand parameter (EDP) — member deformations, lateral drifts, floor accelerations, etc. — is taken as the average value of the EDP if seven or more records are used in the analysis, or its

maximum value over all ground motions if the structure is analyzed for less than seven records (ASCE 7-05 requires a minimum of 3 records). These requirements for ground motion scaling are the same as those in the recently-released ASCE 7-10 (ASCE 2010) document.

To demonstrate the challenges for the reliable use of nonlinear RHA in current practice, Figure 1 shows the estimated peak inter-story drift demands (Morgen and Kurama 2008) for a six-story reinforced concrete frame structure subjected to 10 pairs of "far-fault" MCE ground motion records satisfying the ASCE 7-05 scaling requirement. The drift demands from the 10 pairs of records range from a minimum of slightly less than 1% to a maximum of almost 5%. It is clear that if only the peak demand from 3 records were used in design, as allowed by ASCE 7-05, then the design outcome (i.e., over-design, under-design, satisfactory) can be drastically altered depending on the records selected. Note that the demands in Figure 1 seem to be highly correlated with the ground motion maximum incremental velocity, MIV, plotted on the x-axis. This is further discussed in Section 4 of the paper.

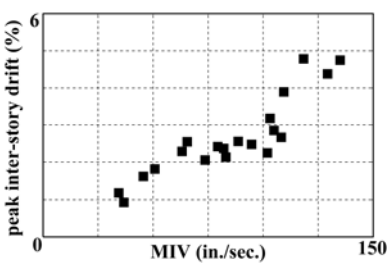

Figure 1: Peak inter-story drift demands of a frame structure (Morgen and Kurama 2008)

2. Research Motivation: The task of selecting and scaling an appropriate set of ground motion records is the single most important challenge facing practitioners required to conduct nonlinear RHAs in seismic design. The primary obstacles in front of the current engineering practice can be summarized as follows:

1) There is currently no consensus on which scaling method(s) would be best suited for use in nonlinear RHA of building structures. In particular, certain scaling methods can result in significant variability in the estimated engineering demand parameters (EDPs), indicating that the resulting EDPs may be biased depending on the ground motion records chosen. This may lead to designs with significant uncertainty and unknown margins of safety.
2) All of the previous research on ground motion selection and scaling is based on numerical analyses with no experimental verification available.
3) Furthermore, most of the research to date studying ground motion scaling has been on single-degree-of-freedom (SDF) systems, and studies on the applicability of these results to multi-degree-of-freedom (MDF) systems are limited.

3. Research Objectives: The objective of a ground motion scaling method is to provide scale factors for a small number of appositely selected records so that the nonlinear RHA of a structure under these scaled records is: (1) **accurate**, that is, it provides accurate estimates of the median EDPs as if a much larger set of records were used; and (2) **efficient**, that is, it minimizes the number of records needed to reliably obtain these accurate median EDP estimates. Although the use of a large number of records may improve the median EDP estimates, this approach is not a practical proposition since it is computationally demanding. Furthermore, the use of a large number of records does not circumvent the question on how these records should be selected and scaled to be representative for a given design scenario. Considering these issues, the current research is seeking answers to the following questions:

1) Given design specifics on the structural properties (e.g., lateral strength, period), seismic hazard conditions (e.g., site soil profile, fault distance, directivity), and hazard levels (e.g., MCE level, DBE level), how should an appropriate suite of ground motions be selected?
2) How should these records be scaled to obtain accurate and efficient estimates of median EDPs that are representative of the structure response considering a wide range of structural properties, seismic hazard conditions, and hazard levels?
3) What is the minimum number of records that should be used to retain accuracy in the median estimates of the EDPs with minimum dispersion?

More specifically, the research is:

1) Conducting a large number of small-scale shake-table experiments of reusable re-configurable (modular) nonlinear MDF structures using an exhaustive set of unscaled ground motion records as well as subsets of scaled records according to different scaling methods including, but not limited to, the ASCE 7-10 (ASCE 2010) procedure, scaling based on spectral acceleration at the fundamental period of the structure (Shome et al. 1998), scaling based on the maximum incremental velocity (Kurama and Farrow 2003), and modal-pushover based scaling (Kalkan and Chopra 2010, 2011; Kalkan and Kwong 2010).

2) Investigating how different site parameters (e.g., geological condition, fault distance, basin, and directivity effects) and structure characteristics (e.g., lateral strength, period) affect the accuracy and efficiency of the ground motion scaling methods.
3) Using the experimental benchmark results to develop design guidelines and procedures for selecting and scaling ground motion records for use in nonlinear RHA of structural systems.

4. Background: Previous research on ground motion scaling has primarily focused on "intensity-based" methods over "spectral matching." Spectral matching methods modify the frequency content or phasing of the record to match its response spectrum to a target spectrum, whereas intensity-based methods preserve the original non-stationary content of the ground motion and only modify its amplitude using one or more intensity measures (IMs) to determine the appropriate scaling factors. The earliest approach to the problem is the scaling of ground motions to match a target peak ground acceleration (PGA), which has been shown to produce inaccurate median EDP estimates with large dispersion (Nau and Hall 1984; Miranda 1993; Vidic et al. 1994; Shome and Cornell 1998). Other scalar intensity measures (IMs) such as the effective peak acceleration, Arias intensity, and effective peak velocity have also been found inaccurate and inefficient (Kurama and Farrow 2003).

None of the above IMs consider any property of the structure being designed in determining the ground motion scaling factors. Including a vibration property of the structure can lead to improved methods. For example, scaling records to a target elastic spectral acceleration, $A(T_1)$ from the code-based design spectrum or a probabilistic uniform hazard spectrum at the fundamental period of the structure, T_1, provides improved EDP estimates for structures whose response is dominated by their first mode of vibration (Shome et al. 1998). However, it has been shown that this scaling method becomes less accurate and less efficient for taller structures with significant higher mode response or for structures responding far into the nonlinear range (Mehanny 1999; Alavi and Krawinkler 2000; Kurama and Farrow 2003). To consider higher mode response, a scalar IM that combines the spectral accelerations $A(T_1)$ and $A(T_2)$ at the first two periods, as well as a vector IM based on $A(T_1)$ and the $A(T_1)/A(T_2)$ ratio have been studied (Bazzurro 1998; Shome and Cornell 1999). While this vector IM improves accuracy, it remains inefficient for near-fault records with a dominant velocity pulse (Baker and Cornell 2006).

The preceding ground motion scaling methods utilize IMs based on the elastic response of the structure, and do not explicitly consider its nonlinear response. To improve the median EDP estimates for structures with significant nonlinear response, scaling methods that recognize the lengthening of the period of vibration due to the yielding of the structure have been proposed (Kennedy et al. 1984; Malhotra 2003; Alavi and Krawinkler 2004; Naeim et al. 2004; Youngs et al. 2007). For example, a scalar IM defined as a combination of $A(T_1)$ and $A(cT_1)$, where $c > 1$ has been considered by Mehanny (1999) and Cordova et al. (2000). Similarly, scaling based on the mean elastic spectral acceleration, $A(T_1 \rightarrow T_\mu)$ over the period range $T_1 \rightarrow T_\mu$ has been investigated (Kennedy et al. 1984; Martinez-Rueda 1998; Shome and Cornell 1998; Shome et al. 1998; Kurama and Farrow 2003), where the elongated period of the structure, T_μ is calculated using the secant stiffness, k_μ, corresponding to the estimated peak lateral displacement demand, Δ_{nlin}.

As compared to the $A(T_1)$ and $A(T_1 \rightarrow T_\mu)$ methods, Kurama and Farrow (2003) showed that a scalar IM, the maximum incremental velocity (MIV, defined as the maximum area under the acceleration time-history of a ground motion between two consecutive zero acceleration crossings), may be a better indicator of the damage potential of a ground motion for a wide range of structural periods, especially for structures expected to undergo large nonlinear displacements on soft soil.

It has also been shown that some of the above scaling methods may not work well for near-fault sites (Bozorgnia and Mahin 1998; Alavi and Krawinkler 2000; Baez and Miranda 2000; Chopra and Chintanapakdee 2004). This limitation has been overcome by using the nonlinear displacement spectrum of the first-mode nonlinear SDF system for scaling (Bazzurro and Luco 2004; Luco and Cornell 2007). Required in this approach are attenuation relationships for the nonlinear SDF displacement (Tothong and Cornell 2008). Most recently, a modal-pushover-based scaling (MPS) procedure has been developed (Kalkan and Chopra 2010, 2011; Kalkan and Kwong 2010). In this method, the ground motions are scaled such that the peak displacements of the first-mode nonlinear SDF system under the scaled records match (within a specified tolerance) a target nonlinear displacement. The nonlinear SDF system properties are determined from a first-mode pushover analysis. Appropriate primarily for first-mode dominated structures, the MPS procedure has also been extended to structures with significant higher mode contributions by considering the elastic displacements of higher-mode SDF systems.

5. Research Tasks: To achieve the research objectives, the project is organized into the following five tasks:

1) Construct ground motion database, identify record subsets, and identify scaling methods.
2) Select a set of building structures, conduct pre-test analyses, and construct test specimens.
3) Conduct monotonic and cyclic pushover experiments of the test specimens.
4) Conduct shake-table experiments of the test specimens.
5) Process/synthesize results and develop recommendations, criteria, and algorithms on ground motion scaling methods, minimum number of records needed, and ground motion selection.

These tasks are described in more detail below.

Task 1 – Construct Ground Motion Database Subsets and Identify Scaling Methods: Due to lack of specific guidelines, practitioners often select ground motion records based only on distance, site conditions, and magnitude of the characteristic event expected to dominate the seismic hazard. However, many other factors, such as directivity of the rupture and basin effects contribute to the intensity and frequency content of a ground motion at a site. For example, as described in Kalkan and Kunnath (2006, 2007, 2008), "forward directivity" (with double-sided pulse) and "fling-step" (with single-sided pulse) motions with long-period, large-amplitude pulses may impose large displacement demands that require the structure to dissipate considerable energy in a single or relatively few cycles.

Similarly, there have been several cases demonstrating the significant effects of deep sedimentary basins on the seismic wave field at distances of about 50 km and more (e.g., Lee et al. 1995; Joyner 2000; Field 2000; Frankel et al. 2001). For example, during the 1999 Hector Mine earthquake, the average PGA had almost no attenuation at 130-230 km from the fault due to dominant basin waves in the Los Angeles, San Bernardino, and San Fernando basins (Graizer et al. 2002). Basin effects exhibit shaking level amplification and may also result in more complex spectral shapes with concurrent multiple peaks at lower (0.3-0.6s) and higher periods (5.0-7.0s). Significant basin amplification at long periods was recently observed in the Los Angeles basin during the 2010 M7.2 El-Mayor Cucapah earthquake.

Despite the range of factors affecting the ground motion intensity and frequency content, current practice is not equipped to fully incorporate many factors, such as directivity, basin, and duration effects in the design process. For example, structural damage under ordinary far-fault records is not only associated with the maximum deformation but also with low-cycle fatigue effects (Fajfar and Vidic 1994; Kunnath and Chai 2004; Sucuoglu and Erberik 2004; Teran-Gilmore and Jirsa 2005). In comparison, most of the damage from near-fault records (within 15 km of fault rupture) is a result of the instantaneous energy associated with intense pulse effects and a few plastic cycles (Kalkan and Kunnath 2006, 2007, 2008). In this case, low-cycle fatigue effects are less pronounced and structural damage is directly related to peak displacement demands. Current code methods do not provide a reliable basis for including these factors.

It is therefore in this project, a set of criteria and identification algorithms are being set to distinguish earthquake records based on their characteristic attributes associated with source, directivity, site, and/or basin effects (e.g., cyclic versus impulsive records; records with high, mid, or low frequency content; short or long duration records). Basin, duration, and pulse attributes of the records are investigated by frequency domain analyses; whereas directivity and fling attributes are identified from the orientation of accelerometers relative to the rupture propagation plane. These attributes are then used to categorize a large library of records [e.g., PEER Next Generation Attenuation of Ground Motions database] to facilitate the selection of the most suitable ground motions for different site-specific hazard conditions. Based on this refinement and pre-selection process, a suite of representative far-fault and near-fault records from a variety of tectonic environments are being judiciously compiled for experimental and numerical investigations. In addition to an exhaustive set covering a range of frequency content, duration, and amplitude, a series of ground motion subsets are also being created within the large set to investigate intra- and inter-set variations with regards to bias and dispersion in the median EDP estimates.

The next stage of this task will be a critical examination of the major scaling methods. The most promising as well as the most commonly used scaling methods will be studied. These methods are (but not limited to) the ASCE 7-10 (ASCE 2010) ground motion scaling procedure, scaling based on spectral acceleration at the fundamental period of the structure, $A(T_1)$ (Shome et al. 1998), scaling based on the maximum incremental velocity, MIV (Kurama and Farrow 2003), and the modal-pushover based scaling procedure (Kalkan and Chopra 2010, 2011; Kalkan and Kwong 2010). Spectral matching scaling

methods will also be studied in a comparative manner with respect to intensity-based scaling methods.

Task 2: Select Building Designs, Conduct Pre-Test Analyses, Construct Test Specimens: A major focus of the investigation to date has been the design and construction of a six-story frame specimen for testing on a medium-scale shake table at the University of Notre Dame. As shown in Figure 2, the test frame consists of a single bay with center-to-center span length of 30 in. and story height of 17 in. These dimensions, which were determined based on the size limitations of the shake table, correspond to a building length-scale of, approximately, S_L=1/10.

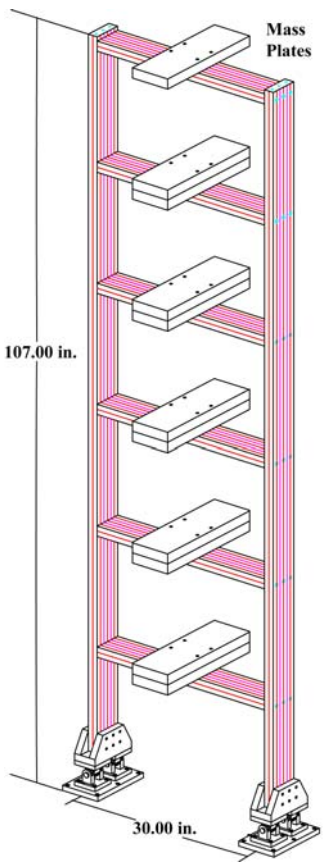

Figure 2: Six-story reusable and reconfigurable test frame

The six-story test frame was fabricated from extruded aluminum 6105-T5 alloy, with a yield strength of 35 ksi. Figure 3 depicts the frame placed on the shake table together with the measurement and out-of-plane bracing frames mounted onto the laboratory floor. The beam and column member cross-sections were determined to result in strength and stiffness appropriate with the scale model. The extruded aluminum cross-section in Figure 4, oriented in the weak direction (with moment of inertia, I=0.7097 in.^{4} and area, A=3.00 in^2) was used for all beam and column members. The column bases were designed and constructed with pinned connections to increase the flexibility of the structure. Figure 5 depicts a close-up view of the pinned base connection. Each connection consists of a steel plate bolted to the shake table top, two steel clevises bolted to the plate, and two steel eye brackets inserted into the clevises and bolted to an aluminum fixture at the column base. A tight-tolerance greased steel pin was used through the eye bracket-to-clevis connection to reduce friction while eliminating backlash effects.

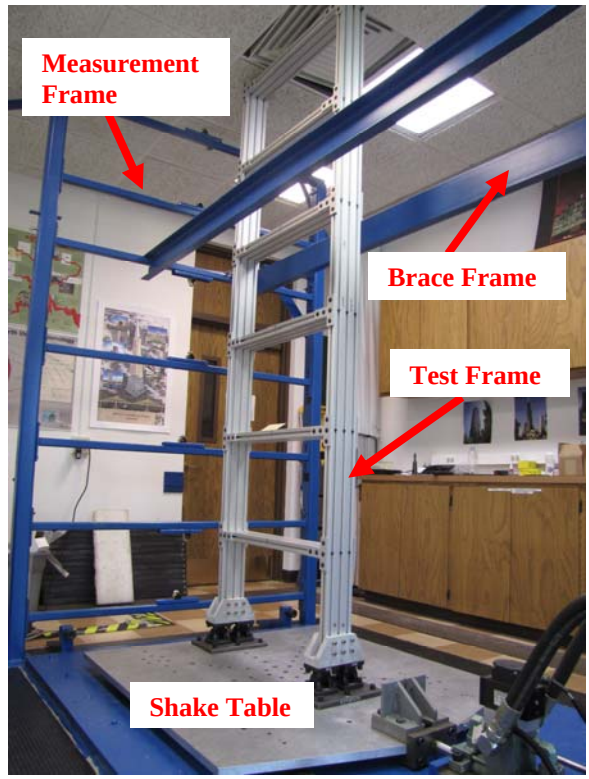

Figure 3: Test setup and frame specimen (superimposed mass plates not shown)

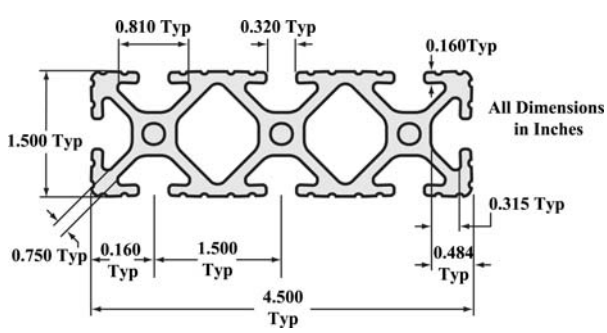

Figure 4: Extrusion profile for beam and column members (courtesy 80/20® Inc.)

To date, the project has focused on a linear-elastic test specimen with "rigid" beam-column connections.

To achieve a modular structure, each beam-column connection was constructed using three high-strength bolts passing through the column and screwing into holes tapped into the beam cross section at each end. Preliminary static and dynamic tests of the linear-elastic structure have been conducted. Analytical models of the structure have also been developed using the OpenSees (Mazzoni et al. 2009) and DRAIN-2DX programs (Prakash et al. 1993). Selected results from the testing and analysis of the linear-elastic structure are given later in the paper.

Figure 5: Pinned connection at column base

As ongoing work under this task, a nonlinear beam-column connection is being constructed to allow for the testing of nonlinear structures with different levels of lateral strength (corresponding to different response modification factors, *R*). As an important feature, the nonlinear connection will also be reconfigurable and reusable, so as to allow for the repeated testing of structures with different lateral strengths under a series of ground motion records. The design of the connection incorporates a friction-fuse concept previously investigated at the University of Notre Dame (Morgen and Kurama 2004). As shown in Figure 6, each beam-column connection consists of two components creating 10 rotational friction interfaces in between. The connection component bolted to the beam end will be fabricated out of stainless steel and the component bolted to the column will be lead-bronze (brass). Lead-bronze is a popular alloy for bearing applications under moderate loads and moderate to high speeds. The use of this alloy at the friction interfaces is desirable because the material continually "self-lubricates" when rubbing against an adjacent metal surface due to the development of a mixture of lead and its oxide at the surface of the lead-bronze alloy. By self-lubricating the frictional surface, the lead-bronze alloy helps reduce the phenomenon of stick-slip and results in a consistent value for the coefficient of friction that is relatively independent of velocity, a desirable characteristic for design and performance. The value of the coefficient of friction depends on the roughness and type of material in contact with the lead-bronze alloy, the surface treatment of the lead-bronze alloy, and the pressure applied on the contact surfaces. As shown in Figure 7 for lead-bronze against stainless steel friction interfaces, the average coefficient of friction associated with the higher excitation frequencies at f = 0.10–5.00 Hz (i.e., the kinetic coefficient of friction, μ_k) is equal to 0.198.

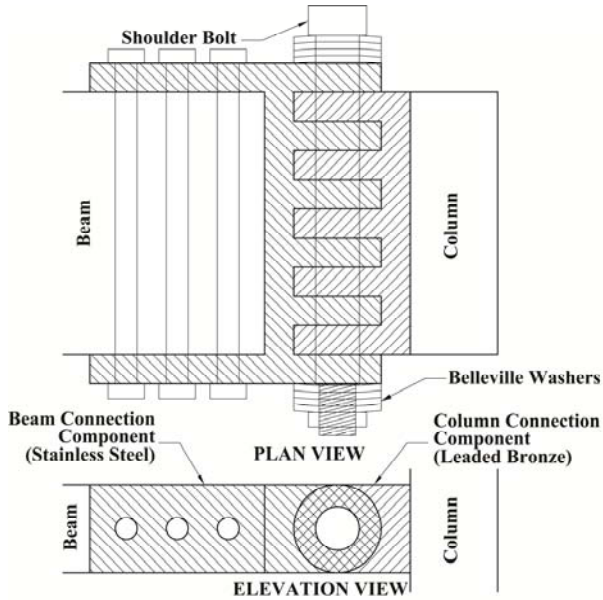

Figure 6: Nonlinear beam-column connection schematic

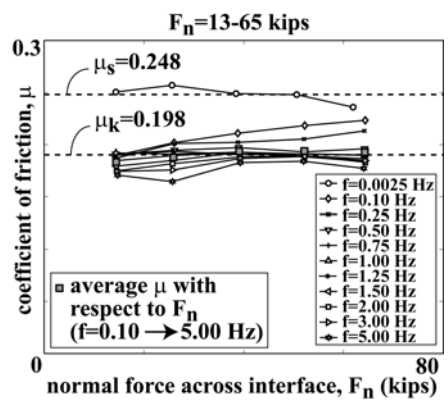

Figure 7: Coefficient of friction for lead-bronze against stainless steel interfaces

Figure 8 shows typical friction force versus slip displacement hysteresis curves at f = 5Hz (Morgen and Kurama 2004). It can be seen that the friction force can be controlled by the normal force, F_n applied on the interfaces. In the nonlinear beam-column connection, this will be achieved using a single high-strength shoulder bolt (see Figure 6) running through the 10 friction interfaces. The prototype connection was designed to produce a total maximum slip moment of about 35 kip-in. at the maximum normal force level. This slip moment was

designed to be greater than the yield moment of the aluminum beam cross-section, and thus, it will be used to simulate a linear-elastic structure (R=1). As the normal force is decreased, the connection is allowed to slip at a smaller moment, simulating structures with lower lateral strengths (i.e., R>1) and nonlinear hysteretic response. Belleville washers will be used to maintain a consistent level of normal force across the friction interfaces during each test. After the test, the bolt through each connection will be loosened, the structure will be brought back to plumb, and the connection bolts will be re-tightened to the desired level. A combination of steel shims and Belleville washers will be used to consistently achieve the desired bolt force in each test. Prior to the use of the connections in multi-story frame testing, isolated beam-column subassemblies will be subjected to static and dynamic loads to characterize the nonlinear connection and calibrate the bolt tightening operation as well as the number/arrangement of the shims and Belleville washers to result in target slip moment levels. This will ensure that the desired connection performance can be achieved with relative certainty and repeatability during the multi-story frame tests.

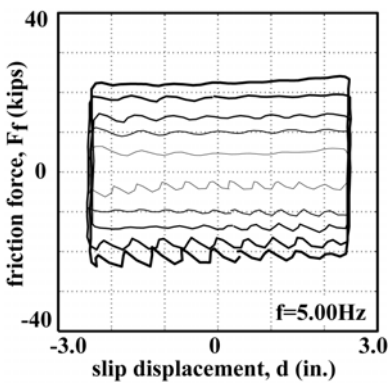

Figure 8: Typical friction force versus slip behavior (thin to thick lines: F_n = 13 to 65 kips)

Task 3: Conduct Monotonic and Cyclic Pushover Experiments: With the linear-elastic test specimen utilizing bolted "rigid" beam-column connections constructed, one of the first tasks was to identify the properties of the structure under static loading. Monotonic and cyclic pushover experiments of the structure were conducted by holding the 4th floor of the frame stationary while displacing the base laterally using the shake table. At the 4th floor level, a steel rod with pin-ended connections (Figure 9) was placed between the test frame and a relatively stiff steel loading frame. As the base of the structure was displaced, the resulting 4th floor force in the pin-ended rod was measured using an intermediary load cell. Two string pot transducers were used to measure the absolute lateral displacements at the base and 4th floor level of the structure (note that the 4th floor displacements were very small but not zero due to the deformations of the loading frame).

The 4th floor lateral force versus relative (with respect to base) 4th floor displacement results for the frame during two cycles of loading are shown using the thin black lines in Figure 10. The structure exhibited consistent and repeatable behavior in both the positive and negative loading directions; however, the onset of nonlinear behavior was observed at a relatively small load (about 500 lbs). It was found that the nonlinear behavior occurred as the beam ends lost full contact with the columns due to the stretching of the beam-column connection bolts. The flexibility of the connection bolts also reduced the initial lateral stiffness of the frame (i.e., the beam-column connections were not rigid as originally assumed). In the DRAIN-2DX and OpenSees analytical models, the flexibility of the beam-column connections was included by placing linear-elastic zero-length rotational springs at the beam ends. The stiffness of the rotational springs was determined by calibrating the model results (thick gray line) with the linear-elastic range of the measured behavior in Figure 10.

Figure 9: Pin-ended rod and load cell assembly for monotonic/cyclic pushover experiments

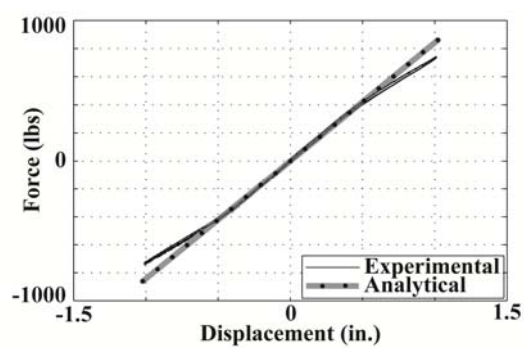

Figure 10: Measured and analytical lateral load-displacement behaviors of the linear-elastic structure

As remaining work under this task, once the nonlinear beam-column connection in Figure 6 is

constructed and characterized, the hysteretic behavior of frames utilizing these connections will be determined. The modeling of these frames will also utilize zero-length spring elements at the beam ends, but, unlike the linear-elastic structure, these spring elements will have nonlinear hysteretic behavior. The properties of the springs will be determined by calibrating the analytical results with the measured cyclic behavior of the structure, thus providing a full range of prediction tools.

Task 4: Conduct Shake-Table Experiments: Shake table experiments form the central component of this research. As shown in Figure 3, these experiments are conducted on a medium-size uniaxial earthquake simulator that consists of a hydraulic actuator/servo-valve assembly and a hydraulic power supply that drive a 4 ft by 4 ft slip table. The table can achieve a maximum acceleration of ±4 g with a 1000 lb test load at a nominal operational frequency range of 0-50 Hz. The data acquisition system is capable of recording the table displacements as well as the displacements and accelerations at each floor of the test specimen. The data is collected at a high sampling rate, resulting in close-to-simultaneous excitation and response measurements. The displacements of the structure are measured using seven free unguided LVDTs (six at the floor and roof levels and one at the base) anchored between the test frame and an isolated measurement frame. An example of a mounted LVDT can be seen in Figure 11. The clamps used to attach the LVDTs to the measurement frame were engineered to mitigate vibrations.

Figure 11: LVDT mounted to measurement frame

A series of preliminary tests were conducted to investigate if the amount of mass on the table affected its performance, especially with regards to the ability of the table to follow a given control signal. Within the expected range of mass, the table was able to accurately follow the input excitation history. To achieve the desired fundamental period of the structure, two steel plates with total weight of 92.6 lb were securely attached to the midspan of each floor beam and one plate with weight of 46.3 lb was attached to the midspan of the roof beam. Then, the structure was subjected to a series of sine sweep experiments to characterize its linear-elastic dynamic properties. A typical roof displacement response at resonance is compared to the base excitation in Figure 12. The results from three series of sine sweep tests with base excitation amplitudes ranging from 0.005 in. to 0.015 in. can be seen in Figure 13, where the y-axis shows the ratio of relative roof displacement amplitude to base excitation amplitude and the x-axis shows the frequency of the sine wave exciting the structure. While there is some amplitude dependency, it can be seen that the fundamental frequency of the structure is around f_1=4.25 Hz, corresponding to a period of T_1=0.235 s. With some guidance from the OpenSees analytical model, the second mode of vibration was measured to occur at around f_2=16.1 Hz. The measured mode shapes for the first two modes of vibration can be seen in Figure 14. Using the log decrement method on the decay of the displacement response at the roof level, the damping ratio of the frame was determined as ξ=1.14%. The time-history of the roof displacement response used in this calculation can be seen in Figure 15.

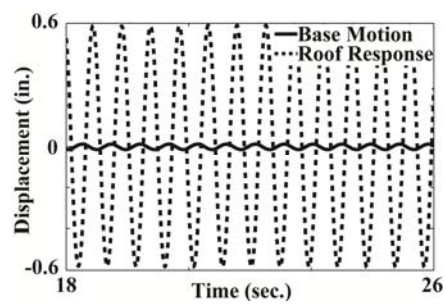

Figure 12: Roof displacement and base excitation at resonance

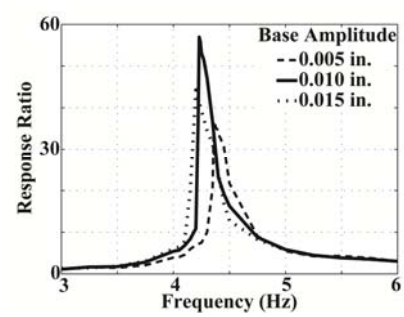

Figure 13: Resonance peaks from three series of sine sweep tests

The above shake-table tests are some of the first dynamic experiments that were conducted on the linear-elastic frame specimen. Concurrently, a subset

of earthquake ground motion records from Task 1 was selected and the structure's response was compared with the estimated response from the OpenSees analytical model as another method of model calibration. Estimated versus measured roof displacement response comparisons from the 1987 Superstition Hills and 1978 Tabas ground motion records reduced to half of full intensity (as a safety precaution during these preliminary tests) can be seen in Figures 16 and Figure 17, respectively. It should be noted that since the table is controlled using displacement rather than acceleration, each base acceleration time-history was integrated twice with respect to time. Furthermore, to achieve dynamic similitude for the small-scale test specimen, the ground motion time-histories were modified using a time scale of S_T=1/3 and a length scale of S_L=1/10. With the selected time scale, the test specimen corresponds to a full-scale structure with a fundamental period of about T_1=0.705 s. In the remainder of this task, an array of structures with different lateral strengths (by varying the slip moment of the nonlinear beam-column connections) and fundamental periods (by varying the superimposed weight plates) will be subjected to the full suite of ground motion records from Task 1, thus developing the experimental database needed to achieve the proposed research objectives. These results will also be used to further validate and calibrate the analytical models.

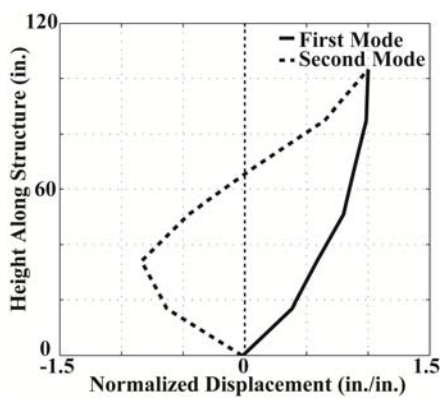

Figure 14: First and second mode shapes

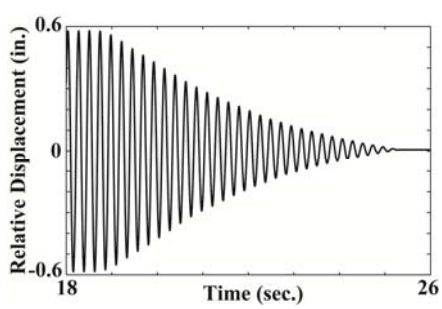

Figure 15: Decay of roof displacement response

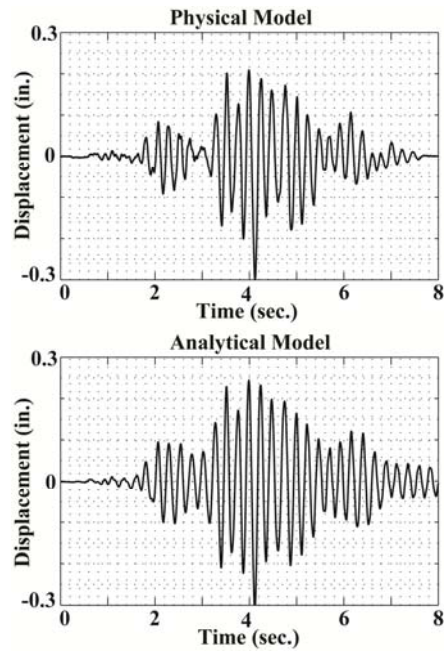

Figure 16: Measured and estimated roof displacement responses under the Superstition Hills record reduced to half intensity

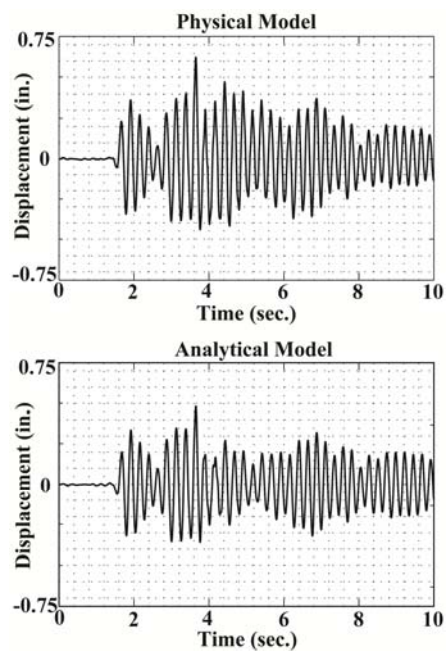

Figure 17: Measured and estimated roof displacement responses under the Tabas record reduced to half intensity

Task 5: Process/Synthesize Results and Develop Seismic Design Recommendations: In this final task, the project results will be processed and synthesized to develop a framework for ground motion selection and scaling. The engineering design parameters determined from the shake-table tests in

Task 4 (i.e., floor displacements and accelerations, interstory drifts) will be used to investigate the effectiveness of the different scaling methods in reducing the statistical dispersion in the seismic demands while also minimizing the number of records needed to obtain accurate estimates of the median demands. This will establish the reliability of the resulting engineering demands for nonlinear analysis and ways to reduce the associated uncertainties in seismic design. Furthermore, testing of structures with different properties and under ground motion records representing different seismic hazard conditions and hazard levels will allow the project to determine how the effectiveness of the ground motion scaling methods changes with these variables. The experimental results will be compared with analytical predictions, and the validated analytical models will be used to expand the data into additional design and seismic loading scenarios. The advantages and shortcomings of each scaling method will be determined systematically by considering a matrix of structures and their seismic demands. This evaluation will also include an assessment of the criteria and identification algorithms used in the selection of the ground motion records based on their characteristic attributes.

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