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SEISMIC PERFORMANCE OF A FULL-SCALE SOFT-STORY WOODFRAMED BUILDING WITH ENERGY DISSIPATION RETROFIT

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ABSTRACT

Seismic damping systems have been shown to be effective in reducing the structural response of steel and concrete structures subjected to earthquakes. However, the successful application to light-framed wood structures remains a challenge due to a number of factors including the inherent flexibility of wood framing connections that leads to losses in displacement transfer between the wood framing system and the damper assemblies. Within the framework of the NEES-Soft project, such implementation issues have been investigated through the shake table testing of a full-scale, four-story woodframe building at the NEES-UCSD site. All dampers are installed in toggle-braced light steel frames that are located at the soft ground story so as to avoid construction-related disruptions to upper-story residents. The location of the damper frames in plan and the amount of supplemental damping introduced are strategically selected in order to achieve a high level of structural performance for the design ground motions. The damper-structure connections are carefully designed to reduce possible displacement transfer losses. In this paper, selected experimental test data is presented to demonstrate the degree to which energy dissipation devices are effective in protecting soft-story woodframe structures from earthquakes. Attention will be given to the displacement transfer losses between damper assemblies and wood structures.

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Seismic damping systems have been shown to be effective in reducing the structural response of steel and concrete structures subjected to earthquakes. However, the successful application to light-framed wood structures remains a challenge due to a number of factors including the inherent flexibility of wood framing connections that leads to losses in displacement transfer between the wood framing system and the damper assemblies. Within the framework of the NEES-Soft project, such implementation issues have been investigated through the shake table testing of a full-scale, four-story woodframe building at the NEES-UCSD site. All dampers are installed in toggle-braced light steel frames that are located at the soft ground story so as to avoid construction-related disruptions to upper-story residents. The location of the damper frames in plan and the amount of supplemental damping introduced are strategically selected in order to achieve a high level of structural performance for the design ground motions. The damper-structure connections are carefully designed to reduce possible displacement transfer losses. In this paper, selected experimental test data is presented to demonstrate the degree to which energy dissipation devices are effective in protecting soft-story woodframe structures from earthquakes. Attention will be given to the displacement transfer losses between damper assemblies and wood structures.

Introduction

Low-rise soft-story woodframe residential buildings, which have a structural weakness due to large openings in their perimeter wall lines and to low, relative to upper levels, interior wall density at the ground level, represent a significant percentage of the building stock in San Francisco, California and some other high-seismicity regions in the Western United States. In addition, the shear walls at ground level in such soft-story buildings are irregularly distributed in plan and thus the entire structure is prone to extensive twisting during earthquakes. A commonly recommended approach for seismic retrofit is to stiffen/strengthen the soft ground story in order to limit the structural deformation that is mainly concentrated at the ground level. However, this approach can lead to an over-retrofitted condition whereby damage is propagated to the upper stories. As an alternative retrofit approach, energy-dissipation devices, such as fluid viscous dampers (FVDs), can be installed in the ground story. Numerical simulations for such a retrofit

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indicated that such devices can reduce the maximum structural response at the soft-story level while avoiding any significant increase in forces in the upper stories. In spite of strong theoretical and analytical evidence of the beneficial effects of such devices, implementation of a supplemental damping system for retrofit of soft-story buildings in North America has been virtually nonexistent. This is due to various impediments including: 1. Difficulty with installing dampers within the confines of wood framing members; 2. The need to integrate the damper bracing system within wood structures without any major displacement transfer losses; 3. The inherent damping in woodframed buildings (which may be high but also partially associated with inelastic behavior) rendering a supplemental damping retrofit less appealing as compared to steel and concrete structures; 4. A lack of experimental test data at the system level to validate the feasibility and effectiveness of a damper retrofit; and 5. The absence of knowledge regarding soft-story building behavior and the associated collapse mechanism.

Relatively few studies have been conducted on evaluating the application of supplemental damping devices in wood-frame structures. Symans et al. presented an exhaustive literature review summarizing the application of seismic protection systems to wood-framed structures, including numerical modeling and physical testing of different types of damper devices integrated in wood shear walls [1], among which, the suitability of viscous fluid dampers for seismic protection of light-framed wood buildings is numerically investigated in detail by the same leading author [2]. The aforementioned studies demonstrate that dampers are capable of providing a reliable, non-degrading source of energy dissipation at the shear wall level, while their behavior at the system level remains unclear. To fill the gap in knowledge, a full-scale, two-story woodframed townhouse, retrofitted via implementation of a modular damper wall, was tested on a shake table as part of the NEESWood project [3]. The test results showed that the engagement of the dampers was limited, mainly due to the inherent flexibility at damper-wood connections. However, the feasibility of the damper retrofit was evident. To improve the interaction between the damper assemblies and the wood framing components, a new design for the modular damper walls with toggle-braced linear fluid viscous dampers was developed and tested at the shear wall level [4]. It was shown that the toggle-braced assembly was capable of compensating for the displacement transmission losses to some extent by its displacement amplification properties, while its performance at the system level hadn't been physically demonstrated. To the authors' knowledge, the research presented herein represents the first successful application of a damping system within a full-scale wood-frame building with a soft ground story.

Test Specimen Configuration

The test specimen, constructed and tested on the NEES@UCSD outdoor shake table, was designed to replicate the structural and aesthetic characteristics of typical soft-story wood-frame construction in San Francisco, CA that was built in the 1920's to 1930's [5]. The designed four-story building, with plan dimensions of 24 ft × 34 ft, featured a large open space at ground level and two two-bedroom apartment units in each upper story (see Fig. 1). The exterior façade of the building was sheathed with horizontal wood siding (HWS) and the interior with gypsum wall board (GWB). The measured building weight was approximately 134 kips which was very close to what was used in the numerical model.

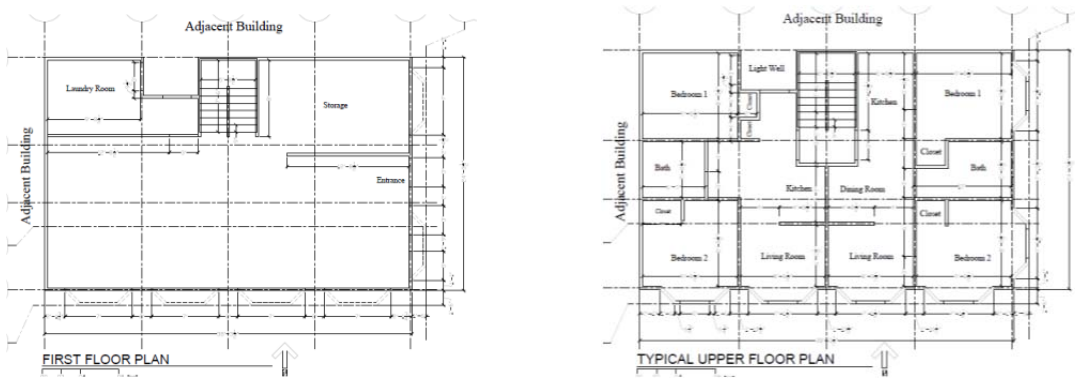
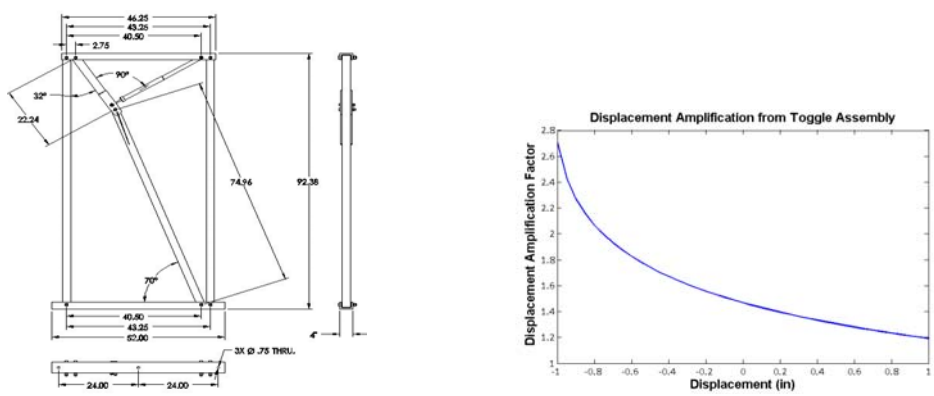


Figure 1. Floor plans of UCSD test specimen: ground floor (left) and upper floors (right)

Supplemental Damping Assembly

Fluid viscous dampers installed in toggle-braced framing systems were selected for implementation in the structure (see Fig. 2(a)). Each linear fluid viscous damper had a damping coefficient of 0.26 *kip-sec/in* which, based on the geometry of the toggle-braced framing, is amplified according to a displacement amplification factor f = 1.57 [this is an average value (in a least square sense) over a displacement range of ± 1 *in*). The damper has a displacement capacity of 4.04 *in* in extension and 2.02 *in* in compression, which corresponds to lateral displacement of the top of the frame of about 1 *in* to the left and 2 *in* to the right (see Fig. 2(a)) relative to the bottom of the frame. The ultimate force capacity of the damper is 4 *kips*.



(a) **(b)**

Figure 2. Toggle-braced damper assembly: (a) geometry and (b) displacement amplification

Unlike other commonly used framing assemblies, the displacement amplification provided by a toggle-braced assembly is highly nonlinear with larger amplification occurring when the damper is extended (damper extension corresponds to negative inter-story displacement in Fig. 2(b)). Given the displacement dependency of the amplification factor on top channel displacement, the lateral force exerted on the damper assembly is given by:

$$F = f(f + uf')C_0\dot{u} \tag{1}$$

where C_0 is the damping coefficient of a single damper, u and $\dot{u}$ are the inter-story displacement and velocity, respectively, and f' is the first derivative of the displacement amplification factor

with respect to the displacement. The abovementioned nonlinearity has been numerically modeled and experimentally validated in a real-time hybrid simulation of a stacked wood shear wall [6]. It is believed by the authors that a simplified analysis, where the displacement-dependency of the displacement amplification is ignored, is sufficient to capture the overall performance of the damper assembly. Thus Eq. 1 can be rewritten to the following form which is the same as that presented by Sigaher and Constantinou [7]:

$$F = C_0 f_{avg}^2 u \tag{2}$$

where f_{avg} is the average displacement amplification factor (equal to 1.57).

Damper Location

Supplemental damping was introduced solely in the ground story so as to avoid construction-related disruptions to upper-story residents. Given the extreme soft-story condition of the test specimen and the objective of achieving a high level of performance for design earthquakes, the upper-stories were also retrofitted. In the upper stories, plywood shear walls were utilized at selected locations with different wall lengths and nail patterns depending on the location of the wall in plan and the floor level in elevation. The design of the upper-story retrofits was consistent with the test phase prior to the tests with damper implementation (due to budget and test schedule constraints).

Based on a previous parametric study conducted by the authors [8] [9], nine damper assemblies were strategically distributed along the perimeter of the test building at ground level to minimize both the translational and torsional response of the building. Since the shake table at NEES@UCSD was excited in a uniaxial direction, seven damper assemblies were located along the longitudinal direction (longer dimension in Fig. 1) of the building (the direction of shaking) and two dampers were located along the transverse direction (see Fig. 3). The damper assemblies along the soft front wall lines were originally designed for another application and thus could not fit within the small space defined by the narrow wall piers. Thus, to avoid unnecessary additional fabrication work, the four dampers along the front wall line were located directly in the middle of the garage door openings. This placement of the dampers was deemed acceptable since the primary purpose of the damper retrofit was to demonstrate the effects that such a retrofit would have on the building, which is largely independent of the location of the dampers along a given line of resistance. A separate study is underway to develop/investigate narrow damper frames that could fit in the available wall space and thus would be better suited for practical application to soft-story buildings.

Data Analysis

Extensive signal processing has been conducted for extracting desired quantities from the measured data (a total of 333 sensors were installed in the test building). The performance of the structure was evaluated for seven white noise tests and five seismic excitations associated with the test phase wherein the building was retrofitted with damping devices, as well as the test phases prior to and after the damper test phase (see Table 1 for the numbering of tests in the damper test phase). Detailed information about different test phases can be found in reference [5]. This section concentrates on the data associated with evaluating the effectiveness of supplemental damping in protecting the soft-story wood-frame structures. Attention will be given

to the system identification technique employed, the maximum inter-story drift ratios which were shown to provide a good indication of structural damage, and the displacement transmission losses between damper assemblies and the wood framing system.

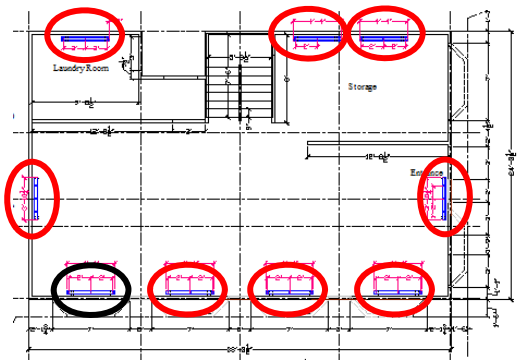


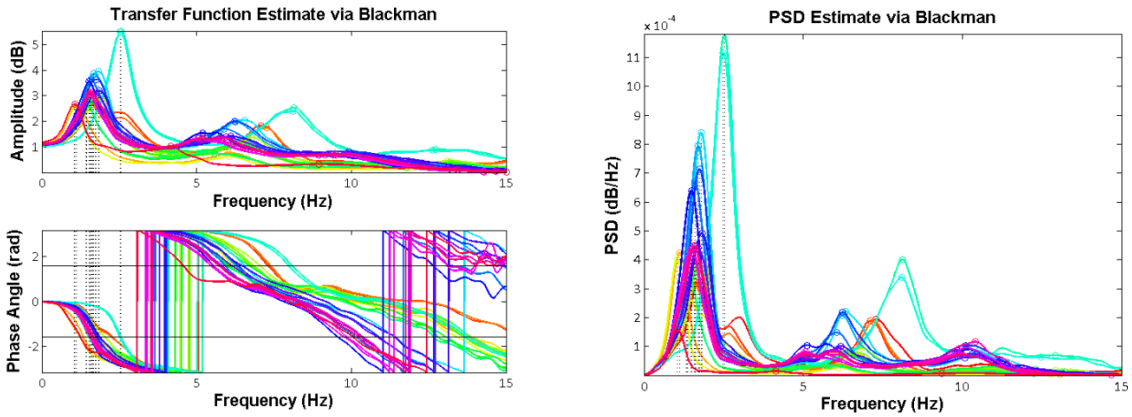
Figure 3. Ground floor plan distribution of damper assemblies

Table 1. Numbering of white noise and seismic tests in damper test phase

WN Test ID	Test Phase	Seismic Test ID	EQ Record	PGA (g)	S_a (g)
T	DAMPER_PBSR_14	14	G03000	0.269	0.5
U					
V	DAMPER_PBSR_15	15	G03000	0.645	1.2
W	DAMPER_PBSR_16	16	RIO360	0.596	1.2
X	DAMPER_PBSR_17	17	G03000	0.967	1.8
Y	DAMPER_PBSR_18	18	RIO360	0.894	1.8
Z					

System Identification

The main objective of the system identification tests was to determine the dynamic properties of the test structure (modal frequencies, mode shapes, and modal damping ratios). Uniaxial white noise tests were conducted with a Root-Mean-Square (RMS) acceleration amplitude of 5% g. The sampling rate of the recorded data was 256 Hz for shake table sensors and 250 Hz for all other sensors. The duration of each white noise test was about 5 minutes. Both the Transfer Functions (TFs) obtained from the acceleration response at the roof CM (see Fig. 4(a)) relative to the base (shake table) motion and the power spectral density functions (PSDs) from accelerometer measurements at the same location (see Fig. 4(b)) were used to detect the modal frequencies. A windowing function and averaging method was utilized to smooth the white noise data so as to improve the ability to identify the modal properties. Apparently, there is a sensitive tradeoff between the spectral leakage protection (resolving disparate strength components with dissimilar frequencies) and the frequency resolution (resolving comparable strength components with similar frequencies) for the Fast Fourier Transform (FFT) of acceleration time-history data. Since the authors believe that the lower modal frequencies of the test building are well separated, the Blackman windowing function was finally selected due to its leakage-protection capability.



(a) **(b)**

Figure 4. Transfer functions (left) and power spectral densities (right) of all WN tests

Depending on the inherent dynamic characteristics of the structure, the level of difficulty in determining the modal properties from system identification tests can vary. Thus far, the first three modal periods have been identified in this study (see Fig. 5). Work is underway to determine the associated mode shapes and modal damping ratios based on data from the 90 accelerometers located in the test building. As can be seen in Fig. 5, white noise tests are labeled alphabetically through all the test phases (damper retrofit is associated with test phase 4 and corresponding to the white noise tests T to Z (see Table 1). A direct comparison of the modal periods between the test phases cannot be made since each retrofit modifies the dynamic characteristics of the structure. However, by observing the large increase in fundamental period during phase 3 and the overall steady growth of the associated higher modal periods, it is suspected that there may have been an accumulation of structural damage prior to the damper test phase. The associated loss in stiffness may have been difficult to fully recover via simple repair work. Extra work is needed to calibrate the numerical model according to the actual condition of the building before each seismic test as indicated by the modal properties.

Seismic Test Results

Two recorded ground motions from different earthquakes, representing 10% and 70% probability of non-exceedence out of 22 pairs of far-field ground motions from FEMA-P695, were scaled to three different intensity levels (0.5g, DBE and MCE) according to ASCE 7-10 scaling procedure over the range of 0.02 to 1.50 seconds and used as the shake table input motions. The two motions were the Gilroy record (0 degree component) from the 1989 Loma Prieta, CA earthquake and the Rio record (360 degree component) from the 1992 Cape Mendocino, CA earthquake (see Fig. 6 for their response spectra). Due to space limitations, only selected results are presented herein. The data from accelerometers was used to determine the absolute displacements at each floor level and the inter-story drifts (note that the approach used to compute these quantities is only valid if shearing deformations dominate the overall structural behavior [11], which is the case for the lower stories where the seismic weight from above prevents the story from overturning). The measured building deformations (amplified for clarity) at a certain instant in time and for each earthquake are shown in Fig. 7.

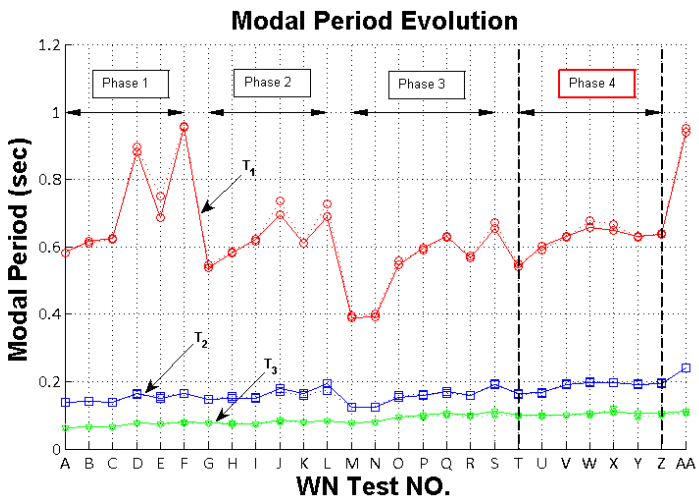


Figure 5. Evolution of the first three modal periods

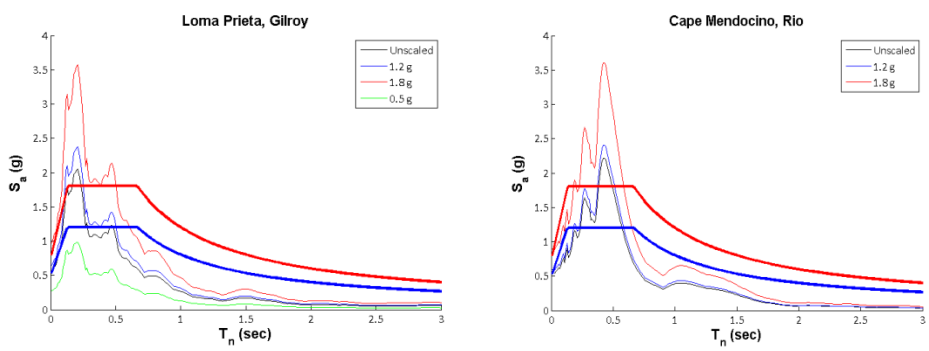


Figure 6. Scaled response spectra of test motions

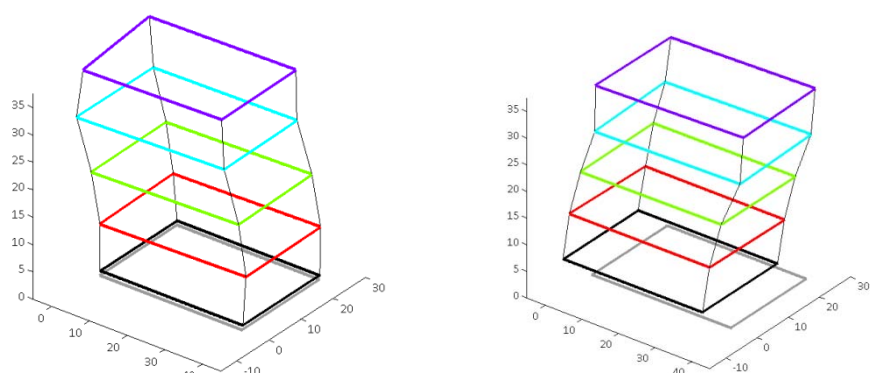


Figure 7. Time capture of building response to Loma Prieta earthquake (left) and Cape Mendocino earthquake (right)

Figure 8 shows the maximum absolute inter-story drift (in inches) of the ground story as measured by the accelerometers at the corners (in red) and the diagonal string pot readings after trigonometric conversion (in blue). Good correlation is achieved between the sensors;

measurements from the accelerometer and the string pot along the same wall line are within a 10% difference. Also, it can be seen that for this MCE level of excitation, the supplemental damping results in peak drifts in the soft story drift ratio that are at about 1% in the direction of ground shaking. Selected drift time-histories in the direction of ground shaking from two types of sensors that were mounted adjacent to each other are shown in Fig. 9, again demonstrating good correlation between the sensor measurements.

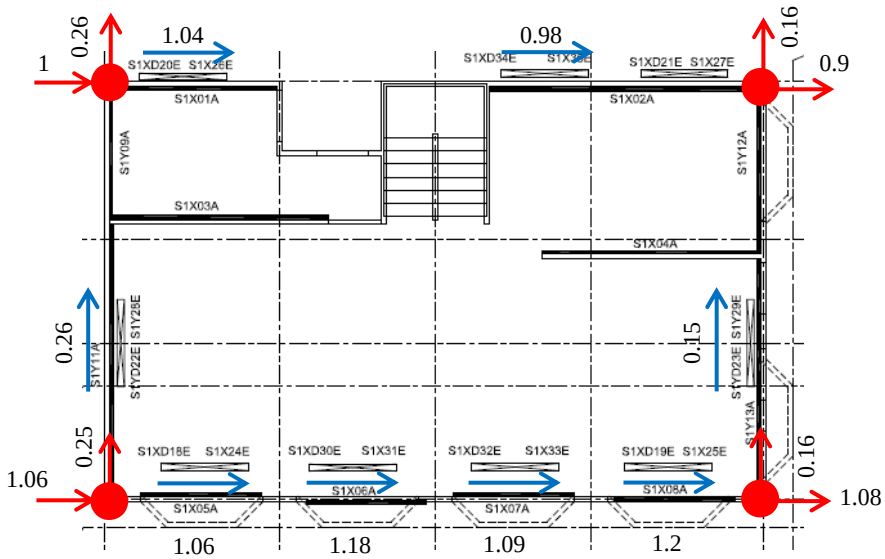


Figure 8. Correlation of sensor measurements (Loma Prieta, G03000, MCE level)

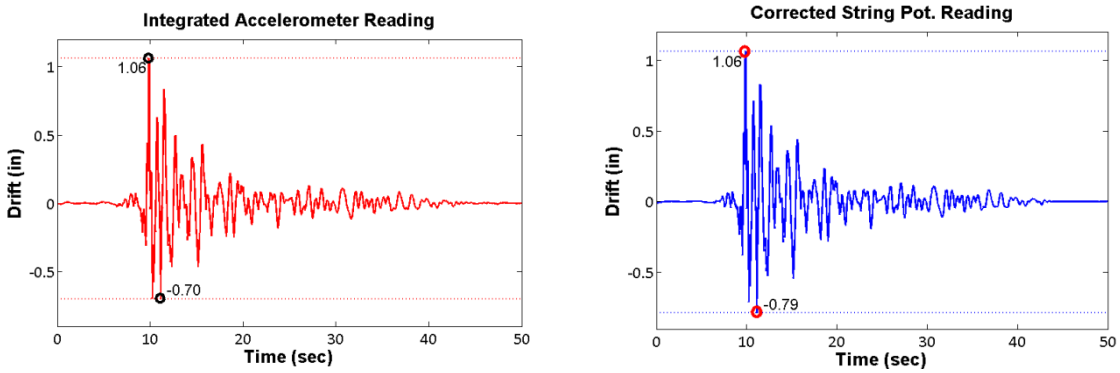


Figure 9. Typical drift time-history from accelerometer (left) and string pot (right)

Another issue to investigate is the efficiency in transmitting displacements from the wood framing components to the damper assemblies. This can be achieved by examining the data from linear pots mounted horizontally (for slippage measurements) and vertically (for uplift measurements) (see Fig. 10). Data accessed from a typical damper assembly (circled in black in Fig. 3) from seismic test 17 (see Table 1) is presented in Fig. 11. It is evident that the damper-structure connections were designed well in that they resulted in nearly pure shearing deformation of the frame (uplift on both ends of the frame lower channel are minimal). Also, lateral slippage at the upper and lower channel connections was on the order of 10^{-3} *in.*, which is negligible. Overall, the damping devices performed as expected, enabling the complete retrofit to

achieve the desired performance objective. More comprehensive data analysis will be presented in future publications.

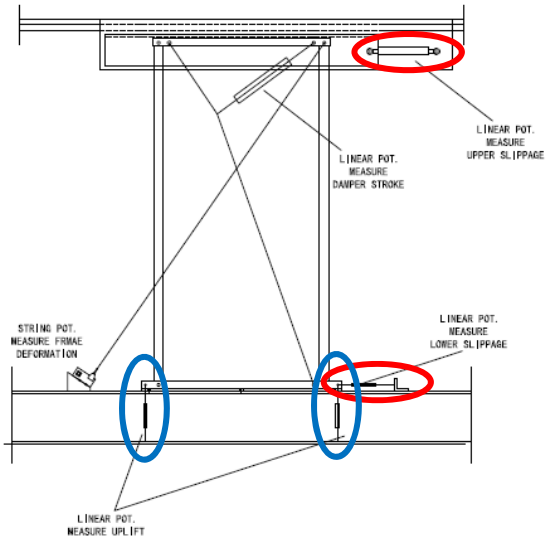


Figure 10. Typical damper instrumentation

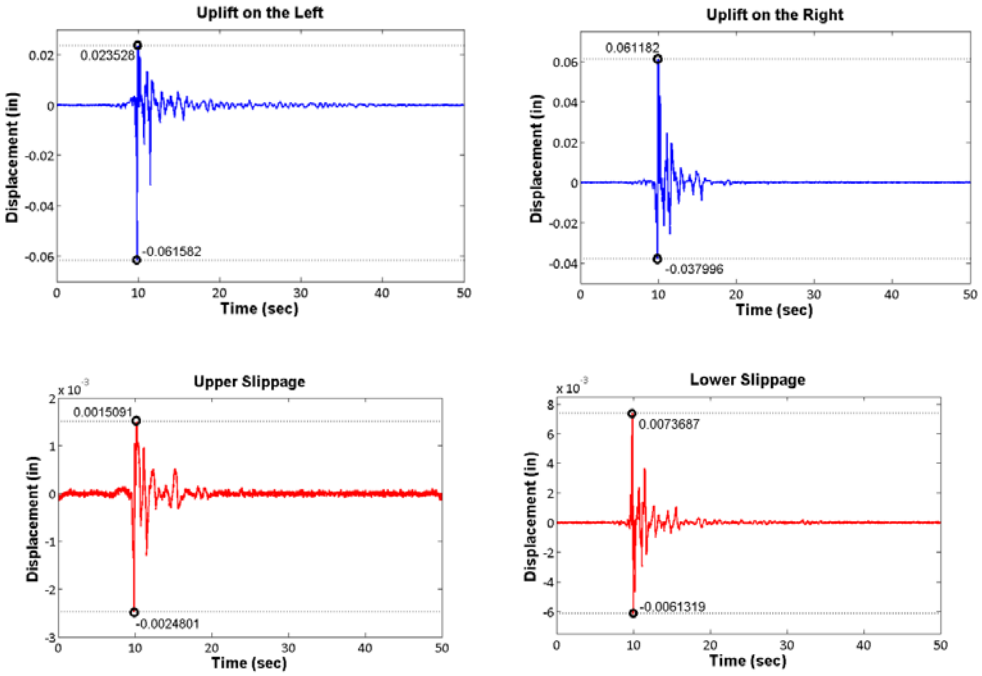


Figure 11. Toggle frame uplift (upper) and slippage (lower) measurements

Conclusions

A supplemental damping system was implemented in a full-scale, soft-story, wood-framed building and successfully tested on a shake table. The test results indicate that the structure response was generally in accordance with numerical predictions, thus validating the numerical model. System identification tests revealed damage accumulation, thereby indicating the need to calibrate the numerical model for proper interpretation of the results. Furthermore, the

performance of the retrofitted structure was consistent with the target performance, indicating that the design of the damping system, including damper location for simultaneous translation and torsion resistance, was adequate to meet a high performance objective.

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