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Full-Scale Experimental Verification of Soft-Story-Only Retrofits of Wood-Frame Buildings using Hybrid Testing

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The FEMA P-807 Guidelines were developed for retrofitting soft-story wood-frame buildings based on existing data, and the method had not been verified through full-scale experimental testing. This article presents two different retrofit designs based directly on the FEMA P-807 Guidelines that were examined at several different seismic intensity levels. The effects of the retrofits on damage to the upper stories were investigated. The results from the hybrid testing verify that designs following the FEMA P-807 Guidelines meet specified performance levels and appear to successfully prevent collapse at significantly higher seismic intensity levels well beyond for which they were designed. Based on the test results presented in this article, it is recommended that the soft-story-only retrofit procedure can be followed when financial or other constraints limit the retrofit from bringing the soft-story building up to current code or applying performance-based procedures.

Keywords Soft-Story; Wood-Frame; Full-Scale Test; Retrofit; FEMA P-807; Hybrid Testing

1. Introduction

Wood-frame construction constitutes the majority of the building stock in North America. These structures include residential single-family and multi-family dwellings, and low-rise commercial buildings. Many of the multi-story buildings are equipped with tuck-under

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parking garages or simply large openings on the first level for parking or commercial space. Unfortunately, the large openings create a weak- and soft-story with a much lower strength and stiffness than the upper stories resulting in a highly vulnerable structure prone to collapse at its soft story.

The U.S. Federal Emergency Management Agency (FEMA) P-807 Guidelines describes a soft-story building as one with "80% open area on one first-story wall or more than 50% open area on two adjacent first-story walls" [FEMA, 2012]. This openness is characteristic of multi-story wood-frame buildings, typically three-, or four-, stories in which the first story uses columns or short wall segments for supporting the gravity load. This type of structure is common for mixed-use buildings and more often for multi-unit apartment buildings which have large parking garages or commercial space on the first story and many units on the upper stories. Often, the soft-story was designed on a different grid with significantly fewer interior wall partitions than the upper stories. This results in a discontinuous load path into the foundation and distinct differences in lateral strength and stiffness between the first and upper stories. During an earthquake, the upper stories act as a rigid box while the first story experiences large translational and rotational displacements making the building susceptible to collapse.

Thousands of structurally deficient soft-story wood-frame buildings have been identified in California and the situation has been recognized as a disaster preparedness problem. Specifically, the 4,400 soft-story buildings identified in San Francisco during the Community Action Plan for Seismic Safety (CAPSS) project were deemed to be structurally deficient and generally were buildings constructed prior to 1974 with outdated or no seismic provisions considered in the design [ATC, 2010]. In San Francisco alone, there is less than 3% rental vacancy and over 58,000 residents living in these structurally deficient buildings, further underscoring the need for retrofit. Mitigation efforts throughout the state to rehabilitate soft-story wood-frame buildings have been encouraged but only the city of San Francisco has, to date, issued a mandate. There are numerous engineering challenges associated with soft-story buildings of this type. Modern construction uses wood structural panels as sheathing to create a stiff wall assembly that provides a high racking strength against lateral deformations [van de Lindt, 2004]. The wall sheathing material used in buildings constructed from the 1920's through the 1960's consisted of brittle materials such as plaster on wood lath and stucco, horizontal wood siding made up of dimension lumber planks, and diagonal bridging which is easily over-stressed. Due to the age of these buildings, they have often been remodeled or renovated by either adding layers of non-structural finishes such as gypsum wallboard (GWB) to the walls or replacing portions of the wall assemblies. The soft-story condition is exacerbated by the lack of shearwalls on the first story and the poor (and often unpredictable) performance of the archaic and non-uniform assemblies on the upper stories making it difficult to design a code-level retrofit using conventional approaches in many cases. FEMA P-807 sought to rectify this by reducing the uncertainty in the retrofit design through a process that resulted from a combination of data analysis and expert elicitation.

1.1. The FEMA P-807 Guidelines

Following the damage observed to multi-unit multi-story wood-frame buildings caused by the 1989 Loma Prieta earthquake and wood-frame apartment buildings with tuck-under parking damaged by the 1994 Northridge earthquake, FEMA began the Applied Technology Council (ATC) Project 71.1 which would eventually result in the FEMA P-807 Guidelines for retrofitting such buildings. The FEMA P-807 Guidelines were developed to aid practicing engineers in retrofitting soft-story wood-frame buildings in

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a cost-effective and practical manner for quick and consistent implementation. Within the FEMA P-807 Guidelines, the retrofit is to be constrained to the soft-story with the intention of limiting disruption to the building's occupants. The soft-story-only retrofit must be adequate enough to prevent the building from collapsing at the first story while not being too stiff and strong such as to potentially collapse the upper stories by driving the earthquake forces upward. The FEMA P-807 Guidelines were the first to take into account the strength provided by existing non-structural walls and assumes that the strength of the upper stories is adequate enough to match the retrofitted first story.

In order to use the FEMA P-807 Guidelines, the soft-story wood-frame building must meet specific criteria set forth by the FEMA P-807 Guidelines for a soft-story-only retrofit. A simplified evaluation procedure is available for qualifying soft-story buildings, and a more detailed evaluation procedure is available for buildings that meet the more restrictive qualifications. The detailed evaluation procedure may be executed using the freely downloadable weak-story tool from FEMA's website, or by following the procedure described in the FEMA P-807 document. Rather than requiring the designer to perform labor-intensive and costly nonlinear modeling of a specific wood-frame building, the FEMA P-807 methodology uses hundreds of previously analyzed surrogate models as a statistical representation of qualifying buildings. In order for a building to be properly retrofitted based on the surrogate models, details about the building floor plan, diaphragm assemblies, building materials and condition, and connection hardware and location are all required. The building's seismic weight is calculated and used to develop load-drift curves and load-rotation curves based on all of the above gathered information. The FEMA P-807 Guidelines do not provide locations for the retrofits within the building plan, but provide the required additional strength and stiffness for mitigating the soft-story and a maximum eccentricity requirement created by the location of the retrofits. The building owner and other stakeholders can set specific performance objectives for the retrofitted building. However, it is of higher priority for the FEMA P-807 Guidelines to design a cost-effective retrofit within the soft-story than to fully achieve any targeted performance objective. The FEMA P-807 Guidelines emphasize that the retrofit is not meant to prevent the soft-story building from being damaged during a seismic event, but rather to prevent the building from collapse and to achieve shelter-in-place following the earthquake. It is critical to note here that the FEMA P-807 Guidelines do not necessarily provide a soft-story structure with a full design code-compliant retrofit. This decision is left to the stakeholders including local and regional governments and building officials. Although the FEMA P-807 Guidelines provide an easy-to-follow retrofit procedure, the results were not experimentally verified.

1.2. The NEES-Soft Project

The NEES-Soft project, whose full title is "Seismic Risk Reduction for Soft-Story Woodframe Buildings," was a five-university, multi-industry, NSF-funded project [van de Lindt *et* al., 2012]. The NEES-Soft project had two main objectives: (1) to enable performance-based seismic retrofit (PBSR) for at-risk soft-story wood-frame buildings; and (2) to experimentally validate the U.S. FEMA P-807 retrofit procedure. Two major tasks of the NEES-Soft project included numerical analysis for development of a performance-based seismic design (PBSD) methodology, and an extensive five-part experimental testing program to better understand the behavior of these at-risk buildings, retrofit techniques, and the building's collapse mechanisms. The five-part experimental program included: (1) real-time hybrid testing (RTHT) of a 20-ft long three-story wood-frame wall with and without a toggle-braced damper assembly; (2) reversed cyclic testing of a light-frame

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wood distributed knee-brace (DKB) assembly [Shao *et al*., 2014]; (3) shake table testing of a light-frame wood DKB assembly to collapse; (4) slow hybrid testing of a full-scale three-story soft-story wood-frame building with various retrofits which concluded with an over-retrofitted collapse testing; and (5) shake table testing of a full-scale, four-story, soft-story, wood-frame building with and without various seismic retrofits concluding with an un-retrofitted collapse testing.

The full-scale hybrid testing conducted in the NEES facility at the University at Buffalo consisted of slow pseudo-dynamic substructural testing of retrofits following the FEMA P-807 Guidelines. The slow testing implies that when the building was subjected to a 40 s earthquake record, the duration of the actual test took much longer, i.e., on the order of two to four hours depending on the time history specifics. There were two main objectives of the experimental program presented here. (1) The hybrid testing aimed to determine the effects of the retrofits on damage to the upper stories and provide direct linkage between the design approach and experimental results. (2) The hybrid tests presented here aimed to verify that soft-story wood-frame buildings retrofitted following the FEMA P-807 Guidelines perform well and meet their designated performance objectives when subjected to moderate seismic events and do not collapse when subjected to maximum considered earthquakes (MCE). No other full-scale tests have been conducted to validate the FEMA P-807 Guidelines, with the exception of the fifth part of the NEES-Soft experimental program (i.e., full-scale shake table testing) which occurred near the same time frame as the hybrid test program presented herein.

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2. FEMA P-807 Retrofit Designs

In order for a retrofit to meet the FEMA P-807 Guidelines it must meet the criteria in three categories: (1) eligibility constraints, (2) strength requirements, and (3) an eccentricity limit. The eligibility constraints are based on geometry and construction, the details of which can be found in the FEMA P-807 document. The general eligibility requirements restrict the building to no more than four wood-frame above-grade stories without an above-grade concrete podium supporting the structure, and require that appropriate soil type and site class adjustment factors be used. The strength requirements specify that the retrofitted building's spectral capacity in each principal direction exceeds the spectral demand. Drift limits are provided for two cases in association with the strength requirements: (1) 4% maximum drift is acceptable for high-displacement capacity stories; and (2) 1.25% maximum drift is acceptable for the low-displacement capacity stories. The FEMA P-807 document specifies the various materials that can qualify a building story as either high-displacement (i.e., wood structural panels, horizontal wood siding, gypsum wall board, etc.) or low-displacement (i.e., stucco, plaster on wood or gypsum lath, diagonal wood sheathing, etc.). The premise of the methodology focuses on eliminating torsion since it exacerbates the soft-story condition and gives rise to structural collapse. In support of this, the eccentricity limits recommend that zero eccentricity between the first-story center of strength and second-story center of strength remains following the retrofit. If this is not possible, the maximum acceptable eccentricity must be less than 10% of the corresponding building dimension.

In this article, two stiffness-based seismic retrofits designed following the FEMA P-807 Guidelines and tested via hybrid testing at full-scale are presented. The first technique utilized cross-laminated timber (CLT) rocking walls for the seismic retrofit and the second technique utilized steel cantilever columns (CC). Prior to the CLT retrofit design, slow-reversed cyclic testing was conducted on individual CLT rocking walls to generate data for hysteretic model calibration. The two soft-story seismic retrofits were designed in the

weak-story tool which uses the hysteretic model of the retrofit elements to capture all of the necessary design information, including the shear connectors.

2.1. Un-retrofitted Soft-Story Building

The un-retrofitted building was a three-story, soft-story, wood-frame building (6.18 m x 7.40 m plan dimension) modeled after typical 1920's style construction to be representative of current structurally deficient, soft-story, wood-frame buildings in California. The first story served as a parking garage with only two interior walls surrounding a stairwell, and the rest of the first story remained open for vehicle parking. The two upper stories were identical in plan and each consisted of single unit apartments. The first-story floor plan with dimensions is shown in Fig. 1a, and the dimensioned floor plan of the two upper stories is shown in Fig. 1b. It is worth noting that these types of soft-story wood-frame buildings are typically larger in plan, but building dimension limitations for this test program were constrained by the size of the NEES facility at the University at Buffalo.

2.2. Cross-Laminated Timber Rocking Wall Hysteretic Calibration

In order to properly model the hysteretic behavior of the CLT rocking walls to be used in the soft-story retrofit, slow-reversed cyclic testing was conducted on four nominally identical pairs of CLT rocking walls. The test set-up is shown in Fig. 2. The dimensions of the individual CLT rocking walls tested here were 0.61 m long and 2.44 m tall, and were connected to a CLT base beam using shear connectors which were anchored onto a steel base beam. Hold-down rods were positioned at both ends of the two rocking walls and connected into the hold-down devices shown in Fig. 2. For conducting the test, the actuator was positioned at the top corner of the left CLT rocking wall in Fig. 2. The force-displacement response to the cyclic testing was averaged for all of the tests, and then normalized by unit length. The averaged dataset was fit with a 10-parameter CUREE hysteretic model [Folz and Filiatrault, 2002] in per unit length units. Figure 3 shows the averaged hysteresis from

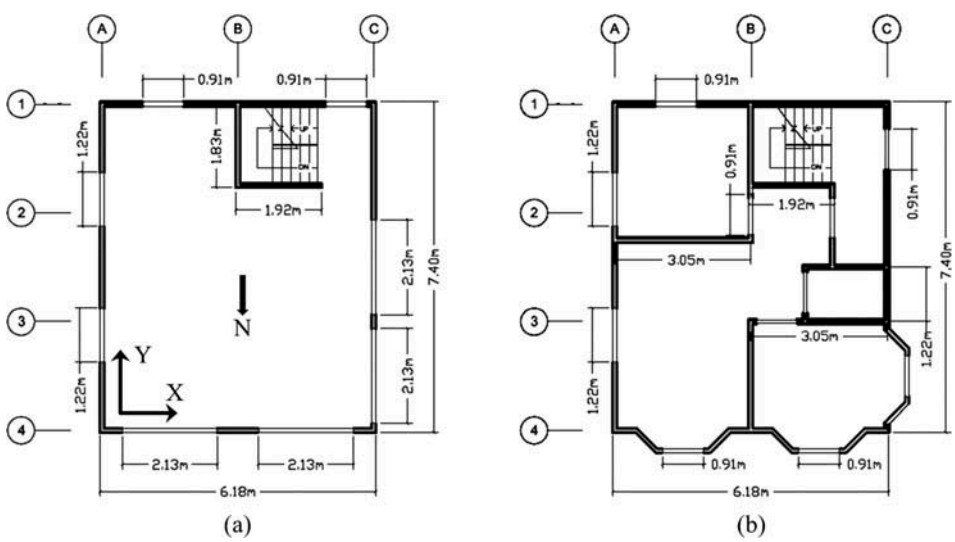

FIGURE 1 Floor plan of un-retrofitted test building: (a) first story and (b) second and third stories.

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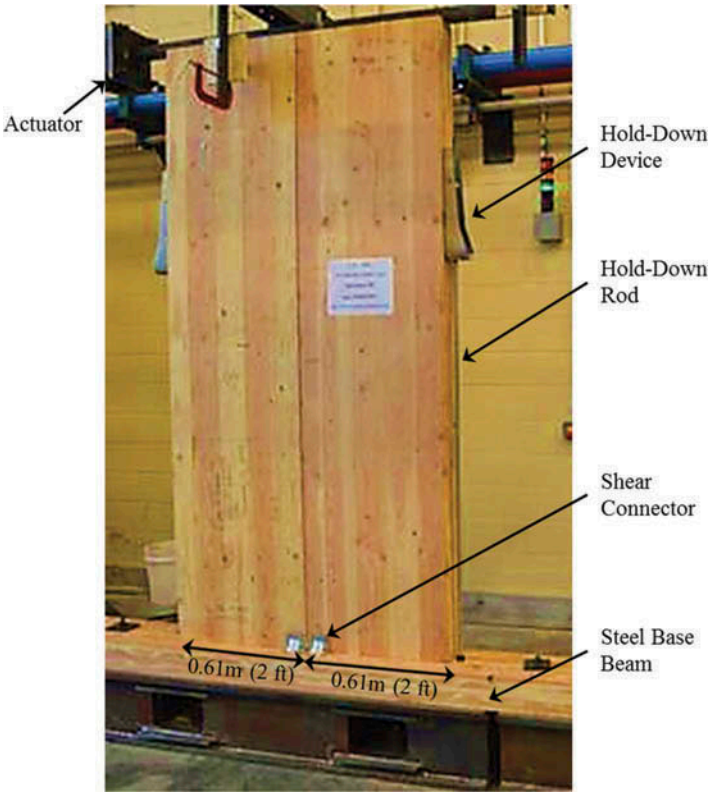

FIGURE 2 CLT rocking wall reversed cyclic test set-up.

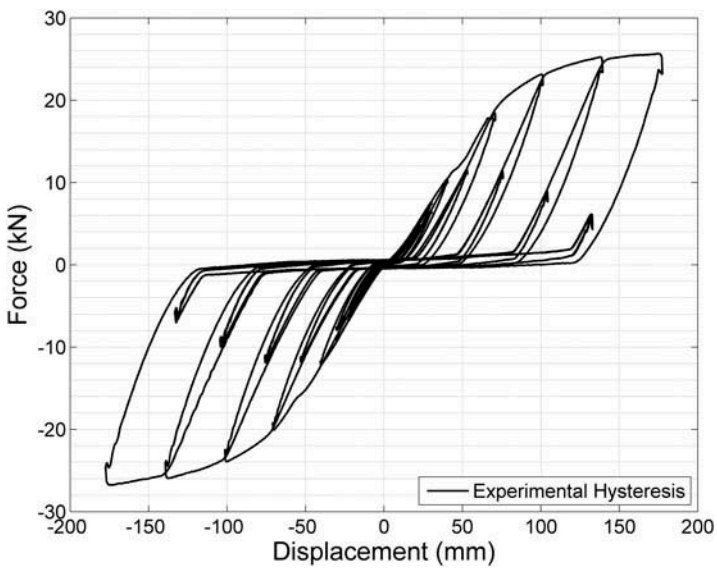

FIGURE 3 CLT rocking wall experimental hysteresis.

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TABLE 1 10 parameter model of CLT rocking wall hysteretic behavior

K_0 (kN/mm)	F_0 (kN)	F_1(kN)	r_1	r_2	r_3	r_4	Δ_u (mm)	α	β
0.354	27.050	0.590	0.077	−2.615	1.501	0.015	177	0.700	1.070

the reversed cyclic testing. As seen in Fig. 3, the actuator reached its displacement limit before the descending branch of the CLT rocking wall's hysteresis was obtained. However, the P-807-focused designs typically limit displacements to be between 50 mm and 100 mm, and as seen in Fig. 3, the CLT rocking wall displacement reached over 150 mm which provided the information required for a retrofit design using the CLT rocking walls as the retrofit elements. Table 1 provides the 10 parameters describing a single 0.61 m (2 ft) length CLT rocking wall's hysteretic behavior from SAPWood [Pei and van de Lindt, 2010].

2.3. CLT Soft-Story Retrofit Design and Location

The CLT retrofit was designed following the FEMA P-807 Guidelines to withstand a spectral acceleration, S_a, of 1.14 g with a 20% probability of exceeding (POE) a target drift of 4% [Park and van de Lindt, 2014]. The combination of 1.14 g S_a and 20% POE were the highest design level which could be achieved by following the FEMA P-807 Guidelines and maintaining full functionality for the first-story parking garage. The CLT retrofit layout is provided in Fig. 4. The resulting design consisted of three 0.61 m × 2.44 m CLT rocking walls positioned in each principal building direction. The rocking walls were located such that full functionality remained for the parking garage while minimizing the in-plane eccentricity. The rocking walls were set adjacent to each other widthwise in the x-direction and aligned lengthwise in the y-direction creating an eccentricity between the center of rigidity and geometric center equal to 6.2 mm in the x-direction and 200 mm in the y-direction, both less than the P-807 Guideline limit of 10% of the corresponding building dimensions.

2.4. Cantilever Column Soft-Story Retrofit Design

The Cantilever Column (CC) retrofit design criteria were the same as for the CLT retrofit, in that it was designed to withstand a S_a of 1.14 g with a 20% POE for 4% maximum inter-story drift on all three stories and to meet the eccentricity limitations. The resulting design required two pairs of CC's, one pair oriented to strengthen each principle direction of the building and to remove torsion. The CC retrofit layout is provided in Fig. 5 with an example of the two-column pair shown in the top left corner, the position of the mid-point (center of rigidity and center of mass for the two-column pair) is provided on the layout.Two W10×19 columns made up the CC retrofit rotated strength-wise in the x-direction and two W12×14 columns made up the CC retrofit rotated strength-wise in the y-direction. The CC layout resulted in an eccentricity of 60 mm in the x-direction and 147 mm in the y-direction, again, both less than the limit of 10% of the corresponding building dimensions. All retrofits were developed using the FEMA P-807 weak-story tool and resulted in satisfying the inter-story drift limit of 4% in all three stories. Table 2 provides the bilinear parameters describing the hysteretic behavior of each of the cantilever columns selected for the seismic retrofit.The hysteresis is provided for each cantilever column; Fig. 6a presents the hysteresis of the W10×19 CC, and Fig. 6b presents the hysteresis of the W12×14 CC.

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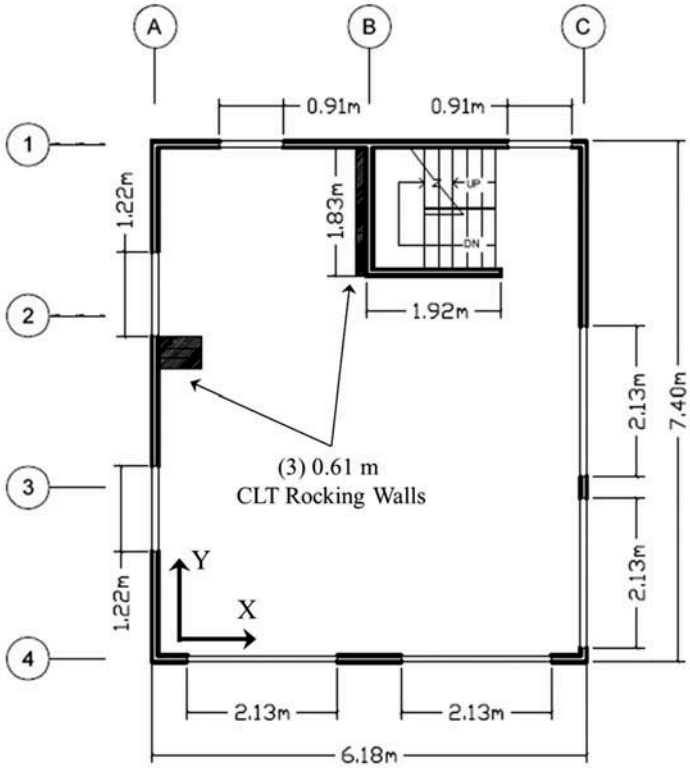

FIGURE 4 Cross-laminated timber rocking wall soft-story retrofit layout.

3. Full-Scale Hybrid Testing for Design Validation

3.1. Hybrid Test Approach

Hybrid testing presents an economical way to conduct full-scale earthquake testing when the damageable components (the first story in this study) can be modeled numerically. In hybrid testing, a portion of the test structure is modeled numerically (i.e., numerical substructure) and the remaining portion is constructed physically (i.e., physical substructure) with interface loading applied directly onto the physical structure through hydraulic actuators (or a shake table). In this study, the hysteretic behavior of the retrofits and their effects on the soft-story were reasonably known, thus the retrofitted soft-story served as the numerical substructure. The behavior of the un-retrofitted upper stories, and more specifically how they interacted with the first story, was less understood and therefore of more interest; hence the physical substructure consisted of the upper two stories which were constructed at full-scale in the laboratory. Figure 7 presents a schematic showing the hybrid testing process employed here. The hybrid test controller coordinated the two substructures by sending the displacement commands from the numerical substructure to the physical substructure (solid arrow) through the actuator controller and xPC target, and feeding the measured forces from the physical substructure back to the numerical substructure through the same path (dashed arrow) which would be used to update the full model for the next time step.

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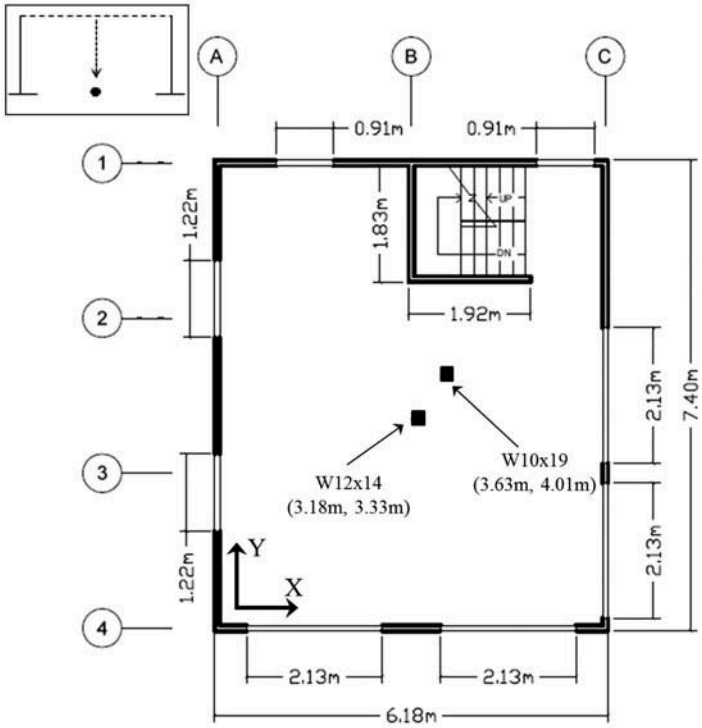

FIGURE 5 Cantilever column soft-story retrofit layout.

TABLE 2 Bilinear parameters describing CC hysteretic behavior

CC	K_0 (kN/mm)	K_y (kN/mm)	Δ_y (mm)
W10×19	0.851	0.009	48.3
W12×14	0.925	0.009	56.1

3.2. Physical Substructure

For the design of the physical substructure, typical construction for this era was reproduced to the extent possible based on several site visits to soft-story wood-frame buildings in Northern California including two that were in the process of undergoing retrofit and renovation. The physical substructure consisted of the upper two stories of the building previously described, constructed at full-scale with finishing materials; the floor plan is provided in Fig. 1b. The physical substructure was anchored to the strong floor through the second-story sill plates which rested on top of a MC6×15.3 steel channel. A 4×4 (88.9 mm × 88.9 mm) (3.5 in × 3.5 in) dimension lumber wood nailer provided the interface between the sill plates and steel channel. Douglas Fir-Larch (DFL) dimension lumber was used for constructing the wall framing, the floor diaphragm, and the roof diaphragm. Horizontal wood siding (HWS) made from 1x10 (19.0 mm × 235 mm) (0.75 in × 9.25 in) DFL dimension lumber planks was used as the exterior sheathing, as seen in Fig. 8a. For fastening, two 8d common nails were hand-driven per board spaced vertically at each stud location at 406.4 mm (16 in) o.c. which formed a couple-moment when racking. One aspect that differed from the typical 1920's to 1960's construction was that GWB was used as

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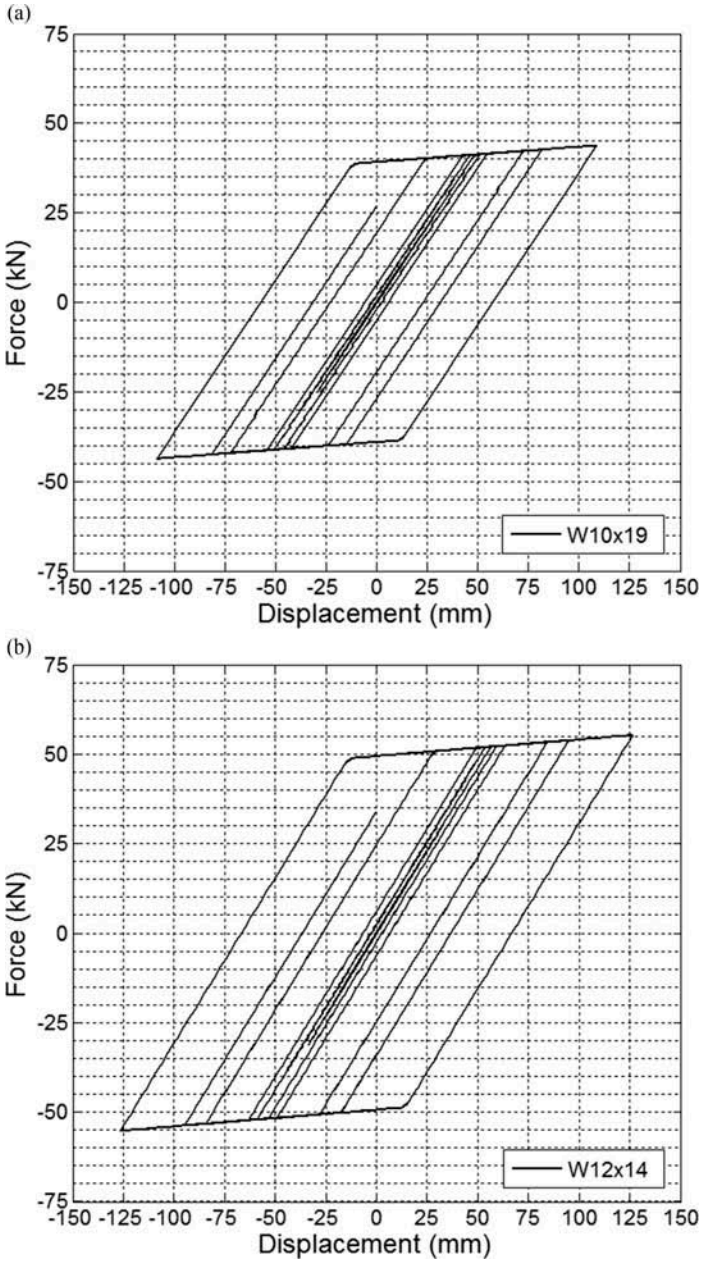

FIGURE 6 Cantilever column hysteresis: (a) W10×19 and (b) W12×14.

the interior wall sheathing as opposed to stucco or plaster on wood lath due to project financial and repair time constraints. Often soft-story wood-frame buildings have been renovated with GWB as the interior sheathing, and the retrofits were designed based on GWB, thus no significant effect on test or project outcomes is envisioned as a result of this substitution.

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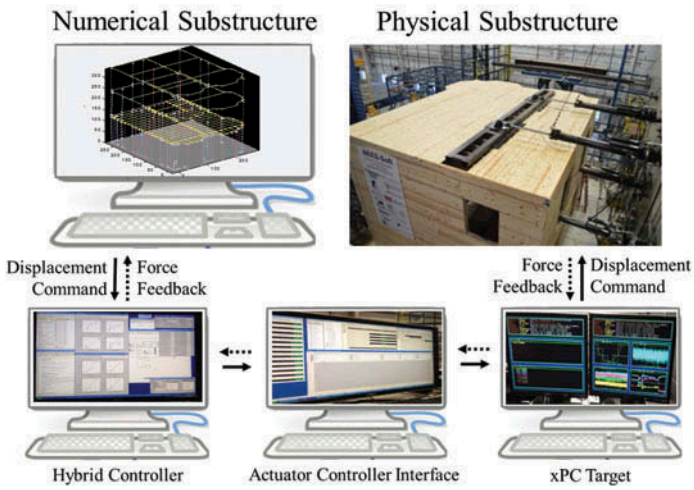

FIGURE 7 Hybrid test process.

(a) (b)

FIGURE 8 Physical substructure: (a) exterior with top actuators connected to roof diaphragm and (b) actuator connection to floor joists.

Four actuators were attached to the floor joists of the third floor and roof diaphragms through a load transfer system, shown in Fig. 8b. Two actuators with a stroke capacity of +/- 1.0 m (40 in) and +/- 13 degrees of rotational freedom in the horizontal direction were mounted at the third floor diaphragm and at the roof diaphragm. Two actuators at each level allowed for control of both translation and in-plane rotation. The two top actuators connected at the roof diaphragm can be seen in Fig. 8a, and the bottom two can be seen going through openings at the second (physical) level.

3.3. System Identification

A system identification (System ID) test was conducted on the physical substructure to identify system properties prior to each hybrid test. The displacement protocol of the System ID test is provided in Fig. 9. From Fig. 9, one can see that a single actuator was moved to + and –2.54 mm while the remaining three actuators were held still and this was

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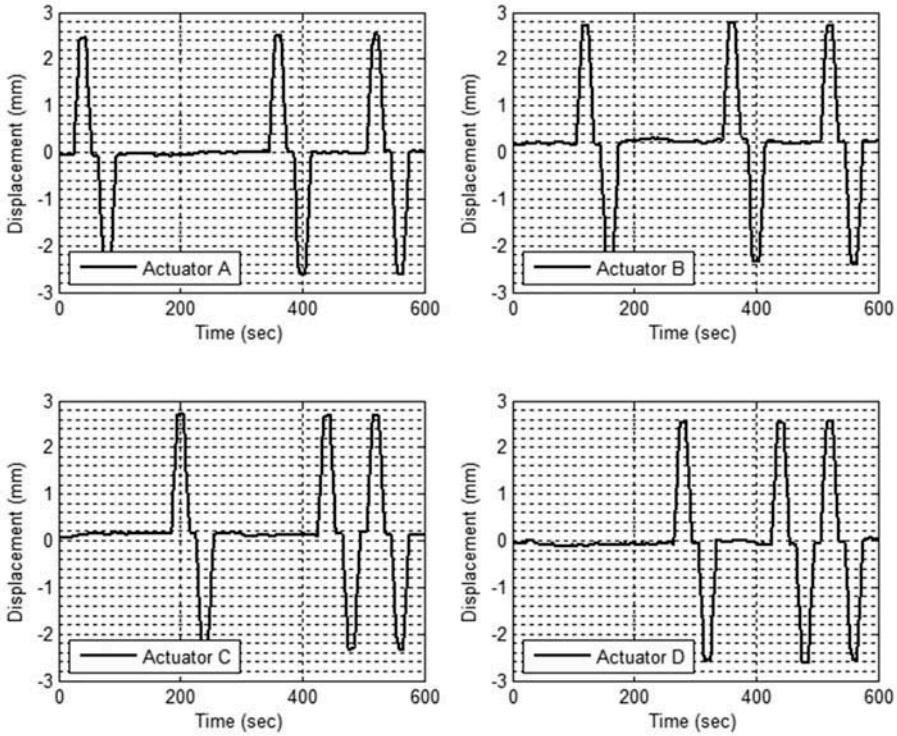

FIGURE 9 System ID displacement protocol.

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repeated for all four actuators one at a time. Once each actuator moved through the displacement loop individually, next, the top two actuators (actuators A and B) moved together, followed by the bottom two actuators (actuators C and D) moving together through the same loop. Finally, a loop with all four actuators moving together concluded the protocol. The recorded data was used to determine the stiffness matrix of the physical substructure and its fundamental period. This information served three purposes: it was used (1) in the preliminary numerical analysis to estimate the structural response and check for numerical instability issues prior to the hybrid test; (2) in conjunction with a visual inspection of the building to determine whether the damage caused by previous testing was too severe to continue; and (3) as the physical substructure properties in the initial integration step of the hybrid simulation. A total of 11 System ID tests and two repairs were conducted throughout the duration of the two hybrid test series. Figure 10 presents the fundamental period of the test building before the initial test, 0.36 s, and after each subsequent test or repair.

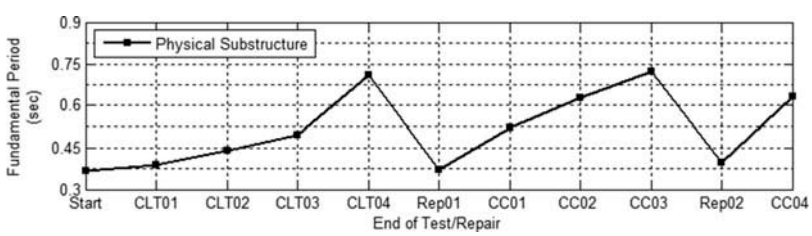

FIGURE 10 Tracking the fundamental period of the physical substructure.

Following the CLT retrofit test program an extensive repair was conducted. Referring to Fig. 10, the System ID test conducted after the first repair (Rep01) shows that the repair was successful in bringing the period down to 0.37 s, approximately the initial starting period which indicated the building was ready for further testing. After the first three CC tests the period had increased to 0.72 s and a considerable amount of visual non structural damage was observed during the routine building inspection. Therefore, a mid-test program repair was required. Looking at point Rep02 on Fig. 10, this repair brought the period down to 0.39 s and the final CC test was conducted.

3.4. Earthquake Ground Motion Selection

The hybrid tests were conducted at three seismic intensity levels. All retrofits were tested against the design basis earthquake (DBE) and maximum considered earthquake (MCE) levels for San Francisco, California, corresponding to ground motions with 10% and 2% probabilities of exceedance (POE) in 50 years, respectively. This was set equal to a MCE spectral acceleration (S_{MS}) of 1.8 g. Only the first hybrid test conducted (CLT01) considered the short return earthquake (SRE) level (i.e., 44% of a DBE ground motion). The FEMA P695 [FEMA, 2009] suite of 22 bi-axial far-field earthquake ground motion records were used in the preliminary nonlinear analysis for the hybrid tests. A numerical model of the un-retrofitted building was subjected to two multi-record nonlinear time history analyses (NLTHA) using all 44 ground motions, one at DBE and one at MCE. The results were used to create cumulative distribution functions (CDF's) based on rank-ordering of peak inter-story drifts (ISD) from the analysis. Figure 11 provides the MCE level CDF created for the un-retrofitted model.The specific ground motion that was selected for the hybrid tests out of the 44 records was based on its ranking along this CDF for all retrofits. A low percentile ground motion corresponded to the earthquake record with approximately 10% probability of non exceedance (PNE) of peak ISD. The high percentile ground motion corresponded to the earthquake record with approximately 70% PNE of peak ISD, shown in Fig. 11. All ground motions on the CDF in Fig. 11 were scaled to MCE level, but ranking

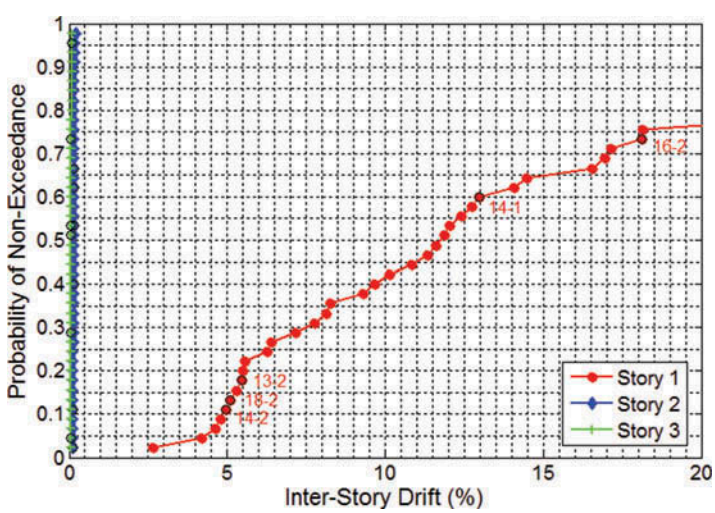

FIGURE 11 Un-retrofitted building inter-story drift probability of non exceedance for ground motion selection for MCE seismic hazard.

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TABLE 3 Earthquake ground motion identification

Identification Number	Earthquake	Recording Station	Component of Recording
13-2	Loma Prieta	Capitola	2
14-1	Loma Prieta	Gilroy Array #3	1
14-2	Loma Prieta	Gilroy Array #3	2
16-2	Superstition Hills	El Centro Imp. Co. Cent	2
18-2	Cape Mendocino	Rio Dell Overpass – FF	2
21-2	San Fernando	LA – Hollywood Stor FF	2

the results of the NLTHA in this way demonstrated the severity of the ground motions. The six California ground motion records were the only ground motions of the 44 records that were considered for testing, to maintain consistency to the extent possible, with the San Francisco Bay Area. These six ground motion records are labeled in Fig. 11 by the order in which they appear in the FEMA P695 suite. Table 3 defines the labels shown in Fig. 11 and provides a description of the six California ground motions considered for the hybrid tests. For example, the fourth hybrid test conducted on the CLT retrofit, CLT04, was a low percentile MCE, thus referring to Fig. 11, the three options for ground motion selection were: 13-2, 18-2, and 14-2. The record selected for the hybrid test was 13-2 (see Table 4). This procedure of record selection for experiment design allows one to assume a lognormal (or other parametric) fit to the peaks of an experimental response quantity for comparison to the fit of the numerical analysis results. This is believed to provide a better measure of comparison since it extends the comparison to the entire range of ground motions within a specified earthquake intensity. Using the information in Fig. 11, prior to each hybrid test, a NLTHA was conducted on the retrofitted model against the three applicable ground motions (either the low or high percentile ranked ground motions) to check for numerical stability. As visible in Fig. 11, all of the displacement occurred in the un-retrofitted soft-story, while the upper stories behaved like a rigid box. As will be demonstrated later, once the first-story was retrofitted, the displacement demand was distributed height-wise over all stories.

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4. Hybrid Test Results and Discussion

The FEMA P-807 Guidelines provide an efficient and economical approach for retrofitting soft-story wood-frame buildings against structural collapse which has been proposed to occur at 7% inter-story drift (ISD) for wood-frame buildings [FEMA, 2009], although the exact ISD at which collapse occurs varies and thus this is an ongoing debate. The test program results presented in this paper sought to verify two inter-story drift limits for each of the FEMA P-807 soft-story retrofits to the extent possible for a typical soft-story wood-frame building: (1) a 4% inter-story drift limit identified as the on-set of collapse in FEMA P807 for the predetermined seismic intensity and DBE; and (2) a 7% inter-story drift limit for MCE intensity, the latter of which is believed to be closer to when collapse actually begins to occurs.

Four hybrid tests of increasing seismic intensity were conducted for each of the two retrofits previously discussed. Table 4 provides a summary of the hybrid testing including the test number, corresponding earthquake ground motion with component and respective seismic hazard level, required scale factor, and resulting scaled peak ground acceleration

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TABLE 4 Description of hybrid tests

Retrofit	Test No.	Ground Motion with Component	Seismic Hazard Level	Scale Factor	Scaled PGA (g)	Peak ISD Response (Story)
Cross-Laminated Timber Rocking Walls (CLT)	CLT01	Loma Prieta @ Capitola – 2	Low Percentile SRE	0.450	0.199	2.28% (1st)
	CLT02	Loma Prieta @ Capitola - 2	Low Percentile DBE	1.022	0.453	1.60% (1st)
	CLT03	Loma Prieta @ Gilroy - 1	High Percentile DBE	1.162	0.645	1.51% (2nd)
	CLT04	Loma Prieta @ Capitola - 2	Low Percentile MCE	1.534	0.680	2.62% (2nd)
Cantilever Column (CC)	CC02	San Fernando @ LA – 2	High Percentile DBE	2.723	0.474	1.72% (1st)
	CC03	Cape Mendocino @ Rio – 2	Low Percentile MCE	1.628	0.893	3.10% (1st)
	CC04	Loma Prieta @ Gilroy - 1	High Percentile MCE	1.743	0.976	2.29% (1st)

(PGA) and peak inter-story drift (ISD) response with location for the seven hybrid tests listed. The order listed in Table 4 is the sequence in which the tests were conducted, starting with CLT01 and ending with CC04. The first CC test does not appear in the table and will not be presented here due to a problem that occurred with the data acquisition at the site.

4.1. CLT Hybrid Tests Results and Discussion

Table 4 provides the peak inter-story drift resulting from all four hybrid tests conducted on the CLT retrofit. The inter-story drifts were computed using the inter-story displacement divided by the specific story height (2.97 m (9.75 ft) for the first story, and 2.72 m (8.93 ft) for the second and third stories). The maximum building drifts were computed using the total roof displacement divided by the total height of the three-story building (8.42 m (27.6 ft)). The first two and the last CLT hybrid tests used the Loma Prieta ground motion recorded at Capitola. CLT01 was conducted at 44% of the intensity of CLT02 (a low percentile DBE) and resulted in a peak inter-story drift of nearly 2.3% occurring on the first story (i.e., the numerical soft-story). The maximum building drift observed during CLT01 was 0.84% which only slightly increased the fundamental period of the physical substructure. CLT02 resulted in a maximum building drift of 0.78% and peak ISD of nearly 1.6% occurring on the first story, and causing a greater increase in the physical substructure's period. A lower peak ISD and a higher total building drift resulted from the higher intensity second test and were likely due to softening of the building. The third hybrid test, CLT03, scaled the Loma Prieta ground motion recorded at Gilroy to DBE. CLT03 resulted in a maximum building drift of 0.74% and nearly 1.5% ISD occurring on the second story causing approximately the same increase in fundamental period as CLT02. The final hybrid test, CLT04, was conducted at MCE and resulted in nearly 2.6% ISD occurring in the second story. Figure 12 presents the ISD time history response for all three stories for CLT04.The first-story data was obtained from the numerical model and the

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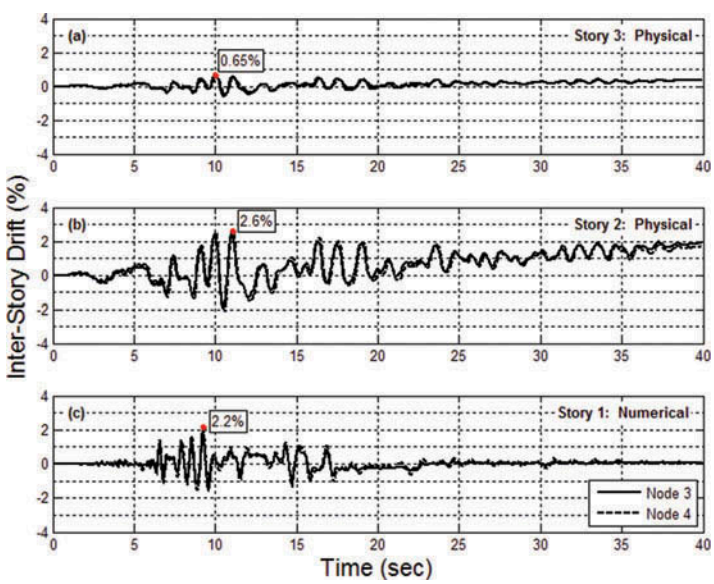

FIGURE 12 CLT retrofit inter-story drift time history for low percentile MCE level hybrid test (CLT04): (a) third story; (b) second story; and (c) first story.

(a) (b)

FIGURE 13 Damage caused during CLT retrofit test program on second story south wall: (a) damaged GWB and (b) close-up of top left window corner.

two upper stories' data were recorded by the instrumentation on the physical substructure. Node 3 corresponds to the southwest building corner and node 4 corresponds to the northwest building corner, demonstrating there was minimal torsion experienced during the test. The first and second stories experienced relatively high ISD, resulting in a maximum building drift of approximately 1.4%. Although the peak ISD of 2.6% occurring on the second story is far below the collapse limit of 7%, this did cause considerable drywall damage, as shown in Fig. 13a and in the close-up in Fig. 13b. This also caused an increase in the physical substructure's period to 0.71 s, as shown in Fig. 11. Figure 13a is the south wall between sections A and B (see Fig. 1b) after CLT04, showing the large shear cracks formed in the drywall initiating at the window corners and cracking along the drywall panel seams. Figure 13b is a close up of the boxed window corner in Fig. 13a. Figure 13b reveals the shear crack formed during CLT03 and the extension caused by CLT04.

Following CLT04, an extensive repair was conducted which consisted of replacing several of the HWS boards on the exterior of the south wall and replacing multiple drywall panels and re-fastening GWB on most of the walls on the second story (i.e., first physical story). Several drywall panels were replaced on the third story and many GWB panels had to be re-fastened as well, often including re-taping and re-mudding.

4.2. CC Hybrid Tests Results and Discussion

Four hybrid tests with increasing seismic intensity, starting with a low percentile DBE and ending with a high percentile MCE, were conducted on the CC retrofit. The results of the first test are not presented here due to a problem with the data acquisition, as mentioned earlier. The second test, CC02, applied the San Fernando ground motion recorded at Los Angeles and scaled to DBE. The maximum building drift was 0.88% and the peak ISD occurred on the first story reaching 1.7%. Test CC03 was a low percentile MCE using the Cape Mendocino ground motion recorded at the Rio station. The resulting maximum total building drift was approximately 1.1% and the resulting peak ISD was 3.1% on the first story. The ISD time history for CC03 is provided in Fig. 14 for all three stories.Based on Fig. 14, it can be seen that torsion was eliminated from the building response. As seen in Fig. 12, again, the first and second stories experienced relatively high ISD, however the third story drifted less than 1% at all times. None of the stories approached collapse and the test ended without any residual drift. Following CC03, the fundamental period of the physical substructure had increased to 0.72 sec which was above the period that was reached

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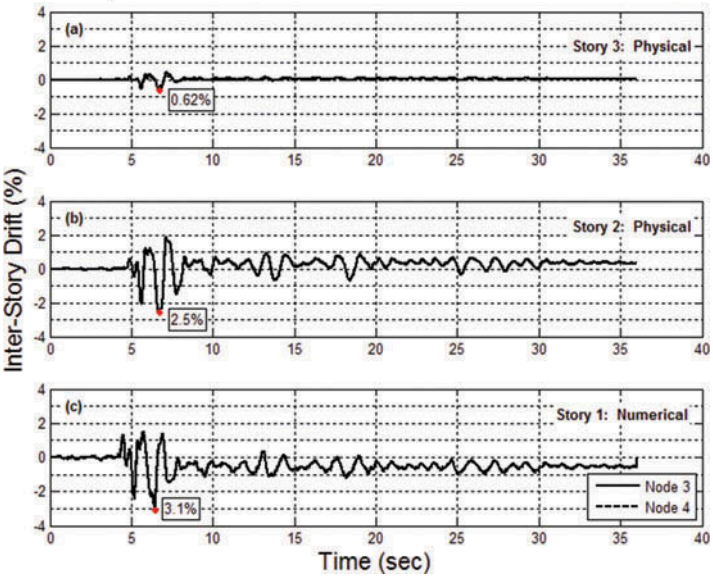

FIGURE 14 CC retrofit inter-story drift time history for low percentile MCE hybrid test (CC03): (a) third story; (b) second story; and (c) first story.

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following the final CLT test (see Fig. 10), indicating the need for repairs. An extensive repair was conducted on the physical substructure, similar to the previous repair (Rep01), except no HWS was replaced. The results from the System ID test conducted after the Rep02 showed the period had been significantly reduced to 0.39 s which was near the initial period value; therefore, the final CC test was conducted. The final test, CC04, was a high percentile MCE which used the Loma Prieta ground motion recorded at Gilroy. The peak ISD occurred on the first story reaching 2.3%, which was less than the peak ISD resulting from the previous low percentile MCE test (3.1%), as expected due to the mid-test program repair. The ISD time history for CC04 is provided in Fig. 15 for all three stories.Similar to Fig. 14, one can see that the torsional response was removed from the building by the CC retrofit. Although the peak ISD for CC04 was less than CC03, the maximum building drift increased from the previous test reaching 1.4% which resulted in a significant increase in period to 0.63 s.

4.3. Retrofit Design Validation

Several of the earthquakes selected for the hybrid testing were ranked in the lower 50th percentile on the retrofitted buildings' CDFs of the 44 earthquake ground motions presented earlier. However, for the four hybrid tests conducted on the CLT and CC retrofitted soft-story buildings, in all cases the 4% drift limit was met in all three stories even at higher intensities than for which it was designed. Figures 16a and 16b present the MCE numerical CDFs for the CLT and CC retrofits, respectively.If collapse is assumed to occur at 7% ISD, there is an 86% PNE of collapse for the CLT retrofit, and a 93% PNE of collapse for the CC retrofit. It has been shown that actual collapse does not occur for wood-frame structures of this type until more than 10% ISD [Bahmani *et al.*, 2014]. If the larger drift limit is assumed

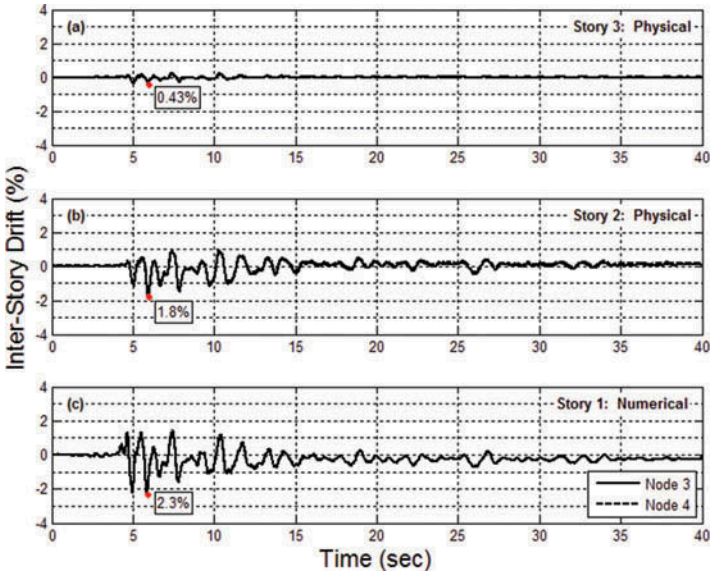

FIGURE 15 CC retrofit inter-story drift time history for high percentile MCE hybrid test (CC04): (a) third story; (b) second story; and (c) first story.

for collapse, there is greater than a 98% PNE for both retrofits. A vertical line is plotted at the location of the peak ISD recorded during the respective hybrid test series. This indicates that the numerical result may have over-predicted the experimental response which would introduce a level of conservatism. If the numerical CDF was shifted to the left in either figure to match the experimental data, the probability of collapse would be even less than either of the two PNEs previously mentioned.

5. Conclusions

A series of full-scale wood-frame building hybrid tests were conducted on two different retrofits designed in accordance with the FEMA P-807 soft-story retrofit guidelines. Both retrofits were tested against a range of seismic intensities with the largest being the MCE level for San Francisco, California ($S_a = 1.8$ g). In all cases, the retrofitted soft-story wood-frame building performed well meeting the 4% drift limit set by the FEMA P-807 Guidelines at an even higher intensity than designed ($S_a = 1.14$ g). The test results demonstrated the effectiveness of the soft-story-only retrofit in strengthening the soft-story while not transferring enough force into the upper stories as to exceed the drift limit or on-set collapse, and in eliminating torsional response. It should be noted that only the lateral resistance of the retrofit elements were modeled as part of the numerical substructure and not the connection details. The connection details for the retrofit elements, specifically the CLT rocking walls, need to be designed and analyzed prior to implementation to check and prevent a localized failure mechanism, e.g. in the diaphragm.

The FEMA P-807 Guidelines provide a logical engineering approach for seismic retrofit of the at-risk soft-story wood-frame buildings when the retrofit must be limited to the soft-story only. Overall the results indicate that the FEMA P-807 Guidelines

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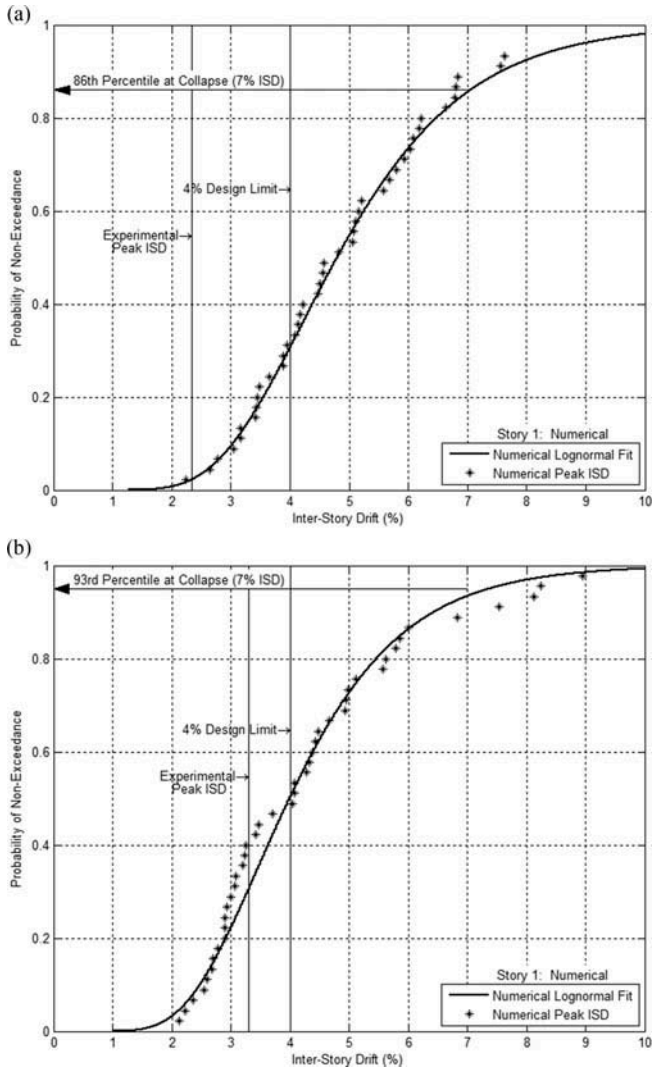

FIGURE 16 MCE numerical CDF with inter-story drift limits: (a) CLT retrofit and (b) CC retrofit.

result in building performance as was intended during the development of the guideline document. The small plan dimensions of the building may have had some effect. It is important to point out that the soft-story-only retrofit designs can (and typically do) result in a retrofitted building that does not meet current design code. Relatively large drifts were experienced by the first and second stories during the MCE hybrid tests causing significant nonstructural damage that could be severe and pose some level of risk for building occupants due to the detachment of GWB panels. However, building collapse would be highly unlikely for DBE level events and most MCE level events. Overall, it is the authors' opinion that the FEMA P-807 retrofit procedure provides an adequate seismic retrofit which prevents collapse at the first story. It is particularly recommended when financial and other constraints limit the seismic retrofit from bringing the soft-story building up to current code level or applying performance-based procedures.

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