Experimental Seismic Behavior of a Full-Scale Four-Story Soft-Story Wood-Frame Building with Retrofits. I: Building Design, Retrofit Methodology, and Numerical Validation

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Abstract: Soft-story wood-frame buildings have been recognized as a disaster preparedness problem for decades. There are tens of thousands of these multifamily three- and four-story structures throughout California and other parts of the United States. The majority were constructed between 1920 and 1970 and are prevalent in regions such as the San Francisco Bay Area in California. The NEES-Soft project was a five-university multiindustry effort that culminated in a series of full-scale soft-story wood-frame building tests to validate retrofit philosophies proposed by (1) Federal Emergency Management Agency's recent soft-story seismic retrofit guideline for wood buildings and (2) a performance-based seismic retrofit (PBSR) approach developed as part of the NEES-Soft project. This paper is the first in a set of companion papers that presents the building design, retrofit objectives and designs, and full-scale shake table test results of a four-story 370-m^2 (4,000-ft^2) soft-story test building. Four different retrofit designs were developed and tested at full scale, each with specified performance objectives, which were typically not the same. Three of these retrofits were stiffness or strength–based strategies and one applied supplemental damping devices in a pinned preassembled frame. This paper focuses on the building and retrofit design methodologies and specifics and the companion paper presents the experimental results of full-scale shake table tests ranging from 0.2- to 1.8-g spectral acceleration for all four retrofits. **DOI: 10.1061/(ASCE)ST.1943-541X.0001207.**

Author keywords: Performance-based seismic design; FEMA P-807; Soft-story; Wood frame; Shake table; Seismic retrofit; Wood structures.

Introduction

Full-scale whole-building tests have been performed around the world only 10 to 20 times. U.S.-based projects for full-scale testing of light-frame wood buildings have increased significantly since the late 1990s as a result of the CUREE-Caltech Project and projects related to the National Science Foundation's (NSF's) George E. Brown Jr. Network for Earthquake Engineering Simulation (NEES). A good summary is provided in a 2009 report prepared by the National Association of Home Builders (NAHB) Research Center (NAHB 2009), but significant testing has occurred in the 5 years since that report. For brevity, only specific projects and tests that have had a direct effect on the planning and execution of the test program presented herein are discussed. Fischer et al. (2001) tested a rectangular two-story house with an integrated one-car garage. The building was full scale but overall size was limited to the shake table dimensions, but regardless provided state-of-the-art test results. The building was designed in accordance with the 1988 Uniform Building Code (UBC) (International Conference of Building Officials 1988) and performed well at code level [design basis earthquake (DBE)] and near-fault [maximum considered earthquake (MCE)] records from the 1994 Northridge earthquake. Gypsum wall board and stucco were shown to provide a very significant increase in strength and stiffness (Filiatrault et al. 2002). As part of the NEESWood Project, Filiatrault et al. (2010) conducted full-scale triaxial tests on a two-story three-bedroom 167.2-m^2 (1,800-ft^2) townhouse with an integrated two-car garage on the twin shake tables of the State University of New York at Buffalo. This building was also designed to the 1988 UBC (International Conference of Building Officials 1988). The results showed that for light-frame wood buildings typical of 1980s to 1990s California, only moderate damage resulted during a design-level earthquake, while significant and costly damage occurred during the MCE. This included a 12 mm (1/2 in.)-wide sill plate crack around the entire building that would be very costly to repair, but did not pose a threat to the lives of would-be inhabitants. The earlier conclusion about the added strength and stiffness based on a test of a smaller floor plan resulting from the gypsum wall board and stucco was confirmed during the NEESWood townhouse test. Full building and results are available in the project report by Christovasilis et al. (2007). During the CUREE-Caltech Wood-frame Project, a three-story apartment building with a tuck under garage was tested by Mosalam and Mahin (2007). Their conclusions confirmed that these types of buildings are prone to torsional response and soft-story collapse.

The world's largest shake table test was conducted as part of the NEESWood Project by van de Lindt et al. (2010a, b) and was a test

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Note. This manuscript was submitted on May 29, 2014; approved on October 24, 2014; published online on December 9, 2014. Discussion period open until May 9, 2015; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Structural Engineering***, © ASCE, ISSN 0733-9445/E4014003(14)/$25.00.

of a 1,300-m^2 (14,000-ft^2) six-story apartment building at Japan's E-defense facility in Miki, Japan. The building was 12.2×18.3 m (40×60 ft) in plan and 17.1 m (56 ft) tall. Full details are available in the project task report (Pei et al. 2010). The objectives were to (1) provide a general understanding of how midrise light-frame wood buildings perform in a major earthquake, and (2) provide validation for the performance-based seismic design philosophy developed within the project which was a variation on direct displacement design (DDD) developed by Pang et al. (2009). Overall performance was excellent at the MCE level, but it should be kept in mind that the test structure was designed at a level expected to provide seismic performance superior to current code (van de Lindt et al. 2014).

In this paper two retrofit design methodologies are summarized and applied to a full-scale four-story test building which has two soft and weak sides due to a combination of garage door openings and archaic materials. The four-story building was tested on the outdoor shake table at the NEES at the University of California San Diego facility as part of the NEES-Soft project (van de Lindt et al. 2014). The first retrofit guideline [i.e., FEMA P-807 Guideline (FEMA 2012)] was developed by the Applied Technology Council (ATC) under its Project 71.1 and published as FEMA Report P-807 (FEMA 2012). The second approach, developed as part of the NEES-Soft project and termed performance-based seismic retrofit (PBSR), is a method that decouples the building deformation resulting from translation and torsion during the design process. In the case of the four-story test building the torsion was removed in the design, which is optimal but may not always be possible due to architectural (or other) constraints. The PBSR method was first introduced by Bahmani and van de Lindt (2012, 2013) with general applicability to multistory buildings.

Test Building Design Approach

As with all test programs the design of a system-level specimen presents unique challenges because it is, by definition, specific but the test objectives necessitate a generalized representation of the building space being studied. A number of site visits were conducted to identify the architecture of the test building with the focus placed on the San Francisco Bay Area in California. However, it should be kept in mind that both retrofits philosophies are applicable to all types of soft-story wood-frame buildings even if they do not look like the building described in this paper and van de Lindt et al. (2014). Fig. 1(a) shows a photo of a typical Bay Area soft-story wood-frame building and Fig. 1(b) shows a photo of the NEES-Soft test building in San Diego just prior to the start of the test program.

Although the exterior architecture was important for aspect ratio and determining locations and number of openings at the soft and other stories, there were several other features of this particular building era that had to be identified, such as interior wall density, room size, nail schedules, floor diaphragm, and flooring details. The test building plan dimensions were for the most part dictated by the shake table size, which was 7.6×12.2 m (25×40 ft), resulting in plan dimensions of 7.3×11.6 m (24×38 ft). Fig. 2 presents the architectural plan views of the first story and upper stories. Fig. 3 presents several elevation views of the building.

As mentioned, certain features observed during site visits were included such as the light well that can be seen in Fig. 2. However, some modifications to the archaic building materials were necessary to ensure that the test building could be repaired in a timely and financially manageable way because a number of tests were conducted at moderate to high amplitude. Table 1 provides a comparison between the features commonly found in these types of soft-story buildings and the NEES-Soft four-story test building.

Overview of the FEMA P-807 Methodology

Recognizing the high risk of collapse for these weak and soft first-story wood-frame buildings, FEMA funded and charged the ATC with creating a new set of guidelines entitled "Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings with Weak First Stories" [FEMA P-807 (FEMA 2012)]. In an effort to make retrofitting more affordable, the P-807 guideline proposes several novel approaches for retrofit. Specifically, the methodology focuses on buildings with soft and weak first stories with retrofitting in the first story only. This significantly reduces the probability that building occupants will have to relocate during retrofit construction. In addition, P-807 focuses on identifying a lower and upper bound of strength and stiffness for the retrofit such that the retrofit does not move the soft story up to the second level. The methodology also explicitly considers the strengths of the nonstructural walls to evaluate their contribution to overall performance, making it, in a sense, performance based although it has not been articulated as such. Finally, the methodology does not require explicit structural analysis of the building in question, which could include potentially expensive nonlinear analysis. This is accomplished by

(a) (b)

Fig. 1. Comparison of the architecture for (a) a soft-story wood building in the San Francisco Bay area (image by Mikhail Gershfeld); (b) the test building designed as part of the NEES-Soft project (image by Pouria Bahmani)

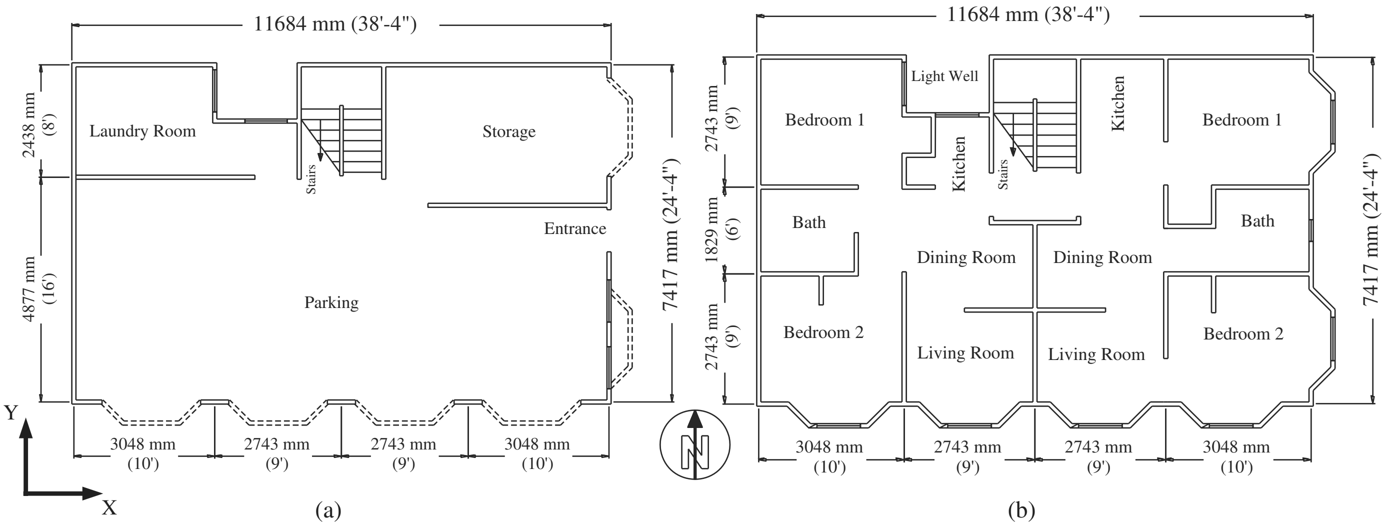

Fig. 2. Floor plans for the four-story test building: (a) first story; (b) upper stories

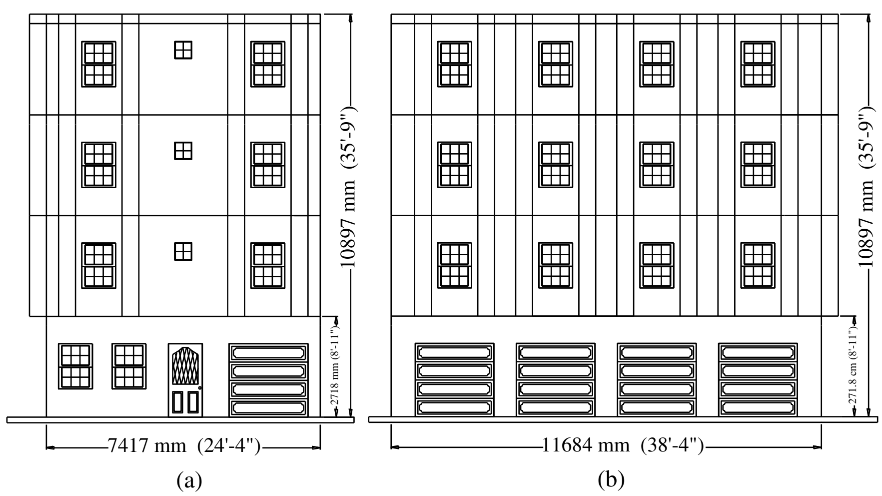

Fig. 3. Elevation views of the test building: (a) east view; (b) south view

comparing the building in question with surrogate models, which are hundreds of similar structures (with many variants each resulting in thousands of possibilities) whose performance has already been assessed using nonlinear response history analysis. After this entire process, performance is evaluated in probabilistic terms based on estimated fragilities from the representative surrogate model. This approach can lead to a wider range of mitigation performance options being available to the building owner voluntarily retrofitting or the jurisdiction requiring retrofitting.

In the FEMA P-807 guidelines any hazard level acceptable to the owner and jurisdiction may be used as a target. For example, as part of the City of San Francisco's mandatory retrofit ordinance, Administrative Bulletin 107 (AB-107) (San Francisco 2012) stipulates in section B.1.1.1 that an acceptable hazard level is a spectral demand of $0.5S_{\mathrm{MS}}$ (50% MCE) calculated in accordance with ASCE-05 (ASCE 2005), but using a value of 1.3 for F_a in Site Class E locations. The default performance level for P-807 is what is referred to as the onset of strength loss (OSL), which is an interstory drift (ISD) limit associated with the combination of wall sheathing materials in a building no longer able to resist lateral forces. There are two types of systems described in P-807, namely, low ductility systems with OSL at 1.25% interstory drift and high ductility systems with OSL at 4% interstory drift. OSL is also the performance level selected for use in AB-107's implementation of P-807. Regarding the probability of exceedance (POE), AB-107 sets this at 30% but allows the POE to increase to 50% when

Table 1. Comparison between Construction Material in Archaic Building and the Test Building

Construction category	Features and items commonly observed during site visit	Features and items provided in test building
Exterior	Wood siding	Included
	Trim	Not included
	Stucco (not all buildings)	Not included
Interior	Plank flooring	Included
	Hardwood floors	Included
	Plaster (on lathe)	Gypsum wall board[a]
	Tile	Not included[b]
Architecture	Bay windows	Included
	Light well	Included
	Large openings (garage)	Included

[a]12.5-mm (0.5-in.) regular gypsum wall board was used in the test building.
[b]Equivalent mass for tile added in kitchens and bathrooms.

the bottom story being retrofitted contains only parking, storage, or utility uses or occupancies (B1.1.3). At the time of the design of the four-story test building presented in this paper the POE in AB-107 was under discussion, so 20% was selected to ensure that reasonable conclusions could be drawn from the test results. Another key limitation for soft-story retrofit within the FEMA P-807 guidelines was the requirement to keep the resulting center of rigidity for the retrofitted floor within 10% of the center of rigidity of the floor above it. This constraint was in place for the NEES-Soft test building because not all retrofit elements were added at the building perimeter due to building functionality requirements.

Proper load path detailing is often one of the more challenging aspects of both design and retrofit. The analytical procedures in FEMA P-807 assume the floor diaphragm above the bottom story is adequate, the foundations below are adequate, and there are sufficient load transfer and load path elements and connections in place to allow the system to perform as desired. If these assumptions are not accurate for the evaluated building, it would be necessary to correct these deficiencies to ensure the performance objective can be met.

Overview of the PBSR Methodology

PBSR, which is essentially a subset of performance-based seismic design (PBSD), is a method that seeks to meet or exceed a minimum performance criteria specified by the owner or stakeholders under a specified seismic intensity level. The biggest challenge in any retrofit procedure including PBSR is that the engineer must deal with an existing structure, which in most cases has not been designed in accordance with current building codes and regulations. Architectural constraints for the building such as openings, load-bearing walls, and living areas further complicate retrofit scenarios. In addition, for retrofit of buildings it is imperative that the disruption to owners or tenants be minimized whenever possible. This being said, the PBSR method can offer an efficient and effective way of retrofitting buildings by distributing the stiffness and strength of retrofits over the height of the building and in the plane of each story while keeping interstory drifts below the target drift assumed during the design process. PBSR allows the engineer to meet the desired performance criteria and, at the same time, accommodate existing architectural constraints. In addition, the PBSR method prevents overstrengthening and understrengthening floor levels by limiting the relative stiffness and strength ratio of stories to the predefined ranges. While desired design performance can be met with this methodology, it is somewhat disruptive to tenants or homeowners because every (or almost every) story level must be retrofitted.

The PBSR methodology presented in this paper was developed based on the displacement-based design (DBD) concept that was originally proposed by Priestley (1998) for reinforced concrete structures and later modified by Filiatrault and Folz (2002) to be applied to wood structures. Pang and Rosowsky (2010) proposed the DDD method using modal analysis, and later Pang et al. (2009) proposed a simplified procedure for applying the DDD method, which was ultimately applied to a six-story light-frame wood building and tested in Miki, Japan (van de Lindt et al. 2010b), validating the simplified DDD procedure. Finally, Wang et al. (2010) extended the work of Pang et al. to allow correction as a function of building height.

The DDD methodology determines the required lateral stiffnesses over the height of the building such that the building meets the target displacement. However, none of the methods described previously can be applied for designing and retrofitting torsionally irregular buildings. In many cases, the existing buildings are structurally and architecturally irregular, which leads to higher displacements caused by torsional moment in addition to lateral force. In both cases, the torsional response of the building has to be considered when designing retrofits and selecting performance criteria. In the PBSD method proposed by Bahmani et al. (2013), translational and torsional mode shapes of the building can be decoupled using the method suggested by Kan and Chopra (1977), normalized by stiffness and weight of an arbitrary reference story (usually the first story), and finally combined using modal coupling factors.

Whenever practical, the torsional response of the structure should be eliminated by distributing the retrofit properly in the plane of each story. A full-scale four-story soft-story wood-frame building was retrofitted using this approach and is presented along with the numerical validation for three of the retrofit designs in this paper. The full-scale shake table validation of the retrofit designs is then presented in the companion paper (van de Lindt et al. 2014), which includes all four retrofit approaches.

If it is not feasible to eliminate the torsion due to some type of building (or other) constraint, the PBSR approach can be applied by assuming a ratio between the displacement caused by lateral force and torsional moments and then satisfying the assumption while applying the retrofit. In fact, there may be cases in which allowing some level of torsion to remain in the retrofitted building is much less expensive and, as such, enables the owner to retrofit. An example of a three-story torsionally unbalanced wood-frame building retrofitted using this PBSR methodology without eliminating torsion can be found in van de Lindt et al. (2013).

The same logic and methodology that was used in the simplified DDD method (Pang et al. 2010) was used to develop the simplified PBSR retrofit method with two major differences: (1) in the PBSR method the stiffness and strength of the existing lateral load resisting system and the retrofit elements should be calculated at the target interstory drift, and (2) the retrofits should be distributed in the plane of each story such that they minimize torsion.

As mentioned previously, in torsionally unbalanced buildings, in-plane torsional moments, and consequently rotational displacements, can be induced when the center of rigidity of a story does not coincide with the center of mass. In this case, additional rotational displacements due to torsional imbalance should be taken into account whenever they occur. Fig. 4(a) presents an N-story building with lumped masses of W_j for the jth story ($j = 1$ to N). It can be seen that the total displacement at the center of mass of the jth story can be divided into two major displacements: (1) the displacement due to lateral force (Δ_j^{Tns}), and (2) the displacement due to torsional moment (Δ_j^{Tor}). If the retrofits are placed such that the center of mass (CM) (the point to which the resultant inertial force at each story applies) and the center of rigidity (CR) (the point to which resultant internal force applies) move toward each other, the torsional response can be eliminated and neglected from the design [i.e., $\Delta_j^{\mathrm{Tor}} = 0$ in Fig. 4(a)]. In this case, the building after the retrofit is close to a symmetrical building with no significant torsion.

In order to simplify the PBSR procedure, the same analogy that was used in the simplified DDD method was used in this study. The N-story building in Fig. 4(a) can be simplified to an equivalent single degree of freedom system [Fig. 4(b)] with effective weight of W_{Eff} and effective height of h_{Eff}. The W_{Eff}, h_{Eff}, and distribution of lateral force can be calculated based on the approach outlined in Pang et al. (2010). The fundamental translational period of the building can be obtained from the displacement response spectrum, which is developed based on the design spectral acceleration maps of ASCE 7-10 (ASCE 2010) and should be modified to take into account the effect of equivalent damping.

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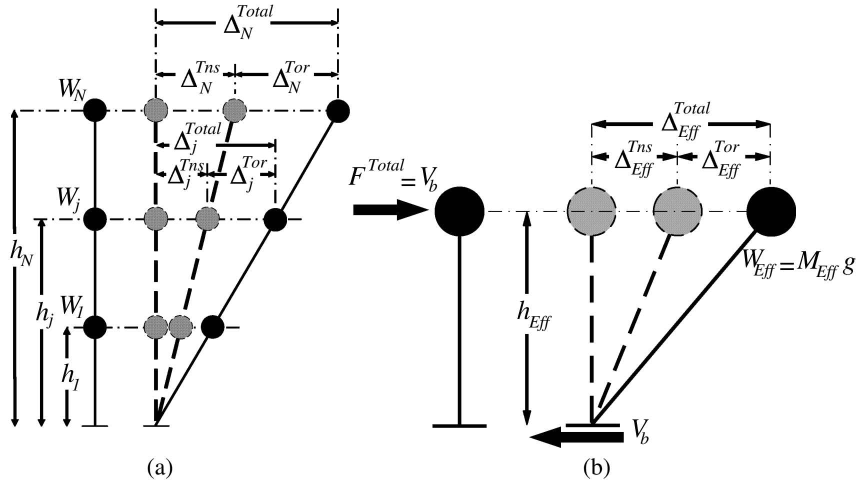

Fig. 4. Translational and torsional displacements in a torsionally unbalanced building: (a) multistory building; (b) equivalent single degree of freedom model of multistory building

After obtaining the base shear, the lateral design forces to be applied at each story can be determined using the lateral load distribution factors (C_v^j), and consequently the distribution of the stiffness for lateral load resisting elements over the height of the building can be calculated. The last step is distributing the lateral force resisting elements such that the distance between the CR and CM at each story is minimized based on the secant stiffness of the elements at the target displacement.

Retrofit Design and Numerical Validation

A four-story wood-frame building with a soft and weak story at the ground level was retrofitted based on the guidelines outlined in FEMA P-807 as well as the PBSR retrofit procedures. The large garage openings and very low shear wall density at the first floor compared with the upper floors resulted in a soft-story collapse mechanism being present (Fig. 2). The calculated ratios of the stiffness of the first story to the average of the stiffness of the upper stories were 62 and 32% in X- and Y-directions, respectively. This places the building into the "stiffness-extreme soft story irregularities" according to the ASCE 7-10 (ASCE 2010) definition. In addition to the soft story, the building was weak due to low strength load resisting elements making up the building. Therefore, this building would be categorized as structurally deficient and prone to collapse.

The building was retrofitted and tested in four different phases based on two different retrofit philosophies, namely, the FEMA P-807 guideline and the PBSR. All approaches and retrofits were validated both numerically and then experimentally. Along with the details of the building and retrofit designs, the numerical validation of the retrofits is presented in this paper and the full-scale shake table experimental validation is presented in the companion paper (van de Lindt et al. 2014). Cross-laminated timber (CLT) panels, wood structural panels (WSPs), and steel special moment frames (SMFs) were the stiffness- and strength-based retrofits deemed as alternatives for this retrofit design. Fluid dampers preassembled in pinned frame assemblies were also examined as an option. Table 2 presents the retrofit methods and specifics of the design criteria that were used in the retrofit designs for the test building.

The design criteria for the FEMA P-807 retrofits were different than those of the PBSR. In this study a 20% exceedance probability of the FEMA P-807 guidelines at 50% MCE intensity was used, whereas the target displacement for the retrofits designed using PBSR procedure was 50% exceedance probability at MCE level. For the generic site used within the NEES-Soft project, the MCE seismic intensity was assumed to be equal to a spectral acceleration of $S_a = 1.8$ g, thus an acceleration of 0.9 g corresponded to a 50% intensity of the MCE, in excess of the 50% MCE that was anticipated to be incorporated into the San Francisco city ordinance. However, this was for an MCE of 1.8 g and there are locations throughout the San Francisco Bay Area that have MCEs approaching and even exceeding 2.2 g, therefore 1.1 g was used because it was possible with this design.

The first retrofit (Test Phase 1) utilized CLT and was designed to satisfy the FEMA P-807 guidelines at 0.9 g spectral acceleration with only a 20% probability of being exceeded [i.e., probability of nonexceedance (PNE) of 80%]. In Phase 2 the retrofit design utilized a steel SMF and wood structural panels to satisfy the FEMA P-807 guideline but at a spectral acceleration of 1.1 g. Then, in Phases 3 and 4, the building was retrofitted using PBSR to meet the performance criteria with a probability of nonexceedance of 50% at MCE level, i.e., 1.8 g spectral acceleration. In order to verify the retrofit designs, extensive nonlinear time history analysis (NLTHA) was conducted using state-of-the-art software (van de Lindt and Pei 2010).

Unretrofitted Building

In order to investigate the behavior of the original unretrofitted building, the building was modeled numerically using a well-known approach that models each shear wall as a 10-parameter hysteretic spring (Folz and Filiatrault 2001). Floors were assumed

Table 2. Retrofit Techniques and Design Criteria

Retrofit method	Testing phase	Retrofit technique	Design criteria
P-807	1	CLT	PNE = 80% at $S_a = 0.9$ g[a]
P-807	2	Steel SMF and WSP	PNE = 80% at $S_a = 1.1$ g
PBSR	3	Steel SMF and WSP	PNE = 50% at $S_a = 1.8$ g
PBSR	4	FVD and WSP	PNE = 50% at $S_a = 1.8$ g

Note: FVD = fluid viscous damper frame assembly; PNE = probability of nonexceedance.
[a]Records were scaled according to the ASCE 7-10 (ASCE 2010) scaling method.

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Table 3. Ten Hysteresis Parameters

Type of resisting element	Fastener spacing [mm (in.)]	K_0 [N/mm/m (lb/in./ft)]	F_0 [N/m (lb/ft)]	F_1 [N/m (lb/ft)]	r_1	r_2	r_3	r_4	D_u [mm (in.)]	α	β
P-807											
HWS[a]	406 (16)	109 (190)	1,378 (94.4)	248 (17)	0.140	−0.980	1.01	0.035	73 (2.88)	0.45	1.06
GWB[b]	406 (16)	318 (554)	3,079 (211)	175 (12)	0.109	−0.064	1.01	0.010	18 (0.69)	0.80	1.10
PBSR											
HWS[a]	406 (16)	85.0 (148)	657 (45)	248 (17)	0.095	−0.95	1.01	0.035	206 (8.1)	0.45	1.06
GWB[b]	406 (16)	259 (450)	1,459 (100)	95 (6.5)	0.023	−0.040	1.01	0.010	28 (1.1)	0.80	1.10
Wood shearwall	51/305 (2/12)	2,431 (4,232)	29,026 (1,989)	3,605 (247)	0.030	−0.073	1.01	0.033	50 (1.97)	0.76	1.24
(wood structural panel)[c]	76/305 (3/12)	2,176 (3,787)	18,635 (1,277)	2,481 (170)	0.032	−0.060	1.01	0.023	48 (1.90)	0.71	1.29
	102/305 (4/12)	1,740 (3,028)	14,680 (1,006)	2,131 (146)	0.026	−0.056	1.01	0.022	47 (1.85)	0.76	1.29
	153/305 (6/12)	1,355 (2,359)	9,850 (675)	1,328 (91)	0.025	−0.049	1.01	0.019	47 (1.84)	0.71	1.29
	305/305 (12/12)[d]	678 (1,180)	4,932 (338)	664 (45.5)	0.025	−0.049	1.01	0.019	47 (1.84)	0.71	1.29
CLT panel[e]	—	804 (1,400)	42,833 (2,935)	1,605 (110)	0.010	−0.080	1.00	0.020	152 (5.99)	0.30	1.10

Note: Parameters are based on 305 mm (1 ft) length of the sheathing.
[a]19 × 184 mm (0.75 × 7.25 in.) horizontal wood siding fastened to each vertical wall stud with two 8d common nails.
[b]12.7-mm (0.5-in.) gypsum wall board fastened with #6 bugle head coarse threaded drywall screws.
[c]12 mm (15/32 in.)-thick sheathing-rated plywood fastened with 10d common nail.
[d]9.5 mm (3/8 in.)-thick sheathing-rated plywood fastened with 6d common nail.
[e]610-mm (2-ft) cross laminated timber used in the retrofitted building.

to be rigid diaphragms (i.e., no out-of-plane deformation or in-plane shear deformation), which have been shown to be effective (Pei and van de Lindt 2009), but it is acknowledged that this is an area that needs more work in wood-frame building modeling. The 10 parameters for the horizontal wood siding (HWS) and gypsum wallboard (GWB) used to construct the four-story wood-frame building model are presented in Table 3 (fifth and sixth rows). These parameters were developed based on tests of wood-frame shear walls sheathed with HWS and GWB at Colorado State University (Bahmani and van de Lindt 2014). The 10-parameter spring model was fitted to backbone curves provided by FEMA P-807 and was used to verify the FEMA P-807 base retrofits. Fig. 5 presents the two spring elements used in numerical modeling of the building with and without retrofits. Fig. 5(a) shows the 10-parameter hysteretic curve used in modeling the existing sheathing, wood structural panel, and CLT panel. Fig. 5(b) presents the bilinear spring used for modeling the steel special moment frames.

Then the numerical model was subjected to 22 far-field biaxial earthquake acceleration records [FEMA P695 (FEMA 2009)] at the MCE level and the maximum drifts in both directions under biaxial analysis recorded and rank ordered. Fig. 6 presents these rank-ordered peak ISD ratios in the form of probability of nonexceedance versus the ISD ratio (Fig. 6). The lognormal distribution was fit to the data in order to interpolate the drifts between the points. Fig. 6 shows much larger ground-floor drifts than the upper stories, reconfirming that the ground level is very soft and weak in both the X- and Y-directions. Furthermore, these graphs illustrate the specific behavior that these buildings are believed to exhibit during earthquakes, specifically the box-like rigid body movement of the upper stories without experiencing significant interstory drifts. The numerical collapse study that was conducted by Pang et al. (2012) showed that this wood-frame building would be expected to collapse at ISD ratios between 11 and 16%. Fig. 6 shows that at 50% probability of nonexceedance, the first story (i.e., soft story) experiences 13.5 and 12.5% at the MCE level in the X- and Y-directions, respectively, essentially indicating at least a 50% collapse probability at the MCE level. Some earthquakes resulted in ISD ratios well in excess of 16%. These ISD ratios were shown and used in producing cumulative distribution function (CDF) curves presented in Fig. 6, but they likely extend beyond the accuracy of models.

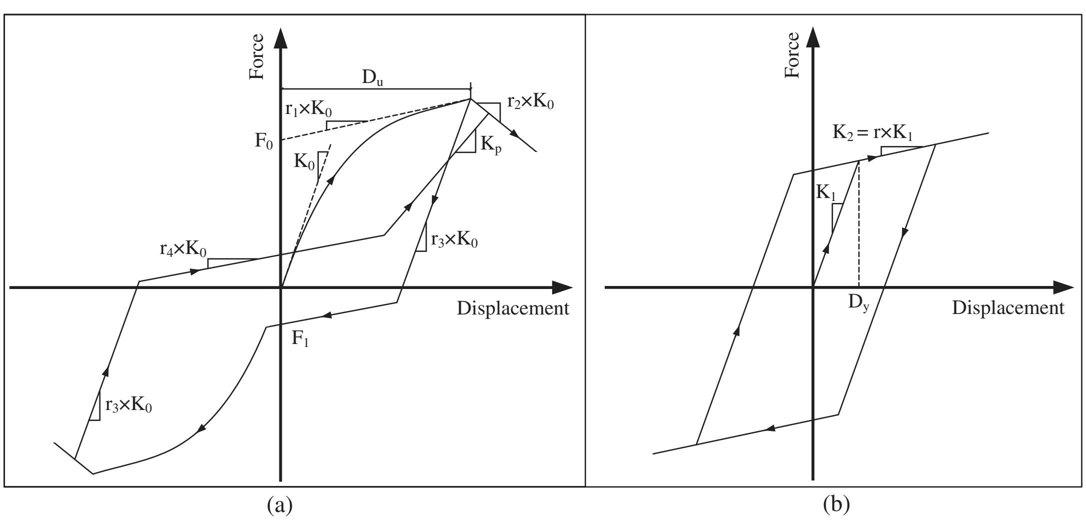

Fig. 5. Spring elements used in numerical model: (a) 10-parameter hysteretic spring; (b) bilinear spring

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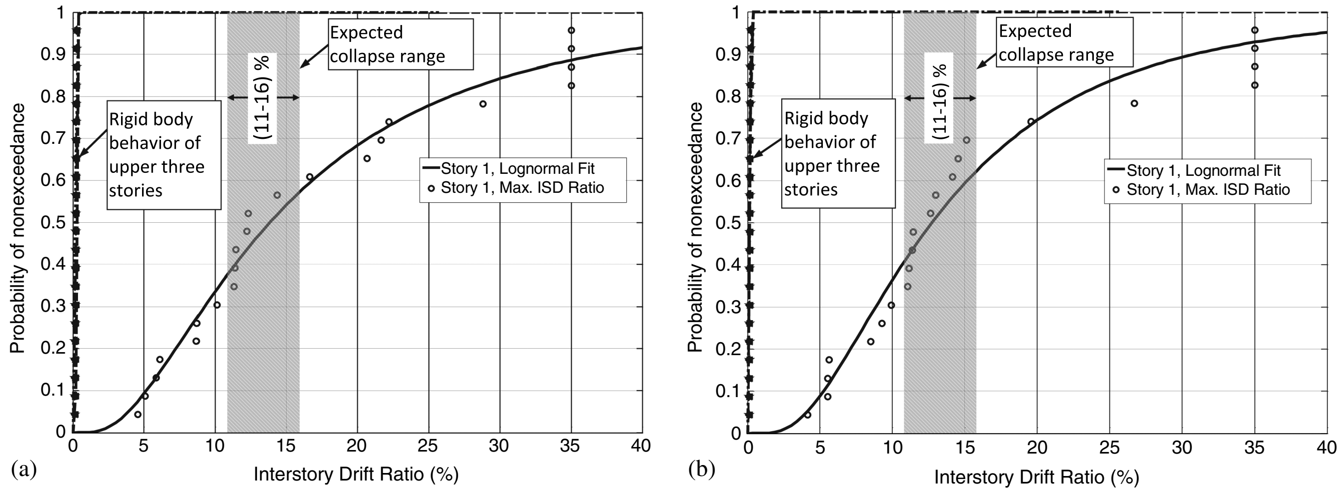

Fig. 6. Probability of nonexceedance versus interstory drift ratio of the unretrofitted four-story building in the (a) X-direction; (b) Y-direction

Phase 1: CLT Rocking Walls (P-807)

The FEMA P-807 methodology was applied to retrofit the ground story using CLT rocking walls. In the P-807 retrofit guideline, the goal is to achieve an acceptable performance by limiting the retrofit to the bottom story (i.e., the soft and weak story) to reduce the cost and duration of the retrofit. CLT is an engineered wood product made up of cross-oriented layers of dimension lumber that is either glued or mechanically connected. The panels themselves behave almost rigidly and the hysteresis is developed using metal connectors, brackets, and hold downs (Popovski et al. 2010; Pei et al. 2013). CLT is a technology that was developed in Europe almost 20 years ago but is only beginning to be used in North America. No seismic provisions for CLT are available in the United States yet, but a project to develop seismic response factors is recently underway and a handbook for engineering guidance was recently published (Karacabeyli and Douglas 2013). The details of the CLT rocking wall assemblies that were used to retrofit the ground story of the four-story building are presented in Fig. 7. The panels were designed such that they could rock freely using vertically slotted holes at the top shear transfer connection. The 16 mm (5/8 in.)-diameter threaded rods were used at each side of the CLT panels to resist the overturning moment and the rods were designed such that they yield if needed, thereby dissipating energy from the earthquake and providing the necessary ductility to the assembly. Shear connectors were added to the CLT panels to transfer the shear force from the CLT panel to the base steel (i.e., foundation) using a metal connector and 6.5 mm (1/4 in.)-diameter self-tapping wood screws.

In order to include the CLT panels into the retrofit procedure the hysteretic backbone of 610 mm (2 ft)-long CLT panels were obtained by using the data from the experimental tests conducted at the University of Alabama (van de Lindt et al. 2013). Then the 10-parameter hysteretic model described previously was fit to the experimental data and used within the FEMA P-807 *Weak Story Tool* (FEMA 2012) and for the NLTHA results presented in this paper. Fig. 8 presents the backbone curve [for use in the *Weak Story Tool* (*WST*)] and the fitted hysteresis curves for a single CLT panel. The 10 hysteretic parameters for CLT panels are presented in Table 3.

The *WST* is a basic software package that is part of FEMA P-807 and accesses the large database of surrogate structures based on user input to determine whether or not a user-inputted retrofit solution meets the user-specified criteria and guidelines.

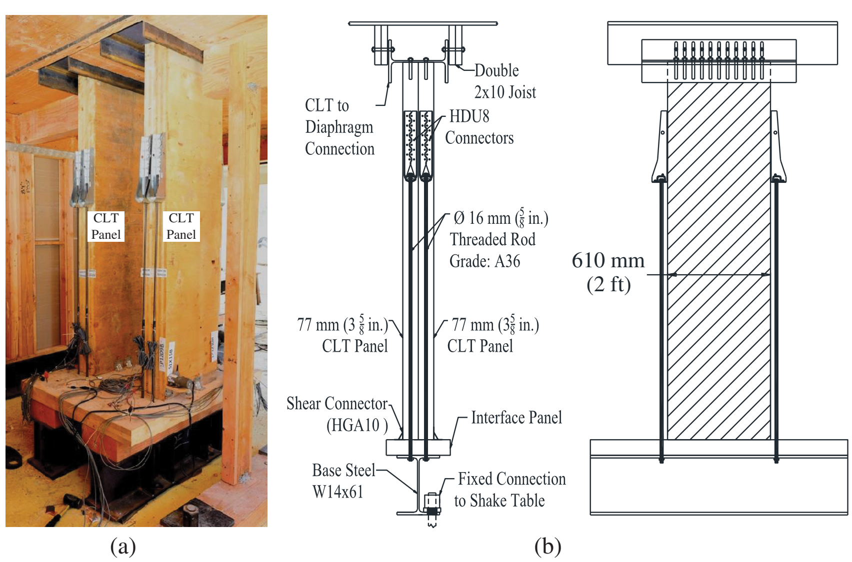

Fig. 7. CLT panels: (a) CLT installed in first story; (b) elevation views and design details

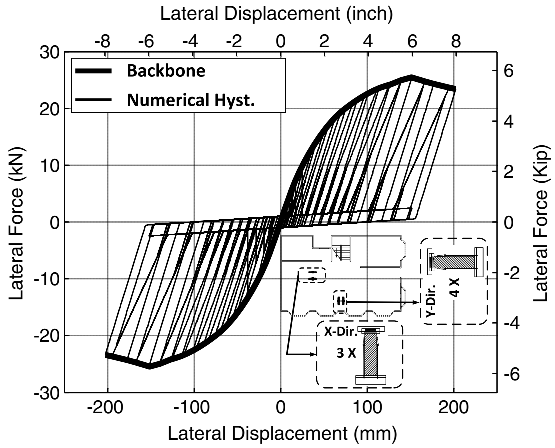

Fig. 8. Hysteresis and backbone curve of a single CLT panel

However, as one can imagine there are in theory an infinite number of solutions for retrofitting a building. Thus, while the FEMA P-807–based solutions applied in this study satisfy the guidelines, it is important to remember that they are not the only solution that could be selected by an engineer. The CLT panels were placed based on basic constraints such as the need to park vehicles after the retrofit and still meet the guidelines of FEMA P-807. Three and four 610 mm (2 ft)-long CLT panels were installed at the ground story in the X- and Y-directions, respectively, as shown in Fig. 9.

In order to verify the effectiveness of the retrofit design, the retrofitted building with CLT panels was analyzed using NLTHA and was subjected to 44 uniaxial earthquake records (FEMA 2009) scaled to a spectral acceleration of 0.9 g using the scaling approach outlined by ASCE 7-10 (ASCE 2010). The building was analyzed uniaxially in each direction independently using 44 ground motions because the FEMA P-807 guideline is based on evaluating the buildings in each direction separately. Fig. 10 presents the rank-ordered peak ISD ratios in the form of probability of nonexceedance versus interstory drift ratios when subjected to the 44 uniaxial far-field earthquake ground motions. The ISD ratios were limited to 16%, which is the upper limit of the collapse range of wood-frame buildings based on the numerical study by Pang et al. (2012). It can be seen that the ISD ratios corresponding to a PNE of 80% (i.e., POE of 20%) is 3.8% in both the X- and Y-directions, which is below the 4% ISD ratio believed to be the onset of collapse as defined by the FEMA P-807 guideline. This ISD ratio and whether it is the onset of collapse is investigated experimentally in the companion paper (van de Lindt et al. 2014). The panels in the Y-direction ensured that the retrofit design met the center of rigidity requirements.

Phase 2: Steel Special Moment Frame and Wood Structural Panels (P-807)

The second retrofit design utilized a steel SMF combined with WSPs and was also in accordance with the FEMA P-807 guideline for a 20% POE but using a spectral acceleration of 1.1 g in the criteria. The column to foundation connection of the steel SMF was a pinned base, making field installation easier and more importantly eliminating the base moments to eliminate epistemic uncertainty from the retrofit design. The SMF retrofit was compatible with the high-ductility class of retrofit material assumptions used in P-807. The ability of the frame to perform with this installation has been demonstrated (van de Lindt et al. 2011). Furthermore, the steel SMF has several unique advantages that make it ideally suited to this particular application (Pryor and Murray 2013). First, complicated and often challenging out-of-plane bracing necessary to prevent lateral-torsional buckling of yielding steel beams in typical SMFs can be eliminated due to the unique design of the beam-to-column connection. Next, the frame is easily assembled on site using bolted, not welded, connections. This eliminates the fumes and potential fire hazard associated with field welding traditional SMF beam-to-column connections in an unprotected wood structure. The field-bolted frame connections do not require special training or tools because they only need to be installed snug-tight. Fig. 11 shows the details of the steel SMF installed in the first story (X-direction) of the test building.

In order to numerically evaluate the performance of the retrofitted building, the steel SMF and wood structural panels were modeled as bilinear and 10-parameter springs, respectively. The stiffness parameters for the steel special moment frames were obtained numerically by conducting pushover analysis for the frames in both directions. Fig. 12 presents the backbone curves of the steel SMF obtained from pushover analysis in both principal directions of the building. The steel SMF elements can be modeled as

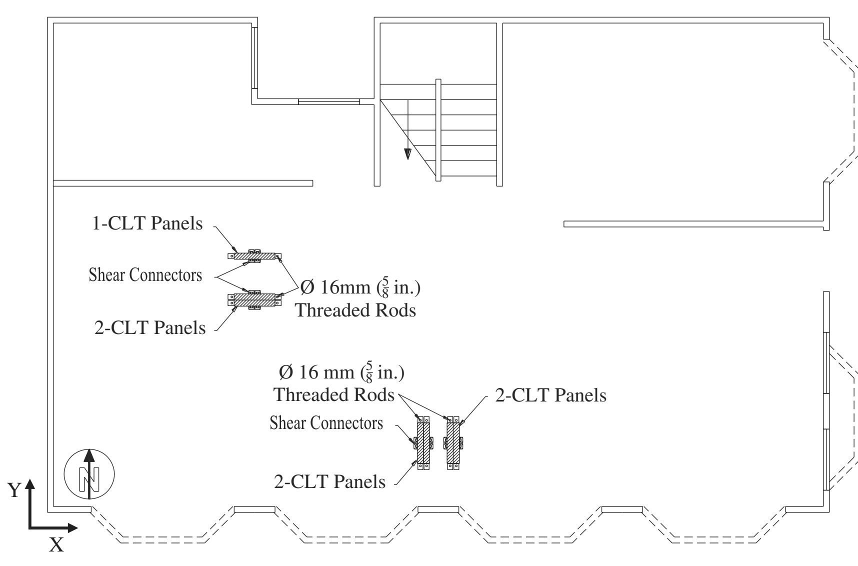

Fig. 9. Location of CLT panels in the ground story

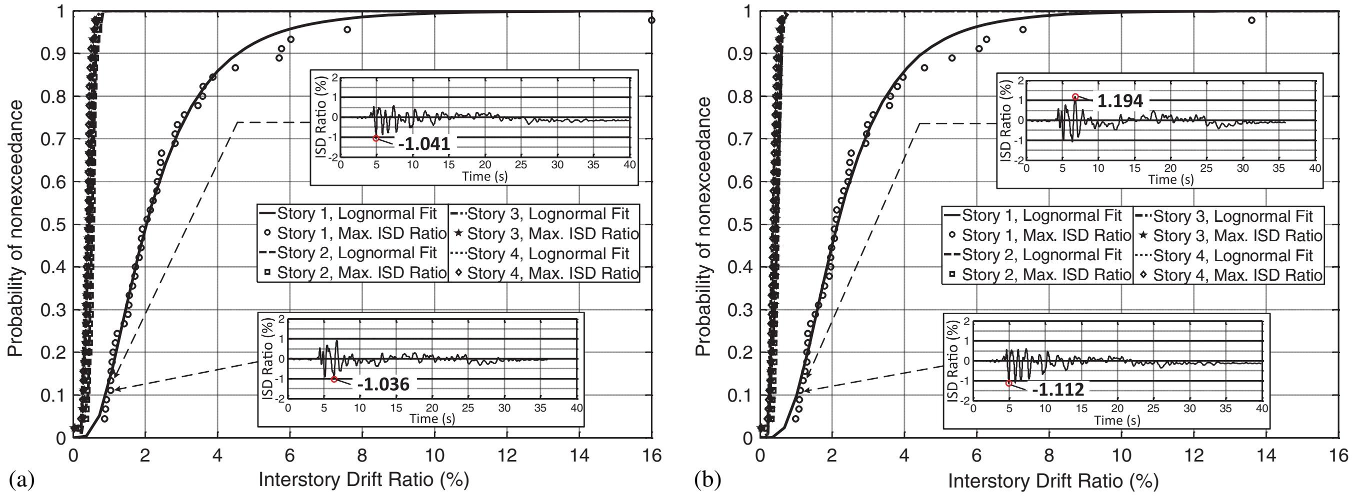

Fig. 10. Probability of nonexceedance versus interstory drift ratio of the four-story building retrofitted in accordance with P-807 guideline using CLT panels in (a) X-direction; (b) Y-direction

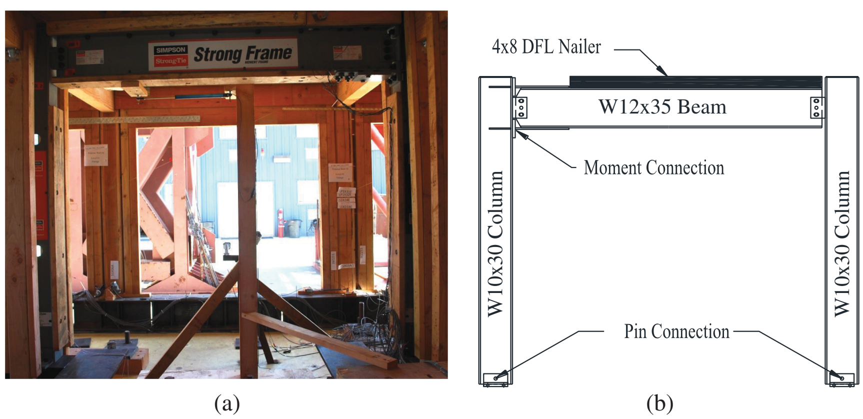

Fig. 11. Steel SMF: (a) frame installed in ground story; (b) elevation views and design details

trilinear springs; however, because only 10% of earthquake records resulted in interstory drifts greater than 4%, bilinear springs with initial stiffness of K_1, stiffness degradation ratio of r, and yield displacement of D_y were used to simplify modeling of the SMF. The steel sections that were installed at ground level and the corresponding lateral resisting parameters are presented in Table 4. WSPs were also used at the ground level in conjunction with

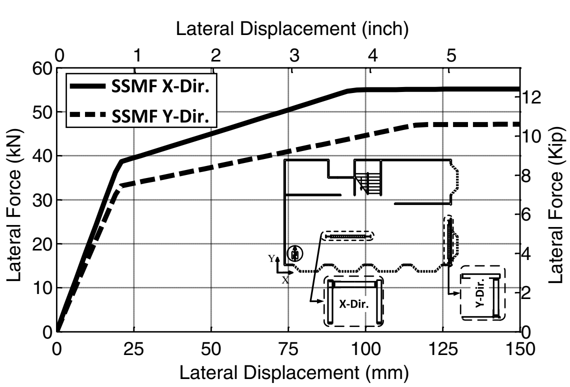

Fig. 12. Backbone curve of a steel SMF in both X- and Y-directions

the steel SMF. In the retrofit design, 12 mm (15/32 in.)-thick wood structural panels with 10d common nails with shank diameter of 3.8 mm (0.148 in.) and shank length of 76.2 mm (3.0 in.) were used as part of the lateral load resisting element at the ground level. The FEMA P-807 retrofit met the criteria with 1,320 mm (4.33 ft)-wide plywood panels with 152 mm (6 in.) on center edge nail spacing and 305 mm (12 in.) field nail spacing in the X-direction and 2,114 mm (6.94 ft)-wide plywood panels with 76 mm (3 in.) o.c. edge nail spacing and 305 mm (12 in.) field nail spacing in the Y-direction. The WSPs in the X-direction (i.e., WSP-A) and Y-direction (i.e., WSP-1) are shown in Fig. 13 and Table 3 provides the 10 parameters used in the numerical modeling for each shear wall per linear foot.

Fig. 13 presents the location of the steel SMF and wood structural panels in both X- and Y-directions in the first story. The steel SMFs were installed such that they did not interfere with the garage space at the ground level and at the same time could transfer the lateral loads from the second-floor diaphragm to the foundation. In the design it was assumed that the floor behaved as a rigid body and that if the engineer felt that it needed strengthening and/or stiffening this would need to be part of the retrofit design, but it is not detailed in this paper. The companion paper (van de Lindt et al. 2014) expands on the need for diaphragm retrofit using plywood sheathing when the added lateral force resisting systems such as the

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Table 4. Steel Special Moment Frame Specifications

Type of resisting element	Retrofit methodology	Column and beam sections	K_1 [N/mm (lb/in.)]	r^a	D_y [mm (in.)]
Steel SMFs[b]	FEMA P-807	Column: W10 × 30 Beam: W12 × 35	1,909 (10,900)	0.113	19.1 (0.75)
		Column: W10 × 26 Beam: W12 × 26	1,634 (9,330)	0.089	19.1 (0.75)
	PBSR	Column: W14 × 38/W16 × 57 Beam: W12 × 50	12,890 (73,600)	0.075	18.0 (0.71)
		Column: 10 × 30 Beam: 12 × 30	6,147 (35,100)	0.107	25.7 (1.01)

[a] $r = K_2/K_1$ [Fig. 5(b)].
[b] All steel is A992 for beams and columns and A572 Grade 50 for links and plates.

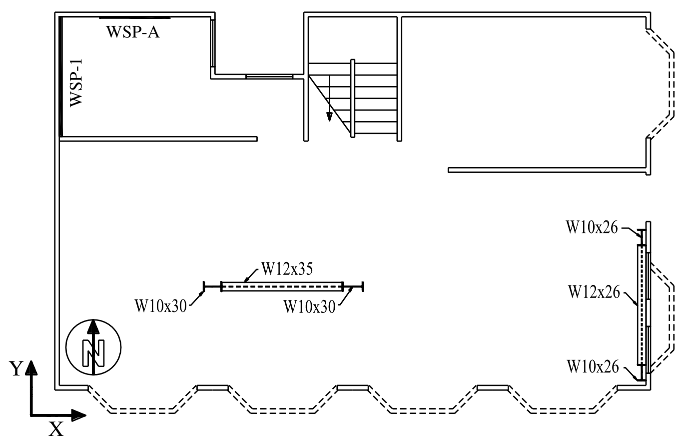

Fig. 13. Location of steel special moment frames and wood structural panels in the ground story

steel SMF (and CLT rocking walls described previously) are validated experimentally.

The behavior based on peak interstory drifts of the retrofitted building was verified using NLTHA by subjecting the building model to 44 ground motions in each direction separately, i.e., the same approach used in the CLT retrofit verification. Fig. 14 presents the rank-ordered peak ISD in the form of probability of nonexceedance versus interstory drift ratios for this analysis. The ISD corresponding to probability of nonexceedance of 80% was found to be 3.8% in both principal directions. Again, this is well in line with the FEMA P-807 guideline.

Phase 3: Steel Special Moment Frame and Wood Structural Panels (PBSR)

The third retrofit was a performance-based seismic retrofit rather than a retrofit based on FEMA P-807. As mentioned previously, in the PBSR procedure the objective is to design the building such that all the stories experience (approximately) the same level of peak interstory drift. Unlike ground story–only retrofits, this utilizes the capacity of the upper stories to resist seismic loads, thereby better distributing seismic demand that is needed for higher earthquake intensities. One drawback is that with distributed seismic demand comes distributed damage, and thus the damage is no longer isolated to the ground floor and may occur at all floors. However, this level of damage is likely nonstructural and very minor at seismic intensities in line with the FEMA P-807 retrofit designs and can be designed to be minor at higher intensities (e.g., MCE) based on the PBSR specification by the designer. To achieve the PBSR goal, the retrofit design for the four-story wood-frame building utilized two steel SMFs in each direction at the ground level and 12 mm (15/32 in.)-thick sheathing-rated plywood shear wall panels with different nail schedules and tie downs on several selected walls of the upper stories. The performance criteria for the PBSR methodology was set not to exceed a 2% ISD with a probability of nonexceedance of 50% at MCE intensity for all stories.

Fig. 15 presents the details of the steel SMF installed in the longitudinal direction (X-direction) of the building. Fig. 16 presents the steel frame at the first story and wood structural panels and anchor tie-down system (ATS) rods with stud packs at the upper stories. The ATS rods were fixed at the foundation (base steel in the test building) and connected to the sill plates at each floor.

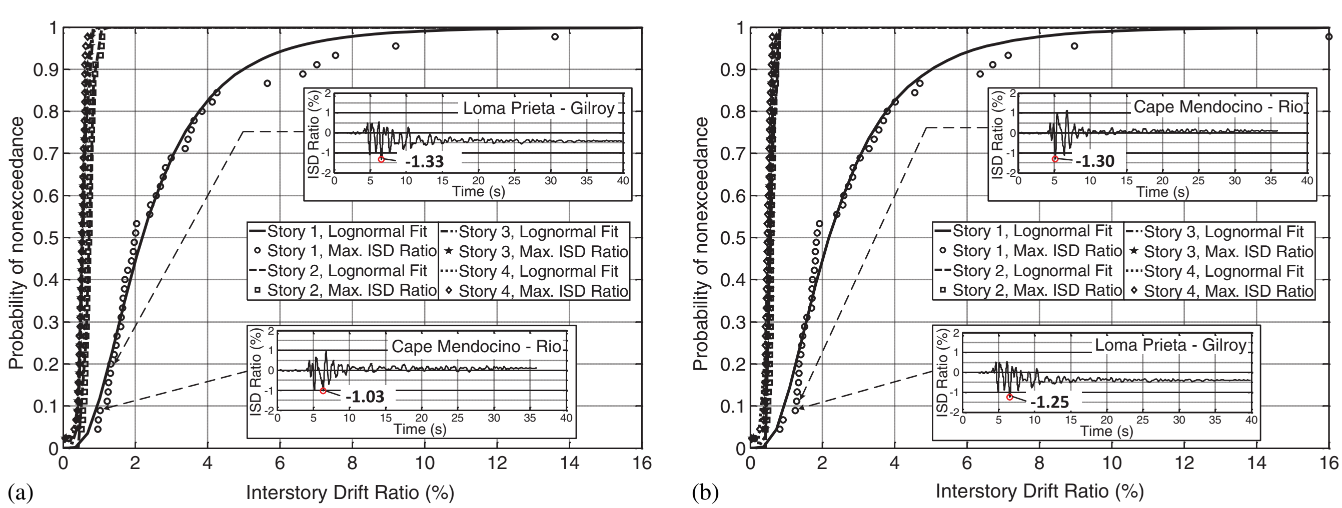

Fig. 14. Probability of nonexceedance versus interstory drift ratio of the four-story building retrofitted in accordance with P-807: SMF and WSP in (a) X-direction; (b) Y-direction

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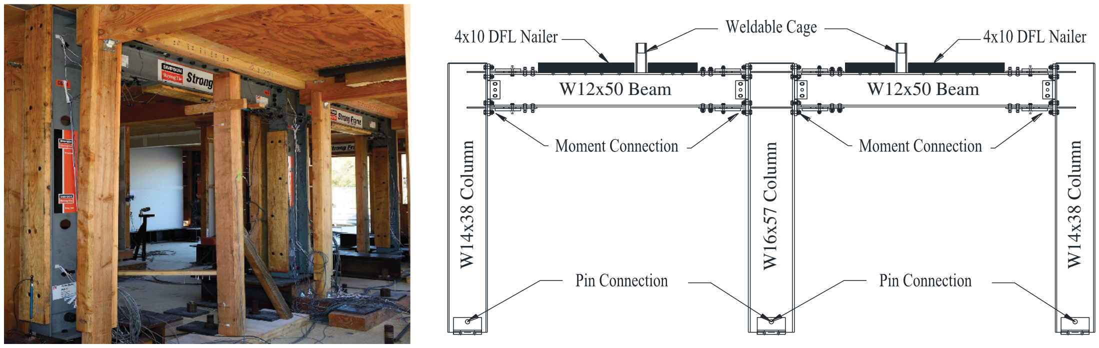

Fig. 15. Steel SMF: (a) frame installed in first story; (b) elevation views and design details

Fig. 16. (a) East span of strong frame installed parallel to the motion of shake table; (b) ATS rods and stud pack prior to attach plywood; (c) plywood panels at upper stories

Table 5. PBSR Retrofit Design Details and Descriptions

		Wood shearwall and fastener properties				
Story	Design properties	WSP-A	WSP-D	WSP-1	WSP-3	WSP-3R
4	Sheathing type[a]	Single[b]	Double	—	Single[b]	Single
	Edge/field nail spacing [mm (in.)]	305/305 (12/12)	153/305 (6/12)	—	305/305 (12/12)	76/305 (3/12)
	ATS rod diameter [mm (in.)][c]	13 (1/2)	16 (5/8)[d]	—	13 (1/2)	16 (5/8)[d]
3	Sheathing type	Single	Double	—	Double	Single
	Edge/field nail spacing [mm (in.)]	153/305 (6/12)	76/305 (3/12)	—	76/305 (3/12)	76/305 (3/12)
	ATS rod diameter [mm (in.)][c]	16 (5/8)	29 (1-1/8)[d]	—	25 (1)[d]	22 (7/8)[d]
2	Sheathing type	Single	Double	—	Double	Single
	Edge/field nail spacing [mm (in.)]	76/305 (3/12)	51/305 (2/12)	—	51/305 (2/12)	51/305 (2/12)
	ATS rod diameter [mm (in.)][c]	22 (7/8)[d]	38 (1-1/2)[d]	—	35 (1-3/8)	29 (1-1/8)[d]
1	Sheathing type	Single	—	Single	—	—
	Edge/field nail spacing [mm (in.)]	51/305 (2/12)	—	76/76 (3/12)	—	—
	ATS rod diameter [mm (in.)][c]	29 (1-1/8)[d]	—	Hold down[e]	38 (1-1/2)[d]	32 (1-1/4)[d]

[a]All sheathing is 12 mm (15/32 in.)–thick sheathing-rated plywood with 10d common nail unless otherwise noted.
[b]9.5 mm (3/8 in.) sheathing-rated plywood with 6d common nail.
[c]Standard steel threaded rod with minimum $F_u = 400$ MPa (58 ksi) and $F_y = 296$ MPa (43 ksi) unless otherwise noted.
[d]High-strength steel threaded rod with minimum $F_u = 827$ MPa (120 ksi) and $F_y = 634$ MPa (92 ksi).
[e]Hold down with allowable tension force of $F_t = 31$ kN (6.97 kips).

Elongation is evaluated by multiplying the measured strain by the length of the rod. Each ATS rod was confined with a stud pack for compression forces. The ATS rods were sized such that they would only elongate 6.4 mm (1/4 in.) at each story when the wall reaches 80% of its ultimate shear capacity, assuming a rigid-body free-body diagram approach for force computation. Similar to the FEMA P-807–based retrofits, it was assumed that the floor behaved as a rigid body; thus, the underside of the second floor diaphragm was sheathed with plywood to ensure sufficient diaphragm strength. Table 5 presents a summary of the different types of wood structural panels, fastener spacing, and type of tie downs (or hold downs) used to retrofit the test building.

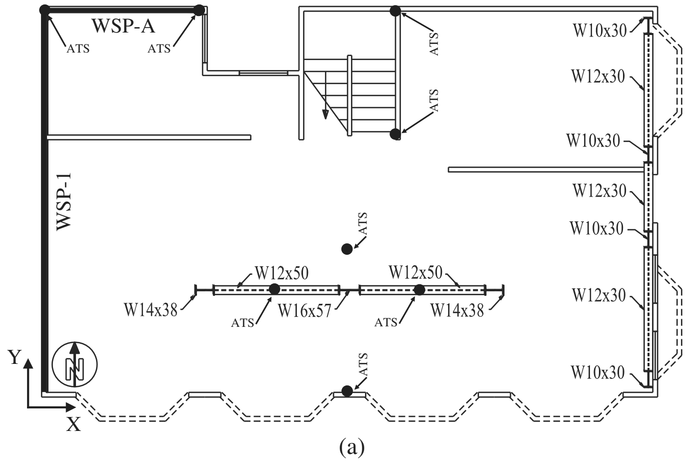

Fig. 17. Location of the PBSR retrofits for Phase 3 testing at the (a) first story; (b) upper stories

Table 6. Eccentricity and Distribution of Stiffness at Each Story Using PBSR Method

Story	Target ISD Δ^{Target} (%)	Secant Stiffness at Δ^{Target} [kN/mm (kip/in.)]						Eccentricity[a] at Δ^{Target} (%)			
		X-direction[b]			Y-direction[b]			Before retrofit		After retrofit	
		$K_{\mathrm{req.}}$	$K_{\mathrm{avail.}}$	$K_{\mathrm{ret.}}$	$K_{\mathrm{req.}}$	$K_{\mathrm{avail.}}$	$K_{\mathrm{ret.}}$	e_x/L_x	e_y/L_y	e_x/L_x	e_y/L_y
4	2	2.45 (14.0)	1.01 (5.79)	1.44 (8.21)	2.45 (14.0)	1.47 (8.39)	0.98 (5.61)	10.6	15.6	3.00	2.22
3	2	4.99 (28.5)	1.01 (5.79)	3.98 (22.7)	4.99 (28.5)	1.47 (8.39)	3.52 (20.1)	10.6	15.6	2.45	1.14
2	2	6.69 (38.2)	1.01 (5.79)	5.67 (32.4)	6.69 (38.2)	1.47 (8.39)	5.22 (29.8)	10.6	15.6	1.53	0.56
1	2	7.55 (43.1)	0.69 (3.96)	6.85 (39.1)	7.55 (43.1)	0.58 (3.31)	6.97 (39.8)	12.9	29.7	0.82	1.07

[a]Eccentricity are defined as $e_x = |X_{\mathrm{CM}} - X_{\mathrm{CR}}|$ and $e_y = |Y_{\mathrm{CM}} - Y_{\mathrm{CR}}|$.
[b]$K_{\mathrm{req.}}$ = required stiffness; $K_{\mathrm{avail.}}$ = available stiffness before adding the retrofit; $K_{\mathrm{ret.}}$ = retrofit stiffness ($K_{\mathrm{req.}} = K_{\mathrm{avail.}} + K_{\mathrm{ret.}}$).

The steel SMFs were designed and located such that they did not interfere with the intended use of the space (i.e., vehicle parking) or conflict with any other architectural aspect of the building. Fig. 17 presents the location of the SMF, wood shear walls (WSPs), and ATS rods that were installed to retrofit the building. Also, at the ground level hold downs were used to transfer the uplift forces for the WSP-1 shear wall. Both the steel SMF and wood shear walls were placed such that the center of rigidity moved toward the center of mass at each story, which was to effectively eliminate torsion. Table 6 presents the design stiffness and calculated eccentricities at each story before and after adding the retrofit based on using DDD procedure (i.e., PBSR procedure) described previously. The eccentricities for the retrofit design are very close to zero, confirming that torsion was effectively eliminated at the target interstory drift ratio (i.e., ISD ratio = 2%).

The behavior of the retrofitted building from the PBSR methodology was evaluated using NLTHA with 22 biaxial far-field ground motions scaled to the MCE level, i.e., a spectral acceleration of 1.8 g. In order to conduct the NLTHA, the SMF and WSP were modeled using bilinear and 10-parameter spring models, respectively (Fig. 5). The spring parameters presented in Tables 3 and 4 were used to model WSP and SMF, respectively. Similar to the steel SMF used in the P-807 based retrofits, the stiffness parameters for the steel SMF in the PBSR retrofit were obtained numerically by conducting pushover analysis for the frames in both directions to ensure that they provided the desired strength and ductility for the ground level. Fig. 18 presents the backbone curves for the steel SMFs in both principal directions of the building.

Biaxial analyses were utilized for the verification of the PBSR because it is a performance-based method and not necessarily tied to either uniaxial or biaxial analysis. Fig. 19 presents the rank-ordered peak ISD ratios in the form of probability of nonexceedance versus interstory drift ratios when subjected to the 22 biaxial far-field earthquakes. Again, a lognormal distribution was fit to the data in order to interpolate the drifts between the points and provide a general feel of the shape of the response CDF. Inset in Fig. 19 are several time history analysis responses for illustration. The two time history responses selected here are for the two earthquakes that were eventually used for full-scale shake table validation in the companion paper (van de Lindt et al. 2014). More detailed design information and shake table test results on utilizing steel special moment frames for retrofitting wood-frame buildings is discussed

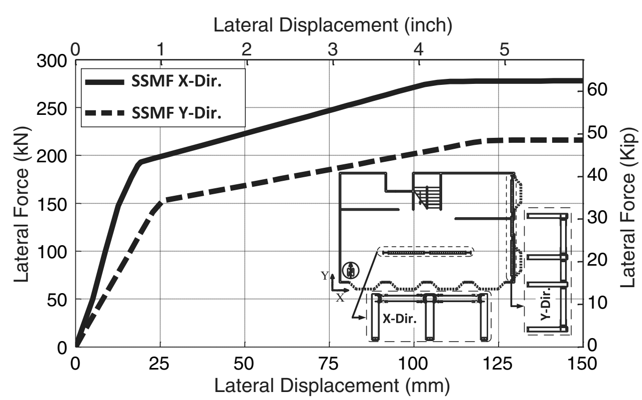

Fig. 18. Backbone curve for the steel SMFs in both the X- and Y-directions

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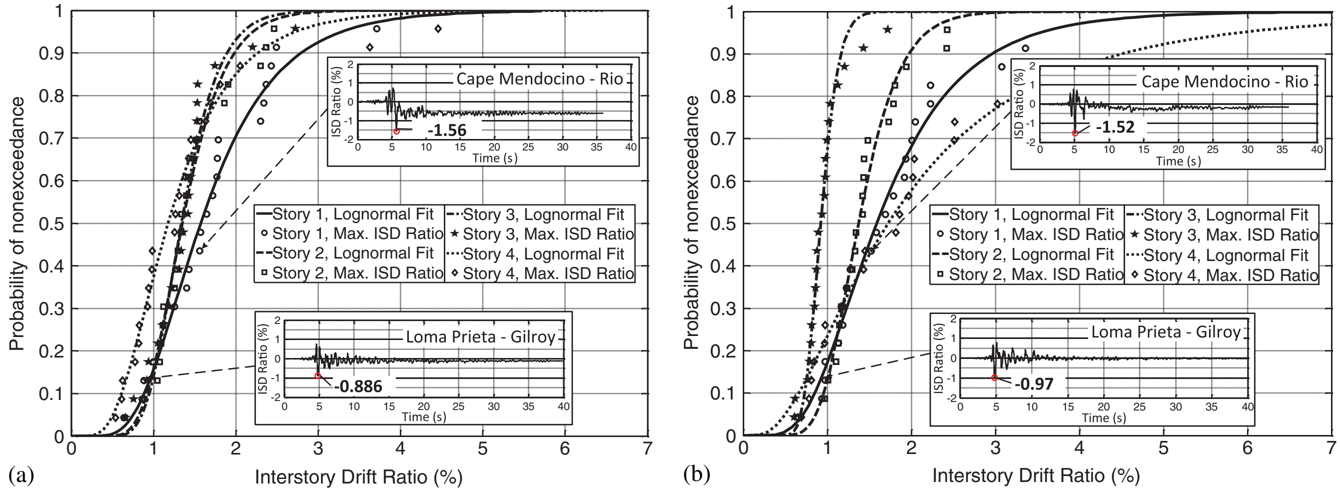

Fig. 19. Probability of nonexceedance versus interstory drift ratio of the four-story building retrofitted in accordance with PBSR: SMF and WSP in the (a) X-direction; (b) Y-direction

by Bahmani et al. (2014b). From Fig. 19, it can be seen that the maximum ISD ratio at the first story (i.e., governing story for this building) corresponding to 50% probability of nonexceedance is 1.65%, which is less than the 2% target ISD ratio. The maximum interstory drift ratio in the Y-direction is 1.85%, which is close to the target ISD ratio, i.e., only 7% error. The ISD ratios in all other stories are between 1 and 2%, which numerically confirms the effectiveness of the retrofit procedure in utilizing the strength of upper stories, and more importantly eliminating the existence of soft and stiff stories in the building.

Two more test phases were conducted during the NEES-Soft shake table test program, which are discussed briefly in this paper: Phase 4 of retrofit testing was conducted using a PBSR approach via NLTHA and utilized nine viscous fluid dampers at the first floor as the retrofit. The shearwall pattern for the upper stories remained the same as what was used in Phase 3 with one difference, namely, there were no WSPs used at the first floor. The detail of design methodology and results for the damper phase (Phase 4) was discussed in detail by Tian et al. (2014). Finally, in order to investigate the collapse margin of the building, all the retrofits were removed and the building was subjected to different ground motions with increasing intensity until it collapsed (Bahmani et al. 2014a).

Summary and Conclusions

In this first of two companion papers, two different approaches for retrofitting soft-story wood-frame buildings were examined, namely, (1) the retrofit guideline explained in FEMA P-807, and (2) a PBSR procedure that utilizes the direct displacement design concepts but for retrofit. The FEMA P-807 retrofit approach focuses on retrofitting only the bottom story without adding any stiffness or strength to the upper stories, whereas the PBSR procedure focuses on the performance of the building and limiting the ISD ratios to a target ISD ratio for high-intensity ground motions (i.e., MCE level), essentially distributing the seismic demand to all stories as evenly as possible. The retrofit methodologies, design of retrofit elements, and numerical validation for each retrofit technique was presented in this paper and the experimental shake table validation and full-scale shake table test results are presented in the companion paper.

The building was retrofitted using four different types of retrofit elements used in combination: (1) CLT rocking walls, (2) steel SMFs, (3) fluid viscous dampers (FVDs), and (4) WSPs with different nail schedules. The retrofit designs were validated numerically using nonlinear time history analysis by subjecting the building to a suite of earthquake records. It was shown that the retrofits used in the study were adequate to meet the prescribed performance criteria.

Acknowledgments

This material is based on work supported by the National Science Foundation under Grant No. CMMI-1041631 and 1314957 (NEES Research) and NEES Operations. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the investigators and do not necessarily reflect the views of the National Science Foundation. A sincere thank you to Simpson Strong Tie for their financial, personnel, and product support throughout the project, including the engineering support for the SMF in San Diego. Cash, personnel, or in-kind contributions were provided by the USDA Forest Products Laboratory, Structural Engineers Association of Southern California, Taylor Devices, and Innovative Timber Solutions/Smartwood. The authors kindly acknowledge the other co-principal investigators (Co-PI's) of the project, Weichiang Pang and Xiaoyun Shao, and other senior personnel of the NEES-Soft project: David V. Rosowsky at University of Vermont, Andre Filiatrault at University at Buffalo, Shiling Pei at South Dakota State University, David Mar at Tipping Mar, and Charles Chadwell at Cal-Poly San Luis Obispo; the other graduate students participating on the project: Ph.D. students Ershad Ziaei (Clemson University) and Jingjing Tian (Rensselaer Polytechnic Institute) and M.Sc. students Jason Au and Robert McDougal at Cal-Poly Pomona; and the practitioner advisory committee: Laurence Kornfield, Tom Van Dorpe, Doug Thompson, Kelly Cobeen, Janiele Maffei, Douglas Taylor, and Rose Grant. A special thank you to all of the research experience for undergraduate (REU) students: Sandra Gutierrez, Faith Silva, Gabriel Banuelos, Rocky Chen, and Connie Tsui. Others that have helped include Asif Iqbal, Andre Barbosa, Vaishak Gopi, Steve Yang, Ed Santos, Tim Ellis, Omar Amini, Russell Ek, Rakesh Gupta, and Anthonie Kramer. Finally, our sincere thank you to NEES and all site staff and site Principal Investigators (PI's) at NEES at the University of California San Diego for their assistance in getting the test specimen ready for testing and in conducting the tests.

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