PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

Project Technical Summary

A Report for the “Quantifying the Performance of Retrofit of Cripple Walls and Sill Anchorage in Single-Family Wood-Frame Buildings” Project

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PEER Report 2020/12
Pacific Earthquake Engineering Research Center
Headquarters, University of California at Berkeley

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ABSTRACT

This report is one of a series of reports documenting the methods and findings of a multi-year, multi-disciplinary project coordinated by the Pacific Earthquake Engineering Research Center (PEER) and funded by the California Earthquake Authority (CEA). The overall project is titled "*Quantifying the Performance of Retrofit of Cripple Walls and Sill Anchorage in Single-Family Wood-Frame Buildings*," henceforth referred to as the "PEER-CEA Project."

The overall objective of the PEER–CEA project is to provide scientifically based information (e.g., testing, analysis, and resulting loss models) that measure and assess the effectiveness of seismic retrofit to reduce the risk of damage and associated losses (repair costs) of wood-frame houses with cripple wall and sill anchorage deficiencies as well as retrofitted conditions that address those deficiencies. Tasks that support and inform the loss-modeling effort are: (1) collecting and summarizing existing information and results of previous research on the performance of wood-frame houses; (2) identifying construction features to characterize alternative variants of wood-frame houses; (3) characterizing earthquake hazard and ground motions at representative sites in California; (4) developing cyclic loading protocols and conducting laboratory tests of cripple wall panels, wood-frame wall subassemblies, and sill anchorages to measure and document their response (strength and stiffness) under cyclic loading; and (5) the computer modeling, simulations, and the development of loss models as informed by a workshop with claims adjustors.

This report is a product of Working Group 7: *Reporting* and is a summary of the PEER–CEA Project work performed by Working Groups 1–6. This report does not present new information apart from the rest of the project, and its purpose is to serve as a reference for researchers and catastrophe modelers wishing to understand the objectives and key findings of the project. The key overall findings of the PEER–CEA Project are summarized in Chapters 8 and 10, which describe the efforts of the WG5 and WG6 Working Groups. The reader is referred to the individual reports prepared by the Working Groups for comprehensive information on the tasks, methodologies, and results of each.

ACKNOWLEDGMENT AND DISCLAIMER

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DISCLAIMER

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Within the PEER–CEA Project, detailed work described above was conducted by seven Working Groups, each addressing a particular area of study and expertise, and collaborating with the other Working Groups. The seven Working Groups are as follows:

Working Group 1: Resources Review

Working Group 2: Index Buildings

Working Group 3: Ground Motion Selection and Loading Protocol

Working Group 4: Testing

Working Group 5: Analytical Modeling

Working Group 6: Interaction with Claims Adjustors and Catastrophe Modelers

Working Group 7: Reporting

This report is a product of the Working Group denoted in bolded text above. The scope of work implemented by the PEER–CEA Project Team is outlined below and formed the guiding process roadmap and the basis of the tasks performed by each of the Project Working Groups. Chapters 2–10 provide more details of the scope of work, methodology, and findings of all working groups. In particular, the key overall findings of the PEER–CEA Project are summarized in Chapters 8 and 10, which describe the efforts of the WG5 and WG6 Working Groups.

The reader is referred to the individual reports prepared by the Working Groups for comprehensive information on the tasks, methodologies, and results of each report: Cobeen et al. [2020]; Mazzoni et al. [2020]; Reis [2020(a); (b)]; Schiller et al. [2020(a); (b); (c); (d); (e)]; Vail et al. [2020]; Welch and Deierlein [2020]; and Zareian and Lanning [2020].

Note: the terms "existing" and "unretrofitted" are used interchangeably in this report.

1.1 TASK 1: LITERATURE REVIEW

The Project Team completed a review of relevant literature during the first six months of the project to ensure that all subsequent efforts will benefit from and complement previous research efforts. The review encompassed the broad range of issues related to the work of the other Project Working Groups, ranging from research on the design and behavior of wood-frame houses, to computational response and loss modeling, to best practices for stakeholder engagement and communication of technical information. Members of the Project Team from all task areas participated in the review to capture a broad perspective that is effectively focused on the objectives of the Project and includes:

(a) *Data Analysis- Identification and Statistical Analysis of Building Inventories*

(b) *Identification and Evaluation of Existing Analytical Models*

(c) *Characterizing the Ground Motion Hazard*

(d) *Software Systems and Models for Nonlinear Structural Analysis*

(e) *Expected Performances*

(f) *Loss Functions*

(g) *Damage and Retrofit of Cripple Walls and Sill Anchorage*

(h) *Seismic Behavior and Performance Assessment of Existing Wood-frame Houses*

(i) *Testing and Loading Protocols*

(j) *Cripple Wall and Sill Anchorage Cost Estimate Information*

(k) *Loss Estimation and Cost-Benefit Assessment as Input to Mitigation Decision Making*

(l) *Communicating Risks and Incentivizing Risk Mitigation*

1.1.1 Milestone Tasks

Preliminary Report. A draft technical report summarizing the literature review findings and, in particular, drawing out the most essential literature and key information essential to date for use by subsequent tasks.

Final Report. A final report incorporating information and feedback from the Preliminary Report and comments from the Project Team.

1.2 TASK 2: ANALYZING BUILDING INVENTORY AND DEFINING REPRESENTATIVE "INDEX BUILDINGS"

The Project Team conducted a careful analysis of existing building inventory and definition of representative Index Buildings to guide subsequent experiments and simulation tasks. The Project Team developed Index Buildings with variants that met two conditions: (a) parameters that have a significant effect on the seismic response of the building; and (b) parameters that have a statistically significant presence in California building stock.

In general practice, catastrophe modelers employ secondary modifiers to refine damage functions (DFs) for their primary classifications, which typically include the structural system, materials, age, and height. These secondary modifiers are unique to each modeler. For example, one modeler's list of secondary modifiers includes material condition, pounding, shape, irregularities, foundation connection, foundation type, wall type and siding, partitions, exterior openings, bracing of water heaters, brick veneer, cripple wall retrofit, soft story, and sill bolting. The Project Team optimized the selection of index variants to include those variants that could potentially become a significant source of damage if retrofitting a cripple wall or sill anchorage "pushes" damage up from the substructure to the superstructure.

1.2.1 Milestone Tasks

List of Building Variants. Development and delivery of a digital archive that will enable both the CEA and Project Team to collect, manage, and access information on typical index houses, earthquake damage to houses, typical cripple wall construction details, and images from prior tests.

Index Buildings to be Tested and Modeled. Drawings and tables of Index Buildings are described in sufficient detail to develop structural analysis and damage models.

1.3 TASK 3: SELECTING GROUND-MOTION RECORDS AND DEVELOPING LOADING PROTOCOLS

Task 3.1: Selecting and Scaling Ground-Motion Records

The purpose of this task is to select and scale a set of earthquake ground-motion records suitable for analytical (i.e., nonlinear response history analysis) investigations for this project. There are different methodologies for selection and scaling of ground-motion records (see, e.g., Haselton et al. [2009]; Baker. [2011]; and NIST [2011]). In this task, the Project Team employed and compared different methods, from which the final selected and scaled motions were recommended for this project.

The Project Team assembled a working group for ground-motion selection and scaling for the project, which identified and selected various sets of "target response spectra" for different typical sites. The working group selected and scaled different sets of ground motions using the target spectra, employing different methodologies for ground-motion selection and modification.

Task 3.2: Adoption of a Loading Protocol

The purpose of this task was to determine if currently available loading protocols were adequate for the testing program, and, if needed, provide an update to those loading protocols. Task 3.2 is tied in with the development of the Index Buildings, in which a set of representative wood-frame buildings on raised foundations in California were compiled; the loading protocols developed were tailored for this target building set.

The Project Team followed the method suggested by Krawinkler et al. [2002] to generate cyclic loading protocols for cripple wall testing by utilizing the validated analytical models from the first stage and current state-of-the-art in ground-motion hazard modeling. The Project Team developed new loading protocols for testing cripple walls with the following considerations:

- The loading protocols were quasi-static (cyclic) for deformation-controlled components;
- Emphasis was placed on performance assessment on a spectrum of seismic hazard return periods (from short- to long-return periods);
- Ground-motion selection and scaling for the purpose of developing loading protocols were conducted as described in Task 3.1, utilizing the PEER NGA-West2 ground-motion database [Ancheta et al. 2013];
- Analytical models for developing loading protocols were geared towards cripple wall characteristics. Validated component models applicable to wood-frame buildings that are representative of light wood frames on raised foundations in California were used; and
- Each loading protocol was anchored to a displacement representing a target performance and independent from subjective parameters such as yield deformation.

1.3.1 Milestone Tasks

Ground Motions: Sets of selected and scaled ground motions appropriate for testing and analytical investigations were developed and a written report documenting the process and outcome provided.

Loading protocols: Loading protocols appropriate for testing and analysis studies were developed and a written report documenting the process and outcome provided.

1.4 TASK 4: EXPERIMENTAL PROGRAM.

In order to develop seismic DFs and ultimately understand losses due to cripple wall and sill anchorage failure, it is essential to have a robust assessment of their physical performance and the impact their response will have on (a) other structural elements in the load path; and (b) the superstructure, including both its structural and nonstructural components. Past cripple wall tests have shed some light on this issue; however, in totality the experimental dataset is sparse and needed to be supplemented to provide the necessary information for numerical model calibration.

The objectives of the Task 4 experimental program were to fill the knowledge gaps identified above and to provide physical data for validating numerical models and characterizing damage states essential for developing DFs. For this purpose, the Project Team identified four key tasks in the proposed experimental program that were undertaken at the testing laboratories of the University of California, Berkeley (UC Berkeley) and the University of California, San Diego (UC San Diego). Collectively, the proposed cyclic tests of wood-frame structural components and subassemblies were devised to produce high-fidelity data to support the evaluation and cost-benefit analysis of the *FEMA P-1100* prestandard developed by the ATC-110 project.

1.4.1 Milestone Tasks

Task 4.1: Literature Survey of Relevant Testing. Past experimental testing efforts and resulting test publications were identified and served to inform priority testing needs for the Working Group 4 Testing Plan. These publications can be found in the Task 1 literature survey. In addition, a digital database of selected past experimental data was assembled.

Tasks 4.2: Tests of Diaphragm-Cripple Wall Subassemblies (Large-Component Testing). Performance data (measurements, photographic, and video documentation) of the earthquake response of assemblies including cripple walls and an occupied story above, with direct comparisons of unretrofitted and retrofitted configurations were provided.

Task 4.3: Quasi-Static Component and Subassembly Tests (Small-Component Testing). Digital database of quasi-static cyclic performance data (measurements, photographic, and video documentation) of the response of the cripple wall components with a variety of variants and including unretrofitted and retrofitted detailing were provided.

Task 4.4: Development of Recommendations for Future Full-Scale Testing. Development of recommendations for future full-scale testing of cripple wall houses, including an overview of the benefits of providing such testing and discussion of what such testing might entail, were provided.

1.5 TASK 5: ANALYTICAL MODELING

Nonlinear dynamic analyses are an integral component of the project to incorporate and translate information from component testing to determine demand parameters (drifts, deformations, and accelerations) of the Index Buildings subjected to earthquake ground motions of varying intensity. One of the key challenges of the nonlinear dynamic analysis is to ensure that the analytical models provide realistic estimates of seismic demands, from the onset of damage through to collapse, while considering the unique characteristics of the Index Building models and taking full advantage of available test data and other information to validate the analyses. Careful validation and calibration are particularly important because past analysis studies of short-period light-frame structures have tended to overestimate demands relative to observed response.

The overall goals and scope of the Working Group 5 analyses were as follows:

- Develop, validate, and calibrate phenomenological component models that reliably simulate the response of wood-frame residential houses with and without cripple wall and sill anchorage weaknesses, and with retrofits to mitigate the weaknesses;
- Examine the implications of site characteristics and other factors that can significantly affect the computed response of short-period light wood-frame structures with strength and stiffness irregularities, such as those caused by cripple walls; and
- Create and analyze models of multiple index house configurations (with varying levels of cripple wall/sill anchorage weaknesses and retrofit) to develop

data on demand parameters under increasing ground-motion intensities, including both ground motion and modeling uncertainties.

The resulting database of demand parameters feed into the damage assessment tasks of Working Group 6.

1.5.1 Milestone Tasks

Task 5.1 Component Model Validation and Calibration: Validated component and building system models to reliably simulate the response of index houses (and effectiveness of retrofit) under varying ground-motion intensity.

Task 5.2 Sensitivity Analyses of Short-Period Response: Site hazard characteristics were incorporated into the nonlinear analysis of stiff short-period residential houses.

Task 5.3 Development of Index House Numerical Models: Validated analysis models of Index Buildings for the subsequent task of developing Index Building response data for loss models.

Task 5.4 Nonlinear Analyses: Database and supporting information of demand parameters for Index Buildings under increasing ground-motion intensity.

Task 5.5 Fragility Function Database: Created a database of fragility functions that formed the basis of the development of the DFs.

Task 5.6: Engineering Demand Parameter (EDP) Function Database: Created a database of Engineering Design Parameters that formed the basis of the development of the DFs.

Task 5.7 Damage Function Development: Damage functions–representations of damage as a function of ground-motion input intensity–for all index buildings and variants considered were developed.

1.6 TASK 6: DEVELOPMENT OF FRAGILITY-MODIFICATION FUNCTIONS

An essential outcome of this task was to develop DFs for the Index Buildings identified, which represent the improvement in building performance achieved through the retrofit of cripple walls and sill anchorage.

The objective of this task was to integrate the test results from the efforts of Working Group 4 and Working Group 5 to populate the EDP database to facilitate the development of DFs for use by catastrophe modeling companies and the insurance industry. Software developed by modeling companies essentially includes three components: stochastic earthquake catalogues, building replacement cost data, and DFs. The scope of this portion of the project was to develop DFs only. *Damage* is differentiated from *Loss* in the insurance industry in that damage represents the actual "ground up" repair cost, whereas loss represents the amount of a claim an insurer is likely to pay, and includes factors to account for deductibles, limits, and thresholds above which a building is considered a total loss. The Project Team's effort was to develop the DFs.

One of the key project challenges that faced the Project Team was to coordinate the DFs developed by the Project for unretrofitted buildings with those of the catastrophe modeling firms. To address this issue, the Project Team devised a "blind" validation process whereby it selected four locations throughout California and ran its loss methodologies on the index at each location to compare with annualized and specific return period values provided by the modelers.

1.6.1 Milestone Tasks

Task 6.1 Project-Catastrophe Model Comparison Framework: Develop a framework for comparing DFs and loss estimates produced by industry catastrophe modelers.

Task 6.2: Claims Adjustor Workshop: Provide a written report of a workshop to augment the fragility functions and inform the catastrophe modeling partners of how the Project's DFs were developed.

Task 6.3 Project-Proprietary Model Comparison: Provide a summary report on the baseline comparison between the Project Team's DFs and that of CEA's catastrophe modeling partners.

Task 6.4 Damage Function Summary and Key Findings: Provide a summary report on the proposed DFs and key findings for use by the catastrophe modelers. Key findings of the DF development are included in the WG5 report (Task 5.7). Key findings of the comparisons with catastrophe modelers are included the WG6 report.

1.7 PEER–CEA PROJECT TEAM MEMBERS

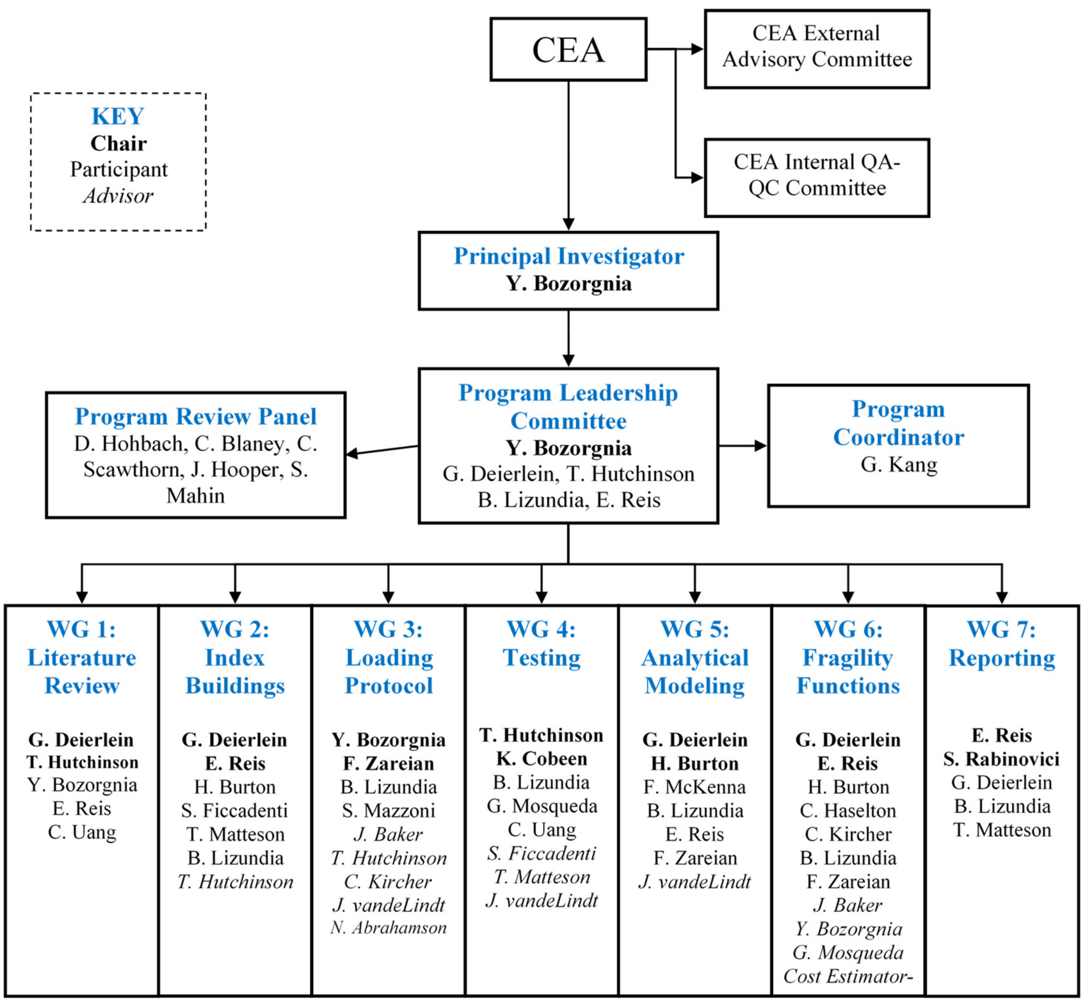


Figure 1.1 PEER–CEA Project Team.

2 Working Group 1: Preparation of Literature Review

Working Group Leaders and Participants: Gregory G. Deierlein, Tara Hutchinson, Yousef Bozorgnia, Sharyl Rabinovici, Evan Reis, and Chia-Ming Uang

2.1 INTRODUCTION

Working Group 1 prepared a literature review as a resource document for the PEER–CEA Project Team. This "Resources Review" aimed at providing easy reference and access to papers, reports, software, and other sources that provide data and information relevant to various aspects of the Project. The objective of WG1 was to support the research and development for the Project Team and may be of use to others. Note: the terms "existing" and "unretrofitted" are used interchangeably in this report.

The resource review is organized into nine sections that cover high-priority areas of knowledge, methods, and data sources that became the historical benchmark for this Project. Under each topic heading are several key research questions. Each topic section contains a table that identifies references that are relevant to the questions initially raised when developing the Project's aims and scope. When reference is relevant to multiple topics, the resources may be mentioned in more than one section. Each of the listed references is annotated with a short description of how the resource is relevant to the project. The ninth and last section lists sources used to produce a working glossary of key terms and concept definitions for the Project. The final Project Glossary is available in Appendix A of this report. Note: the terms "existing" and "unretrofitted" are used interchangeably in this report.

The nine sections and key issues and questions that were researched are:

- Index Buildings and Loss Functions for Wood-Frame Houses
    1. What are the common characteristics (variants) of wood-frame houses in California that should be considered for Index Buildings used to develop DFs?
    2. Do these characteristics meet the three following conditions: (1) are prevalent in California construction; (2) have a large impact on seismic performance; and (3) affect performance differently whether the house is retrofitted with CEA Earthquake Brace and Bolt (EBB) program?
    3. How do these characteristics vary with the age of construction?

4. What are the common descriptive parameters used in DFs for wood-frame houses?

- Behavior and Damage of Cripple Walls and Sill Anchorages
  1. What are the behaviors, modes of failure, and damage to cripple walls and sill anchorages?
  2. What test datasets are available to calibrate analysis and damage models of existing and/or retrofitted cripple walls?
  3. What test datasets are available to calibrate analysis and damage models of existing and/or retrofitted sill anchorages?
  4. What is known from analytical studies regarding the vulnerability of cripple walls and the effectiveness of measures to retrofit the structure?
  5. How has deterioration due to age affected the strength characteristics of cripple walls and sill anchorages?
  6. Under what conditions does the retrofit of cripple walls notably increase damage to the occupied stories? What type of damage may occur?
- Behavior of and Damage to Existing Wood-Frame Houses
  1. What are the behaviors, modes of failure, and damage to existing wood-frame houses from past earthquakes?
  2. What surveys and/or studies have been performed to evaluate damage of existing wood-frame houses from past earthquakes?
  3. What are the important features of house configurations and construction that have been observed to affect earthquake damage?
  4. What correlations can be made between specific eras of houses and damage to houses?
- Analytical Models for Wood-Frame Structures
  1. What are the common model types to analyze the nonlinear response of wood-frame houses?
  2. What are the currently available software analysis programs that are capable of simulating the nonlinear and damage response of wood-frame houses?
  3. What studies have been done to calibrate and validate the reliability of nonlinear analysis models for wood-frame houses?
  4. What are the major gaps in knowledge and test data for analyzing the nonlinear response of wood-frame houses?
  5. How significant is soil–foundation–structure interaction in the nonlinear response analyses of wood-frame houses?
  6. How has or should unmodeled energy dissipation (e.g., viscous damping) be incorporated in the nonlinear dynamic response analysis of wood-frame houses?

7. How have or should modeling uncertainties be incorporated into demand parameters determined using nonlinear dynamic analysis?
8. What studies are available to incorporate material aging and deterioration in nonlinear analysis of wood-frame houses?

- Damage and Loss Assessment of Wood-Frame Houses
  1. What methods are available to develop damage and loss functions of wood-frame houses?
  2. What data are available to calibrate and validate component damage and loss (consequence) functions for wood-frame houses?
  3. What data or studies have been done to validate the overall damage (loss) functions for wood-frame houses?
  4. What are the implications of cripple wall damage and failure on observed losses?
  5. How should large deformations, up to complete failure, of the cripple wall be incorporated in loss analysis? (This relates to at what point the cripple wall damage and resulting house damage is beyond repair.)
  6. How should large deformations—up to complete failure—of the sill plate anchorage be incorporated in loss analysis? (This relates to at what point the anchorage damage and resulting house damage is beyond repair.)
  7. How significant is damage to acceleration sensitive components in the overall damage (loss) functions for wood-frame houses?
- Characterizing Ground Motions for Assessment of Wood-Frame Houses
  1. What ground-motion intensity parameters have or should be used to characterize ground motions (e.g., spectral acceleration, spectral shape, duration, near-fault pulses, etc.) for nonlinear analysis and loss assessment of wood-frame structures?
  2. Of currently available methods, what techniques are best suited to characterize and combine different ground-motion characteristics (e.g., spectral acceleration, spectral shape, duration, near-fault pulses, etc.) in nonlinear dynamic analysis?
  3. What is the range of expected ground motions in regions of California with significant populations of wood-frame houses?
  4. How significantly do site response characteristics affect the ground-motion shaking in the range that is expected to affect wood-frame houses? In addition, how should these site characteristics (mean and dispersion) be incorporated in the input ground motions?
- Loading Protocols for Testing of Wood-Frame House Components
  1. What are the common loading protocols for wood-frame component testing?

2. What is the sensitivity of wood-frame component behavior to various available loading protocols?
3. What are the major shortcomings of available loading protocols for assessing the behavior of cripple walls?
4. What is the suggested/recommended approach for developing loading protocols tailored for cripple wall testing?

- Communicating Risks and Incentivizing Risk Mitigation
    1. What data are available to quantify the cost of cripple wall and/or sill plate retrofit?
    2. What data, methods, and information are available to demonstrate the cost-benefit of cripple wall and/or sill plate retrofit?
    3. What concepts, stories (qualitative information), and data are available to quantify the benefits of cripple wall and/or sill plate retrofit?
    4. What are effective ways to communicate risks and retrofitting concepts and opportunities to California homeowners toward encouraging changes in behavior?
    5. What are some existing products and programs aimed at communicating about earthquake risks and retrofits to homeowners that this Project can learn from and potentially reference?
- Glossary
    1. What are the key terms and definitions used in the earthquake research and risk management community that can be used in this study to standardize and enhance communication of study methods, thereby creating a vernacular that is easily understood by a wide range of audiences?

3 Working Group 2: Identification and Development of Index Buildings

Working Group Leaders and Participants: Evan Reis, Gregory G. Deierlein, Henry Burton, Seb Ficcadenti, Thor Matteson, Bret Lizundia, and Tara Hutchinson

3.1 INTRODUCTION

Working Group 2 focused on identifying common variations and combinations of materials and construction characteristics of California single-family wood-frame dwellings. These were used to develop "Index Buildings" that formed the basis of the PEER–CEA Project testing and analytical modeling programs. The loss modeling component of the Project ultimately quantified the damage–seismic hazard relationships for each of the Index Buildings. Note: the terms "existing" and "unretrofitted" are used interchangeably in this report.

Based on discussions among the Project Team, a review of available documentation and research, and discussions with representatives from ATC-110, CEA, and the PEER–CEA Project Leadership and Program Review Panel, the Project Team identified the building variants to be considered in the development of the Index Buildings. The inclusion of a variant was based on three criteria:

- A significant representation among California homes (in excess of approximately 10% of housing stock, based on census or research data or expert opinion);
- The potential to have a significant impact on building earthquake damage (assuming approximately +/-5% of replacement cost, based on expert opinion to be verified through testing and analysis); and
- The amount of damage *reduction* resulting from the seismic retrofit of the cripple wall is dependent upon the presence of the variant.

Variants were divided into five categories.

- Primary, observable characteristics that broadly define the building (date of construction and number of stories):
- Secondary, observable characteristics (building weight, bolting, and cripple wall dimensions):

- Secondary characteristics that the typical underwriter or home inspector cannot generally observe, but which may vary by age (siding/sheathing combination, building shear capacity, and condition);
- Unobservable characteristics that are likely to have similar values regardless of age (building size and configuration, sheathing nailing, and sill bolt diameter and hole size); and
- Variants that were initially considered but determined to not meet the three qualifying criteria (plan irregularities, split levels, slabs on grade, chimneys, rotated foundations, soft stories, and roof sheathing).

Based on the selected variants identified, the Project Team developed an initial list of Index Buildings; see Table 3.1.

A raised foundation refers to the condition where the first floor is built on a wood stud cripple wall that sits on the concrete footing. Cripple wall heights of two, four, and six feet were initially considered. A stem wall foundation refers to the condition where the first-floor joists rest directly on the sill plate, which sets on the footing. This can also be referred to as a zero-height cripple wall.

Following the completion of the testing program conducted by Working Group 4 and analytical modeling conducted by Working Group 5, the initial list of building variants and Index Buildings was refined. Table 3.2 contains the final list of index buildings for which DFs were developed. The report from Working Group 2 explains in detail, the justification for including or excluding particular variants.

Table 3.1 Initial list of building variants considered in Project study.

All cases	Size and configuration		Generally 1200 sf rectangular footprint with 4:3 aspect ratios
	Nail spacing in siding/sheathing		Two nails per board
	Foundation bolt diameter		1/2 in. (if present)
	Bolt hole diameter		1/4 in. oversize (if present)
	Building shear capacity		As per ATC-110 recommendations as function of age
	Condition		Apply upper and lower bounds to achieve an average value of each Index Building
	Retrofit		Both unretrofitted and retrofitted conditions
Combinations	Pre–1945	Stories	1 or 2
		Sill bolting	Unbolted (wet-set sill)*
		Building weight	Heavy or light
		Cripple wall height / slope differential	Stem wall (zero-height), 2-, 4-, and 6-ft raised cripple walls, and differential heights being combinations of these
		Siding / sheathing combinations	Stucco/none, stucco/horizontal, stucco/diagonal, and horizontal/none
	1945–1955	Stories	1 or 2
		Sill bolting	Unbolted (wet sill), bolted (6 ft or better)
		Building weight	Heavy or light
		Cripple wall height / slope differential	Stem wall (zero-height), 2-, 4-, and 6-ft raised cripple walls, and differential heights being combinations of these
		Siding / sheathing combinations	Stucco/none, stucco/horizontal, stucco/diagonal, and horizontal/none
	1956–1970	Stories	1 or 2
		Sill bolting	Bolted (6 ft or better)
		Building weight	Heavy or light
		Cripple wall height / slope differential	Stem wall (zero-height), 2-, 4-, and 6-ft raised cripple walls, and differential heights being combinations of these
		Siding / sheathing combinations	Stucco/none, stucco/horizontal, stucco/diagonal, horizontal/none, and T1-11/none

* Unbolted (wet-set sill) condition occurs typically when contractor installs mudsill into the concrete footing when it is cast, using spikes or heavy nails in the sill to provide nominal anchorage to the footing.

Table 3.2 Final list of building variants considered in Project study.

All cases	Size and configuration		Generally 1200 sf rectangular footprint with 4:3 aspect ratios
	Nail spacing in siding/sheathing		Two nails per board
	Foundation bolt diameter		1/2 in. (if present)
	Bolt hole diameter		1/4 in. oversize (if present)
	Building shear capacity		As per ATC-110 recommendations as function of age
	Condition		A single best estimate assumption of condition factors
	Retrofit		Both unretrofitted and retrofitted conditions
Combinations	Pre–1945	Stories	1 or 2
		Sill bolting	Unbolted (wet-set sill)
		Building weight	Lath and plaster interior finish Exterior wood siding or stucco siding
		Cripple wall height / slope differential	Stem wall (zero-height), 2- and 6-ft raised cripple walls, with no slope differential
		Siding / Sheathing combinations	Stucco with no sheathing, Horizontal siding boards with diagonal cut in stud wall bracing
	1945–1955	Stories	1 or 2
		Sill bolting	Bolted (6 ft or better), with no differentiation in strength with wet-set sill condition
		Building weight	Average of lath and plaster and gypsum interior finish Exterior wood siding or stucco siding
		Cripple wall height / slope differential	Stem wall (zero-height), 2- and 6-ft raised cripple walls, with no slope differential
		Siding / Sheathing combinations	Stucco with no sheathing, Horizontal siding boards with diagonal stud wall bracing
	1956–1970	Stories	1 or 2
		Sill bolting	Bolted (6 ft or better), with no differentiation in strength with wet-set sill condition
		Building weight	Average of gypsum wall board interior finish Exterior wood siding or stucco siding
		Cripple wall height / slope differential	Stem wall (zero-height), 2- and 6-ft raised cripple walls, with no slope differential
		Siding / sheathing combinations	Stucco with no sheathing, horizontal siding boards with diagonal stud wall bracing, and T1-11 with diagonal let-in stud wall bracing

* Unbolted (wet-set sill) condition occurs typically when contractor installs the mudsill directly into the concrete footing immediately after it is cast. Often the sill will be spiked with nails to provide some anchorage.

3.2 TESTING PROGRAM

Based on a discussion with the leaders of the PEER–CEA Project Working Group 4 (Testing) and Working Group 5 (Analytical Modeling), the Project Leadership Panel (PLP) and the Project Team developed a list of high-priority testing cases, shown in Table 5.1, which were implemented by Working Group 4. To provide the type of information that could best be incorporated into the analytical modeling, the setup and form of the testing, including whether the individual components might be combined into system tests, was determined by Working Group 4 and coordinated with Working Group 5.

4 Working Group 3: Selection and Scaling of Ground-Motion Records and Development of Loading Protocols

Working Group Leader and Participants: Yousef Bozorgnia, Farzin Zareian, Bret Lizundia, Silvia Mazzoni, Jack Baker, Tara Hutchinson, Charlie Kircher, Joel Lanning, John van de Lindt, Norm Abrahamson, Nick Gregor, Linda Al Atik, David P. Welch, and Gregory G. Deierlein.

4.1 TASK 3.1: GROUND MOTIONS

The objective of Working Group 3, Task 3.1 was to provide suites of ground motions to be used by other working groups—especially Working Group 5—for simulation studies. The ground motions that are used in the numerical simulations are intended to represent seismic hazard at each building site. The seismic hazard is dependent on the location of the site relative to seismic sources, the characteristics of the seismic sources in the region, and the local soil conditions at the site. To achieve a proper representation of hazard and population density across the State of California, ten metropolitan sites were selected and a site-specific Probabilistic Seismic Hazard Analysis (PSHA) was performed at each of these sites for both a soft soil (V_{s30} = 270 m/sec) and a stiff soil (V_{s30} = 760 m/sec). The PSHA used the UCERF3 seismic-source model, which represents the latest seismic-source model adopted by the US Geological Survey [2013]. The PSHA was carried out for structural periods ranging from 0.01 to 10 sec. An example of the deaggregation data for the San Francisco site is shown in Figure 4.1.

At each site and soil class, the results from the PSHA—hazard curves, hazard deaggregation, and uniform-hazard spectra (UHS) —were extracted for a series of ten return periods, prescribed by Working Group 5, ranging from 15.5 to 2500 years. For each case (site, soil class, and return period), the UHS was used as the target spectrum for selection and modification of a suite of ground motions. Additionally, another set of target spectra based on "Conditional Spectra" (CS), which is more realistic than UHS, was developed. The CS are defined by a median (Conditional Mean Spectrum) and a period-dependent variance [Baker 2018]. At each site and soil class, a suite of 40 horizontal record pairs was selected and modified for each return period and target-spectrum type. Thus, for each ground-motion suite 40 record pairs were selected using the deaggregation of the hazard, resulting in 200 record pairs per target-spectrum type at each site. Figures 4.2 and 4.3 show a sample of the ground-motion suites corresponding to the UHS and CS target spectra, respectively, for the San Francisco site. Each figure shows the average spectrum of the selected and scaled ground-motion suite compared to the UHS at each return period, the variability as a function of period, a sample suite of 40 records at a particular return period, and a superposition of all records for all return periods, along with the average of each suite and the target UHS.

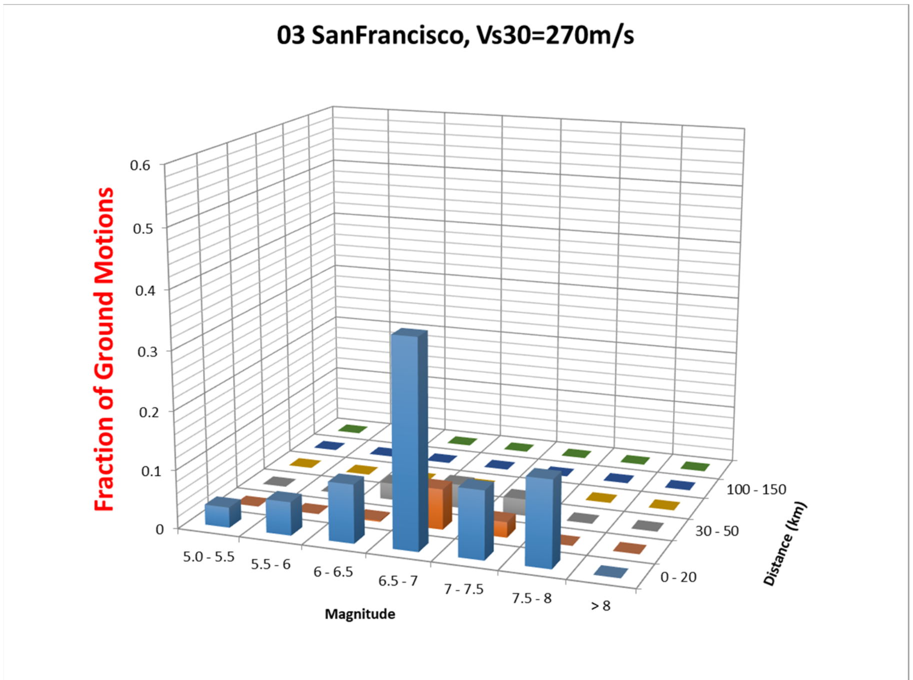


Figure 4.1 Deaggregation of site seismicity: San Francisco, V_{s30} = 270 m/sec.

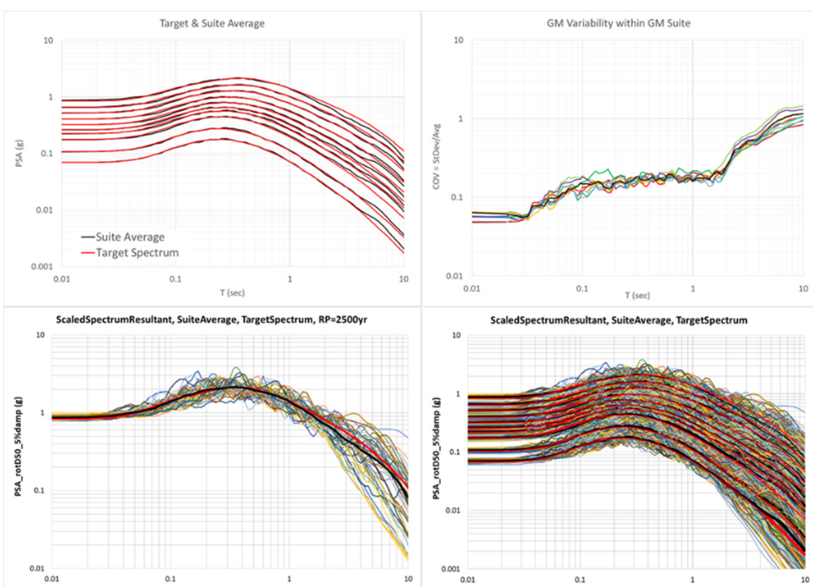


Figure 4.2 Sample ground-motion suites for UHS: San Francisco, V_{s30} = 270 m/sec.

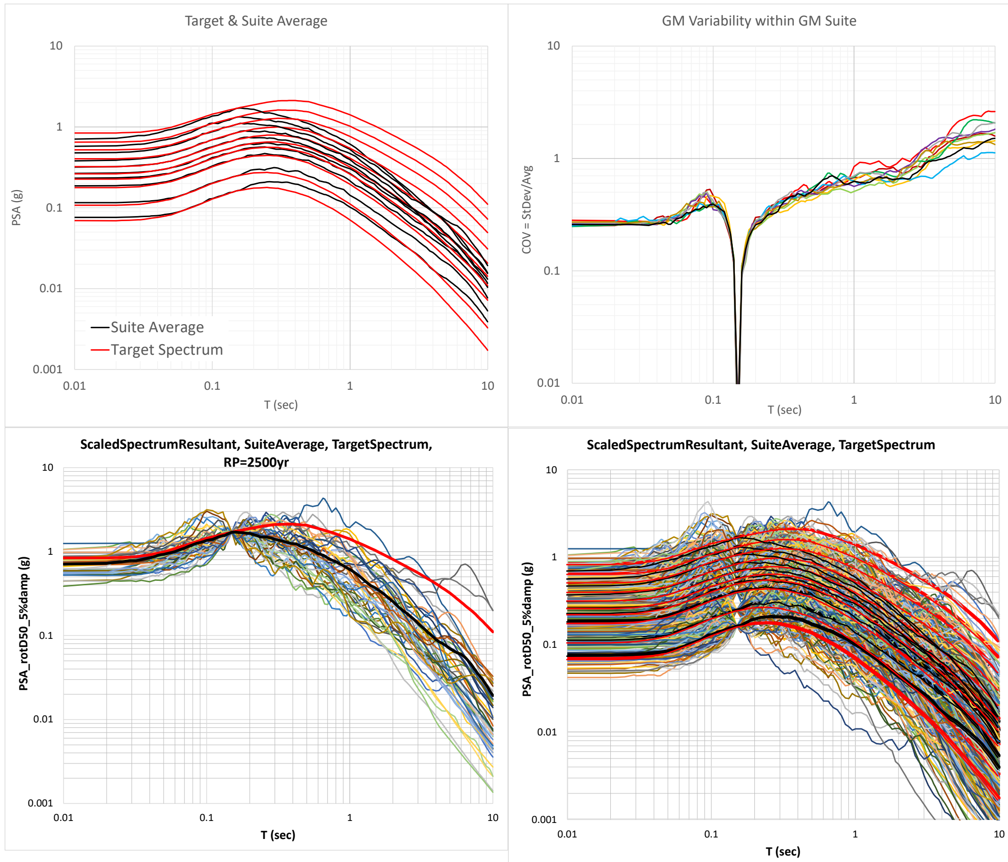


Figure 4.3 Sample ground-motion suites for CMS/CS: San Francisco, V_{s30} = 270 m/sec.

For the case of UHS as the target spectrum, the selected motions were modified (scaled) such that the average of the median spectrum (RotD50) [Boore 2010] of the ground-motion pairs followed the target spectrum closely within the period range of interest to the analysts. In communications with Working Group 5 researchers, for ground-motion (time histories or time series) selection and modification, a period range between 0.01 and 2.0 sec was selected for this specific application for the Project. The duration metrics and pulse characteristics of the records were also used in the final selection of ground motions. The damping ratio for the PSHA and ground-motion target spectra was set to 5%, as this is standard practice in engineering applications.

For the case where the CS is the target spectrum, the ground-motion suites were selected and scaled using a modified version of the CS ground-motion selection tool (CS-GMS tool) developed by Baker and Lee [2018]. This tool selects and scales a suite of ground motions to meet both the median and the user-defined variability. This variability is defined by the relationship developed by Baker and Jayaram [2008]. Computation of CS requires a structural period for the conditional model. In collaboration with Working Group 5 researchers, a conditioning period of 0.25 sec was selected as representative of the fundamental mode of vibration of the buildings of interest in this study.

A detailed description of the assumptions, as well as the methodology, findings and results are summarized in the Working Group 3.1 report [Mazzoni et al. 2020]. The report provides details of the selected sites, the seismic-source characterization model, and the ground-motion characterization model used in the PSHA, followed by the selection and modification of suites of ground motions described above. The selected ground motions are all part of the NGA-West2 database and be downloaded from https://ngawest2.berkeley.edu/ .

4.2 TASK 3.2: LOADING PROTOCOL

4.2.1 Objectives

The main objective of the PEER–CEA Project was to develop building DFs that reflect the benefit of the cripple wall and sill anchorage strategies to retrofit a structure as a function of shaking intensity. Therefore, a representative loading history was needed to conduct the laboratory component testing portion of the Project. The objective of Working Group 3, Task 3.2: *Loading Protocols* was to summarize the efforts that led to the development of such a loading protocol. The Project Team started by investigating if the currently available loading protocols in literature, namely, CUREE–Caltech [Krawinkler et al. 2002] and *FEMA 461* [2007], were adequate for the testing program envisioned for the PEER–CEA Project or was an update to those loading protocols needed.

4.2.2 Connection to Other Efforts in the PEER–CEA Project

The development of the loading protocol for cripple wall components is tied with three other efforts within the PEER–CEA Project. First, a set of representative wood-frame buildings on raised foundations in California were compiled, and the loading protocol developed was tailored for this target building set. Second, multiple sets of ground motions representing various sites and hazard levels were utilized for the development of the loading protocol aimed at capturing the possible displacement history of cripple wall components at various places in California. Third, the loading protocol developed was vetted by the experimental research group, and updates were made if the initial versions of the loading protocol needed any adjustments.

4.2.3 Background

With current advances in performance-based engineering of structural systems, the need for an in-depth understanding of the behavior of structural components subjected to seismic excitation is becoming even more relevant and critical. Such an understanding will help to fill the knowledge gaps in the modeling of structural components, leading to a more accurate estimate of seismic demands and better quantification of structural component damages in dynamic action. Such knowledge can reliably be acquired through laboratory testing in which test setups replicate real-world conditions to the extent possible. The loading history (i.e., loading protocol) used for laboratory testing is one of the critical components of any test as it controls the sequence and amplitude of cumulative damage in a structural component.

Development of a single loading protocol requires consideration of many constraints and conditions, including the type and material of the structural system/component that is being tested, amplitude and frequency of seismic excitation, and available resources for the experimental program. The main issue, however, is to account for cumulative damage effects through cyclic loading. Currently, available loading protocols can be categorized in the following three groups: (1) ATC-24 [1992], Clark et al. [1997], and Krawinkler et al. [1997] for testing structural components of steel structures; (2) Porter [1987], and Krawinkler et al. [2002] for testing structural components of wood-frame buildings; and (3) FEMA [2007] for testing nonstructural components. A detailed review of these loading protocols is offered in Krawinkler [2009].

The main advantages of the latest loading protocols for structural components of wood-frame buildings conducted during the CUREE–Caltech Project [Krawinkler et al. 2002] compared to the earlier ones are threefold: (1) a migration from anchoring loading history to yield deformation (a parameter that is determined based on subjective assumptions) to a deformation associated with performance objective; (2) introduction of separate loading protocols for different ground-motion hazard levels aimed at capturing the behavior of structural components in different seismic demand regimes; and (3) introduction of "trailing cycles" that follow the preceding larger "primary cycle" at each step, which is statistically justifiable and leads to a more realistic loading history compared to the SPD Protocol [Porter 1987] in which excessive number of cycles results in migration of dominant mode of failure from nail withdrawal to nail fatigue failure [Gatto and Uang 2002; SEAOSC 2001].

The CUREE–Caltech loading protocols were developed using the technology and information available in early 2000. In particular, these loading protocols were developed using old seismic hazard models (i.e., $Sa(T_1)$-type scaling), and old ground-motion record databases (i.e., the old PEER ground-motion database). Moreover, the CUREE–Caltech loading protocols are mostly geared towards shear-walls that possess low cyclic deterioration and periods between 0.2 and 1.0 sec; these assumptions are in contrast with what was observed in cripple walls tests by Chai et al. [2002] where the fundamental period of SDOFs representing cripple walls was between 0.05 and 0.2 sec, with a high rate of cyclic deterioration.

Thus, it was determined that an updated loading protocol based on new technology and information would be appropriate.

4.2.4 Scope and Plan of Study

The entire loading protocol development process is summarized here for completeness.

The target of the first stage of the two-stage process was to validate the analytical models developed by the Project Team for cripple walls and gain valuable knowledge about their dynamic behavior. The outcome of this first stage would then guide the Project Team in the next stage, where it followed an approach similar to what was used to generate the CUREE–Caltech and *FEMA-461* [2007] cyclic loading protocols. For the first stage, the Project Team suggested a few dynamic laboratory tests using a set of recorded ground motions that included records with peculiar characteristics, including near-field and soft-soil effects. The results of these tests alongside cyclic tests at University of California, Davis (US Davis) as part of the CUREE–Caltech Woodframe Project [Chai et al. 2002] would then be used to calibrate our analytical models of cripple walls at the component level as well as the system level. In particular, our validated multi-degrees-of-freedom (MDOF) models would be capable of addressing essential aspects of the dynamic response of wood-frame buildings on cripple walls, including the effect of uplift, torsion, and uneven distribution of seismic forces on stepped cripple walls.

With data available from the outcome of ATC 110 project, the first stage of the two-stage process was deemed unnecessary. The second state of this project involved developing a loading protocol for cripple wall components. The loading protocol team followed the method suggested by Krawinkler et al. [2002] by utilizing analytical models and ground-motion sets developed by

PEER–CEA Project researchers. The new loading protocol for testing cripple walls was developed considering the following:

- Consistent with numerous recent and past loading protocols, the loading protocols will be quasi-static (cyclic) for deformation-controlled components.
- Emphasis was placed on performance assessment on a spectrum of seismic hazard return periods. The loading protocol represents the average return periods of 72, 475, 1000, and 2475 years. The addition of two hazard levels (i.e., 72- and 1000-year average return period) to what was proposed in Krawinkler et al. [2002] is warranted given the high rate of cyclic deterioration observed in cripple wall tests at UC Davis Chai et al. [2002].
- Ground-motion selection and scaling to develop loading protocols was conducted using the state-of-the-art methods as described in by Working Group 3.1.
- Analytical models (i.e., MDOFs) using the recommended loading protocol to explicitly model the cripple wall component with expected characteristics were developed. The characteristics of the MDOF models were informed by results from ATC 110 project and are representative of light wood-frame structures on raised foundations located in California.
- The loading protocol is anchored to a displacement representing a target performance. In contrast with the CUREE–Caltech loading protocol, this anchor point is identified without a need to conduct a monotonic test. This approach provides the opportunity to utilize all testing resources to conduct cyclic tests.

4.2.5 Current Study

Task 3.2 provided the essential knowledge and data for the development of a quasi-static loading protocol for cyclic testing of cripple wall components of wood-frame structures. The recommended loading protocol for component testing was developed to formulate analytical models for cripple wall components. These analytical models were utilized for the performance-based assessment of wood-frame structures in the context of the PEER–CEA Project.

The recommended loading protocol was developed using nonlinear dynamic analysis of representative MDOF systems subjected to sets of single-component ground motions that varied in location and hazard level. Cumulative damage of the cripple wall components of the MDOF systems was investigated using rain flow cycle counting (RFCC) routines. The result is a testing protocol that captures the loading history a cripple wall may experience in various seismic regions in California.

4.2.6 Future Study

The proposed loading protocol for the quasi-static loading of cripple wall components was limited by the assumptions considered for its development. Additional loading histories can be developed to incorporate the issues listed in the following depending on the availability of data and testing equipment/setup.

- Using dynamic loading protocols [Retamales et al. 2011] rather than quasi-static, to demonstrate loading rate effects in the behavior of cripple wall components;
- Investigate the development of non-symmetric loading cycles;
- Investigate the applicability and possible updates to the suggested loading protocol for stepped cripple walls, and other related variation in cripple wall configuration and boundary conditions;
- Investigate the effect of ground-motion directionality and ground-motion sequences for the development of new loading protocols to ensure that analytical models of cripple wall behavior capture their behavior accurately; and
- Should a performance expectation standard arise from this effort and other cripple wall studies, developing prequalifying test loading protocols would be warranted.

4.2.7 Recommended Loading Protocol for Cripple Wall Component Testing

The suggested loading protocol presented herein is intended for cripple walls (as components of wood-frame structures) where deformation is the primary source of damage. This loading protocol is intended for quasi-static testing as the basis for the development of cripple wall component models to be used for the numerical simulation of structural system assessment in the context of performance-based earthquake engineering.

4.2.7.1 Presentation of the Recommended Loading Protocol for Cripple Wall Component Testing

The general form of the loading history for quasi-static loading of cripple walls are illustrated in the figures below. The horizontal axis of Figure 4.4 and Figure 4.5 shows the number of cycles (denoted as i), and the vertical axis shows the relative amplitude of each cycle (denoted as a_i). The loading history suggested herein utilizes a 0.01 drift ratio as its reference deformation, i.e., $\delta_y / h = 0.01$, where h is the height of the cripple wall component. Figure 4.6 shows the drift ratio of ordered excursions for each step of the loading history; each cycle consists of two steps (i.e., one forward and one backward displacement). The loading history for cripple walls with h = 2 ft, 4 ft, and 6 ft, is illustrated in Figure 4.7, Figure 4.8, and Figure 4.9, respectively. With reference to Figure 4.4 and Figure 4.5, the following sequence of cycles is to be executed:

- Seven cycles with a relative amplitude a_i of 0.05, i $\in \{1, 2, \cdots, 7\}$
- Seven cycles with a relative amplitude a_i of 0.15, i $\in \{8, 9, \cdots, 14\}$
- Seven cycles with a relative amplitude a_i of 0.20, i $\in \{15, 16, \cdots, 21\}$
- Four cycles with a relative amplitude a_i of 0.40, i $\in \{22, 23, 24, 25\}$
- Four cycles with a relative amplitude a_i of 0.60, i $\in \{26, 27, 28, 29\}$
- Three cycles with a relative amplitude a_i of 0.80, i $\in \{30, 31, 32\}$

- Three cycles with a relative amplitude a_i of 1.40, $i \in \{33, 34, 35\}$
- Three cycles with a relative amplitude a_i of 2.00, $i \in \{36, 37, 38\}$
- Two cycles with a relative amplitude a_i of 3.00, $i \in \{39, 40\}$
- Two cycles with a relative amplitude a_i of 4.00, $i \in \{41, 42\}$
- Two cycles with a relative amplitude a_i of 5.00, $i \in \{43, 44\}$
- Increasing steps of the same pattern; two cycles with an increase in relative amplitude a_i of 1.00, $i \in \{45, 46, \cdots\}$

Other items for the loading protocol include:

- It is not necessary to conduct cycles with less than 1/32-in. amplitude;
- Each experiment should continue until the load applied in each cycle decreases to 20% of the maximum load recorded during the entire experiment;
- For increments in relative amplitude a_i beyond 2.00, performing one cycle instead of two cycles is allowed;
- Material testing, fabrication of test specimens, experimental plan, and instrumentation should be based on existing standards, and best practices applicable to the project;
- Specimens should be investigated for possible damage, the formation of cracks, general behavior, and other standard monitoring practices at the end of each cycle; and
- Reported test results should include the following: (i) specimen geometry; (ii) specimen construction and mobilization details; (iii) specimen boundary conditions and instrumentation detail; (iv) material testing; (v) deformation control history (input and output); (vi) instrumentation read-out for all exercised cycles; and (vii) observations made during each experiment at the end of each cycle.

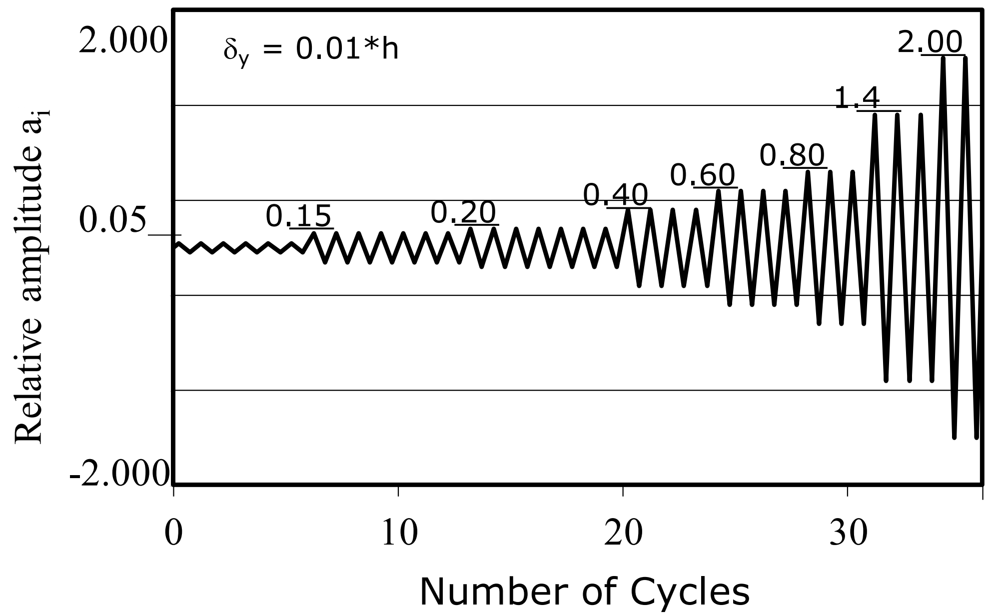


Figure 4.4 General form of the suggested quasi-static loading protocol for cripple wall component testing.

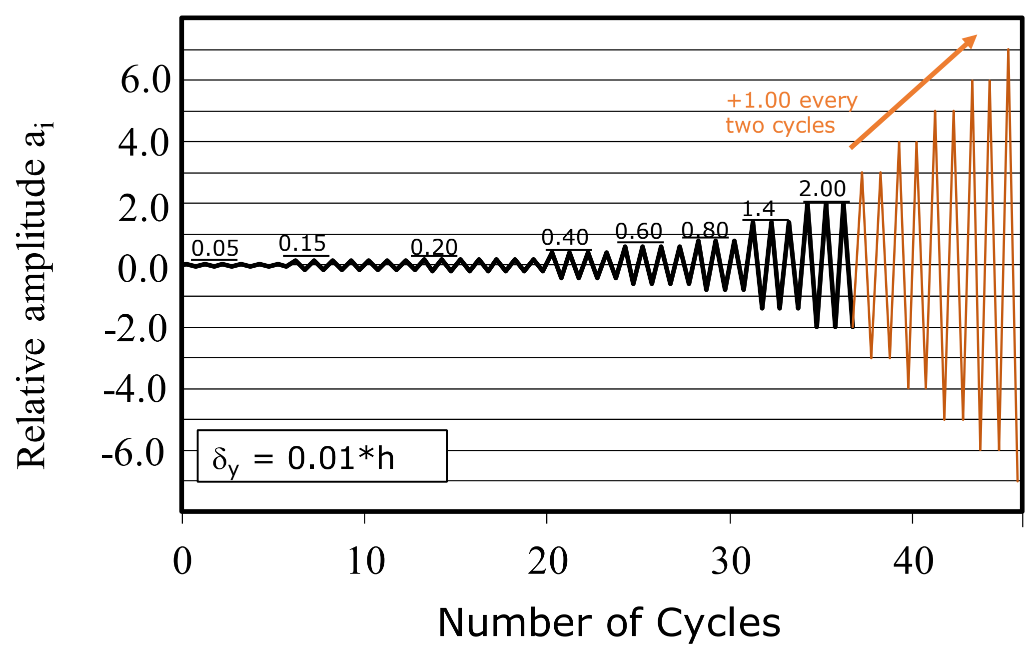


Figure 4.5 General form of the suggested quasi-static loading protocol for cripple wall component testing.

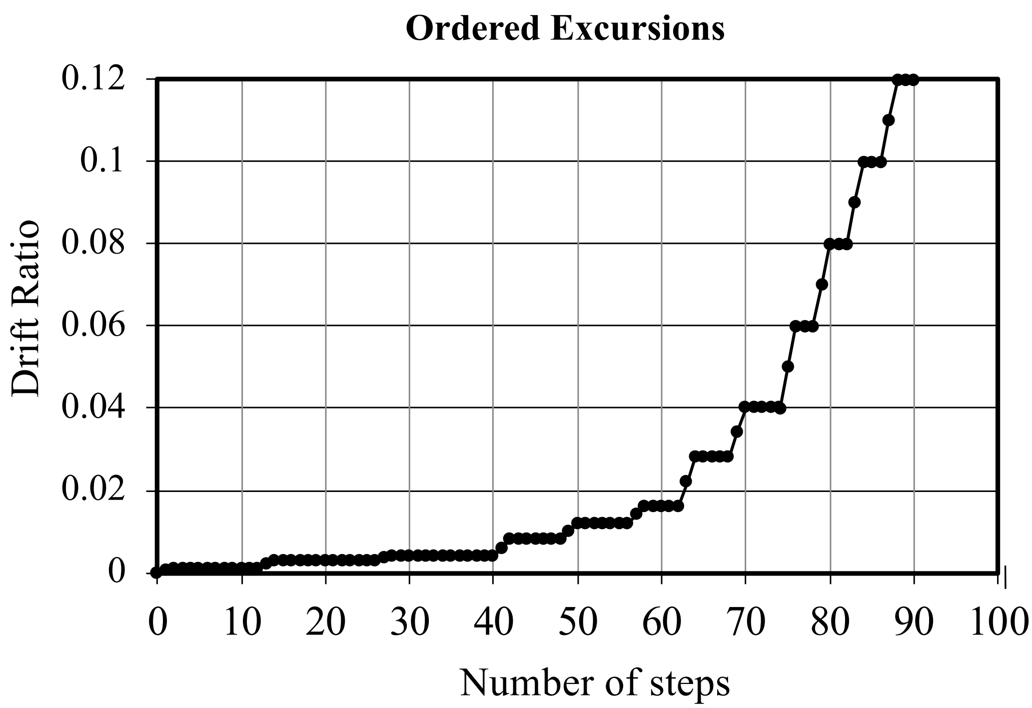


Figure 4.6 **Drift ratio vs. number of steps for the suggested quasi-static loading protocol for cripple wall component testing.**

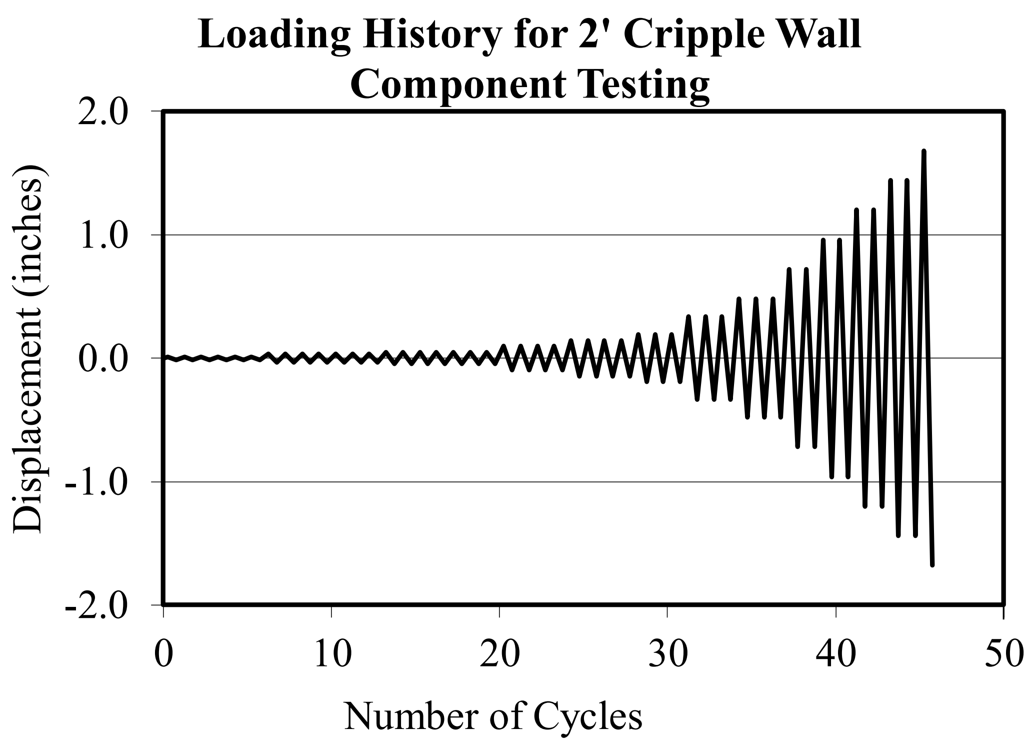


Figure 4.7 **Suggested quasi-static loading protocol for the 2-ft-tall cripple wall component testing.**

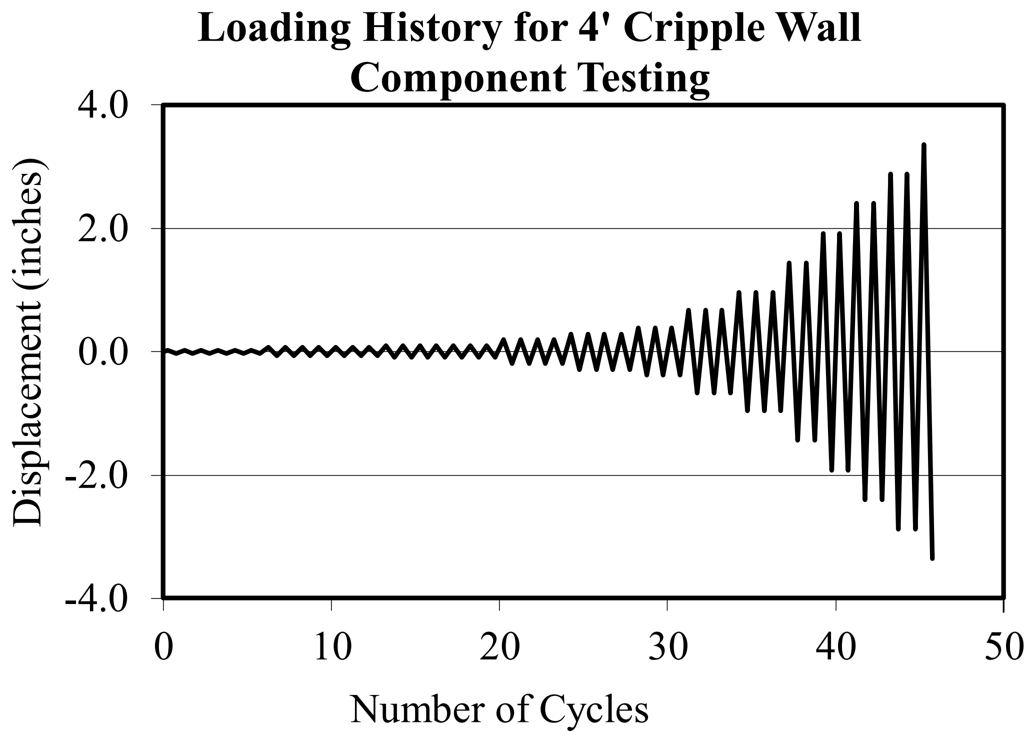


Figure 4.8 Suggested quasi-static loading protocol for the 4-ft-tall cripple wall component testing.

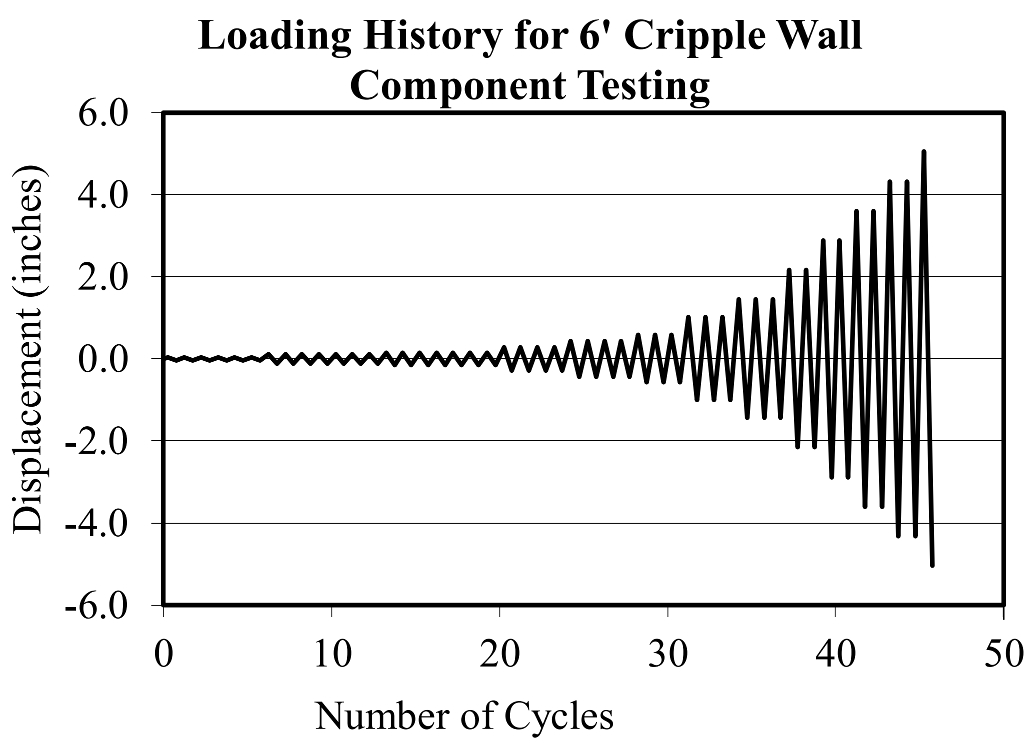


Figure 4.9 Suggested quasi-static loading protocol for 6-ft-tall cripple wall component testing.

5 Working Group 4a: Small-Component Testing

Working Group Leader and Participants: Tara Hutchinson, Brandon Schiller, Kelly Cobeen, Bret Lizundia, Gilberto Mosqueda, Chia-Ming Uang, Seb Ficcadenti, Thor Matteson, and John van de Lindt

5.1 INTRODUCTION

Working Group 4 focused on small-component testing, conducting an experimental investigation to study the seismic performance of retrofitted and existing cripple walls with sill anchorage. Field observations of past earthquakes have shown that inadequate lateral bracing of cripple walls and inadequate sill bolting are the primary reasons for failures of residential homes even in the event of moderate earthquakes. While methods to retrofit weak cripple walls and improve sill anchorage have been developed, the economic benefits have not been quantified. In addition, little knowledge is available to quantify, particularly with supporting experimental observations, the performance of existing houses. Note that the terms "existing" and "unretrofitted" are used interchangeably in this report.

Conducted in parallel with a large component test program conducted at UC Berkeley (Working Group 4b), Working Group 4a conducted multiple phases of small-component tests at UC San Diego, focusing predominantly on the most vulnerable pre-1960s construction detailing. Small component tests entail testing 12-ft-long cripple walls fastened to a foundation with no superstructure elements above the cripple wall. Parameters examined were cripple wall height, finish materials, gravity load, boundary conditions, and anchorage condition anchorage. The small-component test program at UC San Diego was divided into four phases, with six–eight specimens tested per phase. Subdividing the program into multiple phases allowed analysis of one phase of test results to aid in the design of subsequent phases. In addition, this resulted in a manageable number of full-scale specimens within the laboratory space. Each of the test phases were complimentary to other phases for cross comparison upon completion of subsequent phases to allow for meaningful comparisons amongst specimens within a particular phase. The first and third tasks focused on wet specimens, those specimens designed with stucco exterior finishes (i.e., Phase 1, Phase 3, and a portion of Phase 4). The second task focused solely on dry specimens, those specimens finished with wood-absent stucco (i.e., Phase 2 and a portion of Phase 4). The final (fourth) task conducted a cross comparison of all 28 specimens, with both wet and dry finishes. The full test matrix is shown in Table 5.1. Results from these experiments were intended to provide an experimental basis to support numerical modeling used to develop loss models, which are intended to quantify the reduction of loss achieved by applying state-of-practice methods identified in *FEMA P-1100, Vulnerability-Base Seismic Assessment and Retrofit of One- and Two-Family Dwellings*.

Table 5.1 Testing matrix developed and implement by PEER–CEA Project Working Group 4.

Phase	Specimen	Test #	Existing/ retrofit	Era	CW Height (ft)	Anchorage WS = wet set, S = spacing	Exterior finish†	BC***
1	A-1	4	E	Pre-1945	2	S(64 in.)	S+HSh	A, a
1	A-2	3	E	Pre-1945	2	S(64 in.)	S+HSh	B, a
1	A-3	6	E	Pre-1945	2	S(64 in.)	S+HSh	C, a
1	A-4	1	E	Pre-1945	2	S(64 in.)	S+HSh	B, b
1	A-5	5	R	Pre-1945	2	S(32 in.)	S+HSh	B, a
1	A-6	2	E	Pre-1945	2	WS	S+HSh	B, b
2	A-7	7	E	1945-1955	2	S(64 in.)	HS	B, c
2	A-8	8	R	1945-1955	2	S(32 in.)	HS	B, c
2	A-9	11	E	1945-1955	2	S(64 in.)	HS+DSh	B, c
2	A-10	12	R	1945-1955	2	S(32 in.)	HS+DSh	B, c
2	A-11	9	E	1956-1970	2	S(64 in.)	T	B, c
2	A-12	10	R	1956-1970	2	S(32 in.)	T	B, c
2	A-13	13	E	1945-1955	6	S(64 in.)	HS	B, c
2	A-14	14	R	1945-1955	6	S(32 in.)	HS	B, c
3	A-15	20	E	Pre-1945	2	S(64 in.)	S+DSh	B, c
3	A-16	21	R	Pre-1945	2	S(32 in.)	S+DSh	B, c
3	A-17	18	E	Pre-1945	2	S(64 in.)	S	B, d
3	A-18	22	R	Pre-1945	2	S(32 in.)	S	B, d
3	A-19	19	R	Pre-1945	2	S(32 in.)	S+HSh	B, c
3	A-20	15	E	Pre-1945	2	S(64 in.)	S+HSh	B, d
3	A-21	17	E	Pre-1945	2	WS	S+HSh	B, c
3	A-22	16	E	Pre-1945	2	S(64 in.)	S	B, c
4	A-23	23	E	1956-1970	6	S(64 in.)	T	B, c
4	A-24	24	R	1956-1970	6	S(32 in.)	T	B, c
4	A-25	27	E	Pre-1945	6	S(64 in.)	S	B, c
4	A-26	28	R	Pre-1945	6	S(32 in.)	S	B, c
4	A-27*	26	E	Pre-1945	2	S(64 in.)	S+HSh	B, c
4	A-28**	25	E	1945-1955	2	S(64 in.)	HS+DSh	B, c

* All tests except A-27 used a cyclic loading pattern. Test A-27 employed a monotonic loading pattern.
** All tests except A-28 used a vertical load of 450plf. Test A-28 used a vertical load of 150plf.
*** BC = Boundary Conditions.; see Working Group 4 reports for details.
†S = stucco only, HS = horizontal siding, HS+DSh = horizontal siding over diagonal sheathing, S+HSh = stucco over horizontal sheathing, S+DSh = stucco over diagonal sheathing, T = T1-11 siding.

5.2 CRIPPLE WALL SMALL-COMPONENT TEST PROGRAM: WET SPECIMENS I

This effort, which addressed the first phase of testing, consisted of six specimens. Phase 1 including quasi-static reversed cyclic lateral load testing of six cripple walls, 12 ft in length and 2-ft in height; see Figure 5.1. All specimens in this phase were finished on their exterior with stucco over horizontal sheathing (referred to as a "wet" finish), typical of pre-1945 dwellings in California. Parameters addressed in this first phase included: boundary conditions on the top, bottom, and corners of the walls, attachment of the sill to the foundation, and the retrofitted condition. Details of the test specimens, testing protocol, instrumentation, and measured and physical observations are summarized in the report from this Working Group. In addition, this task established the rationale and scope of subsequent small-component test phases. Observations regarding the effects of the various conditions considered are summarized below.

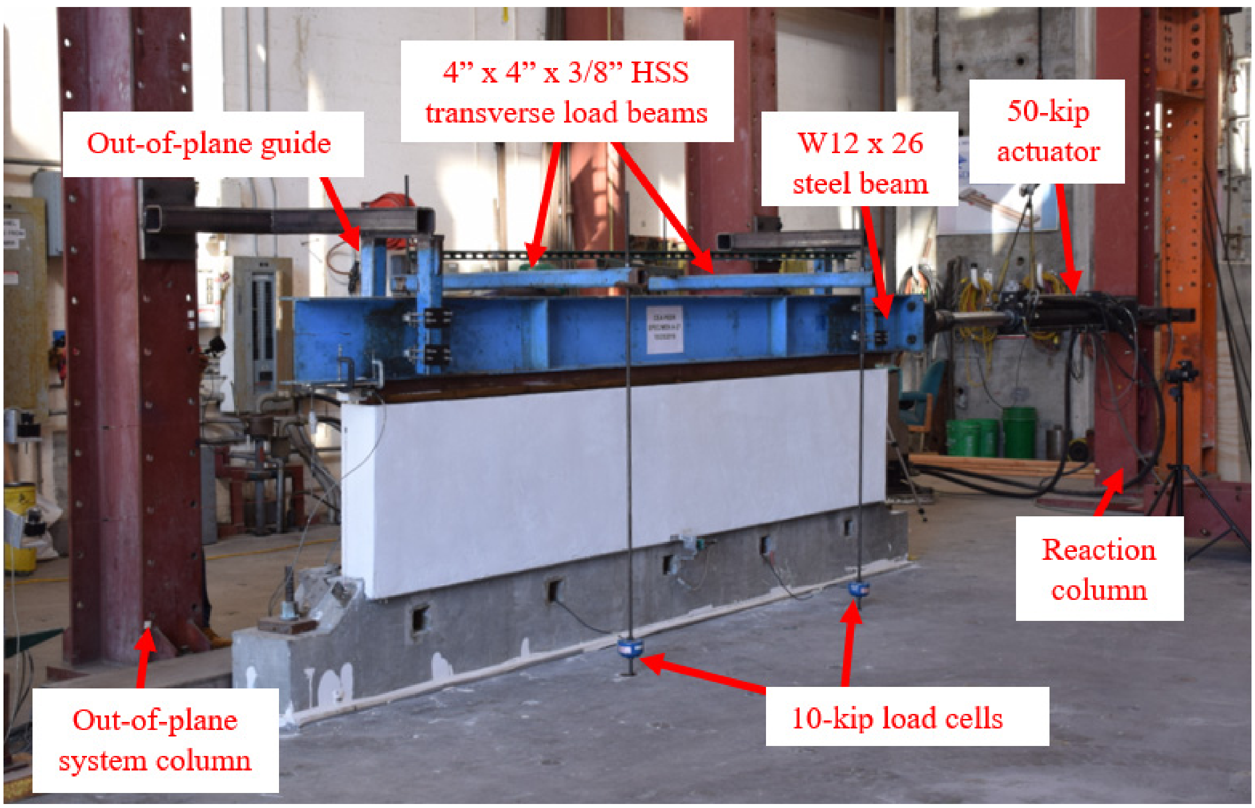


Figure 5.1 **Isometric view of the test setup for 2-ft-tall cripple walls [Schiller et al. 2020(c)].**

5.2.1 Impact of Boundary Conditions

Various boundary conditions were considered within the small-component testing program. For ease in discussion, they were delineated as either top or bottom boundary conditions; see Schiller et al. [2020(a)]. To facilitate understanding of these conditions, a brief discussion of each is provided in this section.

Top Boundary Conditions

The top boundary conditions implemented were intended to examine the effects of enhanced top plates and built-up end framing (corners), including C-shaped walls (i.e., built-up corners with a wall return), and, in particular, to compare with the response of walls tested by Chai et al. [2002], which did not contain enhanced top plates or corners. In addition, at the top of the cripple walls variations in furring nail arrangements connecting the stucco to the framing were implemented with the purpose of simulating stucco continuity into the floor above. The top boundary conditions used were denoted as top boundary condition A, B, and C.

Top boundary condition-A was similar to the specimen details used in the CUREE-Caltech Woodframe Project at UC Davis by Chai et al. [2002]. These specimens were cripple walls framed with two 2 × 4 top plates connected with 16d common nails at 16 in. on center. Studs were 16 in. on center and connected to the lower 2 × 4 top plate and 2 × 6 sill plate with 2–16d common nails per stud, top and bottom. The furring nails were spaced at 6 in. on center around the edges and through the field. Details of top boundary condition A are shown in Figure 5.3.

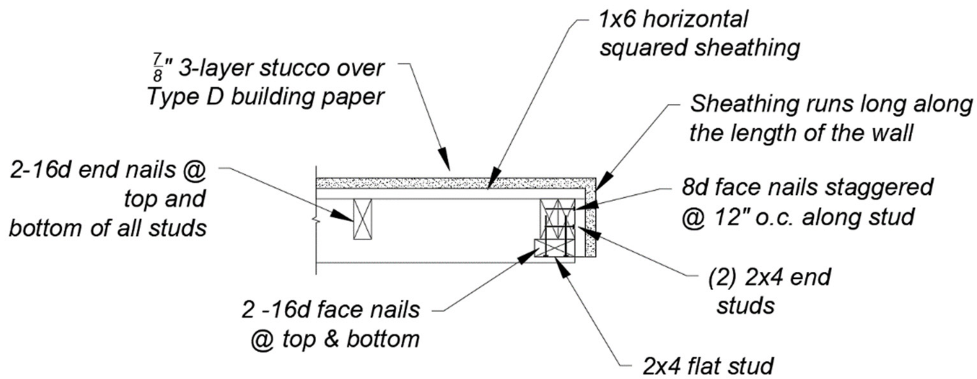
For top boundary condition B, the cripple wall was constructed with built-up corners (ends) as well as an additional top plate. The built-up wall ends are typical to those seen in California houses at re-entrant corners (corners where return walls would be present). These simulated corners contained two 2 × 4 studs instead of a single 2 × 4 stud and an additional 2 × 4 flat stud abutted against the interior side of the framing; see Figure 5.2(a) and (c) for details of the corner construction as well as the stucco and sheathing arrangement. The additional top plate was provided to allow for a denser furring nail arrangement at the top of the cripple wall, which was intended to simulate the increased stiffness provided by the continuity of the stucco running from the cripple wall into the upper story of the house. The prescribed nailing pattern was two rows of #11 × 1-1/2 in. (0.121-in. diameter) furring nails connected to the uppermost top plate and the middle top plate at 3 in. on center. Along the edges and the studs, #11 × 1-1/2 in. furring nails were attached at 6 in. on center, and along the sill plate; the same furring nails were attached with three nails per stud bay or 5-1/3 in. on center. Top boundary condition B was selected as the baseline top boundary condition for the entire testing program.

Top boundary condition C contained the same detailing at the top of the wall and stucco and sheathing attachment details as top boundary condition B; however, this boundary condition incorporated a return wall at each end, effectively resulting in a C-shaped wall specimen. The purpose of the return wall was to determine if the detailed end conditions adopted in top boundary condition B sufficiently contributed to the response of the wall considering the presence of a return wall. The return walls were 2 ft long on both ends of the specimen. The first stud bay was 16 in. on center, and the second was 8 in. on center. The return wall corners were framed with two 2 × 4 studs; the return wall was tied down with two anchor bolts, one within each stud bay; see Figure 5.2(b) for details of the corner construction. The following conclusions were made regarding the effect of the top boundary conditions.

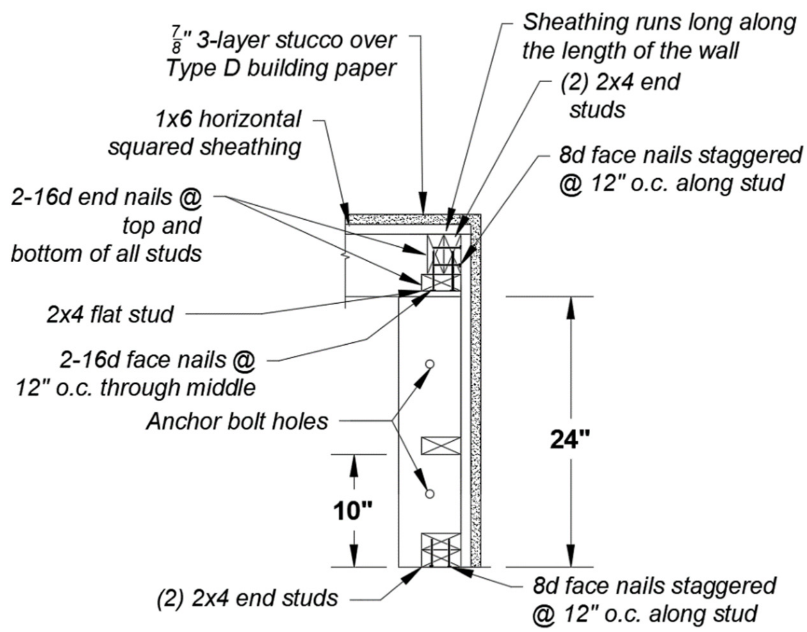
- The implementation of an extended corner return (denoted as top boundary condition C) had an insignificant effect on the lateral strength of the cripple wall when compared with top boundary condition B.;
- The cripple wall without built-up corners, i.e., top boundary condition A, had 60% of the strength and 50% of the secant stiffness of the cripple wall compared to top boundary condition B. The secant stiffness is defined as the secant stiffness associated with the relative drift at 80% pre-strength. In addition, the cumulative energy dissipated by the cripple walls with built-up ends was twice that of the cripple wall without;
- The implementation of a denser furring nail arrangement at the top of the cripple walls as well as a built-up corner (top boundary conditions B and C) provided a dramatic increase in the stiffness and lateral strength of a cripple wall compared to top boundary condition A, which featured a furring nail arrangement absent in this emulation of the continuity of the stucco finish to the upper floor; and
- The denser furring nail arrangement at the top of the cripple wall provided a more accurate representation of the continuity of the stucco running from the cripple wall up through the superstructure of the house.

Bottom Boundary Conditions

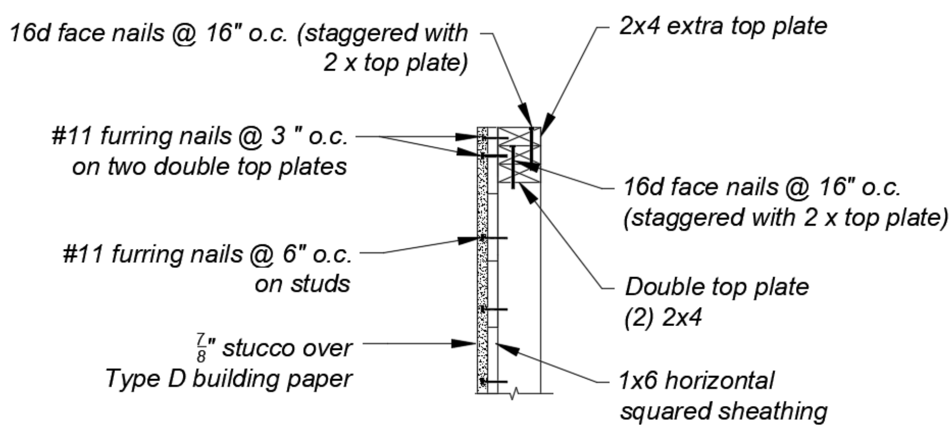
For the boundary conditions on the bottom of the cripple wall, four different conditions were used; denoted as bottom boundary condition "a", "b", "c", and "d". Bottom boundary condition "a" pertains to instances when there is a combined exterior finish. This boundary condition orientated the cripple wall so that the siding or stucco overhung the face of the footing while the sheathing (if present) remained bearing on the foundation; see Figure 5.4(a). Bottom boundary condition "b" also pertained to combined exterior finishes. This configuration had both the siding or stucco and sheathing material bearing on the top of the foundation, where the bottom nailing of the sheathing was attached to the middle of the sill plate. A photograph of this boundary condition is shown in Figure 5.4(b). Bottom boundary condition "c" orientated the cripple wall so that all exterior finishes were an outboard of foundation. This is the same whether there was a combined finish material or only the presence of a siding or stucco finish. Regardless of a single or combined finish material, the first layer of material attached to the framing was flush with the face of the footing; see Figure 5(c). Bottom boundary condition "c" emulates the condition where there is no bond between the stucco and foundation by having the stucco terminate at the top of the foundation. Finally, bottom boundary condition "d" pertained to cripple walls with stucco only or stucco over sheathing exterior finish. This boundary condition was similar to bottom boundary condition "c" except that the stucco was extended down the face of the footing. It is believed that this also a very common condition in California houses where the foundation stem wall is extended above grade. In this scenario, home builders would often extend the stucco to meet the soil grade, rather than terminate it at the base of the sill plate. As seen in Figure 5.4(d), the tail extension of the stucco was extended 8 in. down the face of the foundation.



(a)

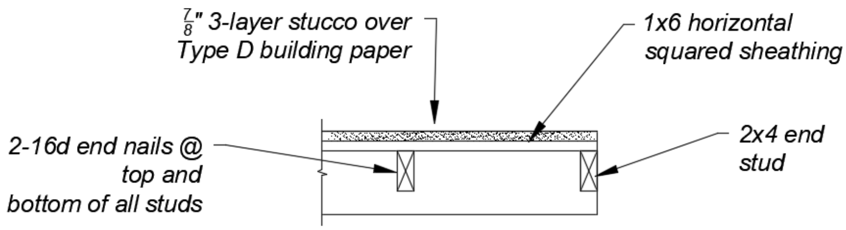


(b)

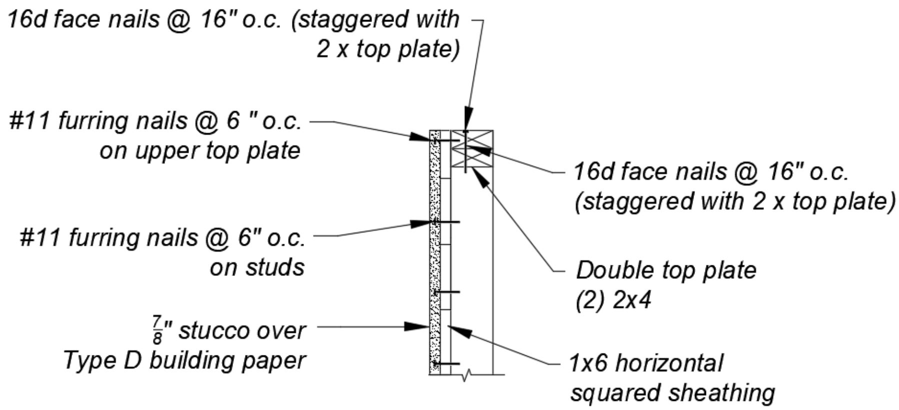


(c)

Figure 5.2 **Corner and top of wall details for stucco over horizontal sheathing: (a) plan view detail of top boundary condition B; (b) plan view detail of top boundary condition C; and (c) top of wall detail for top boundary condition B and C [Schiller et al. 2020(a)].**



(a)

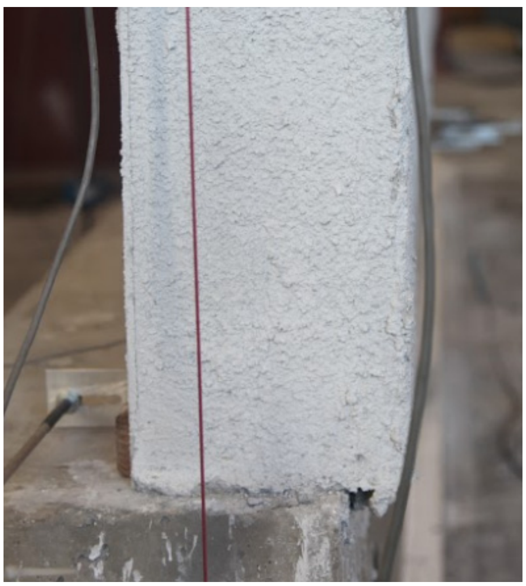


(b)

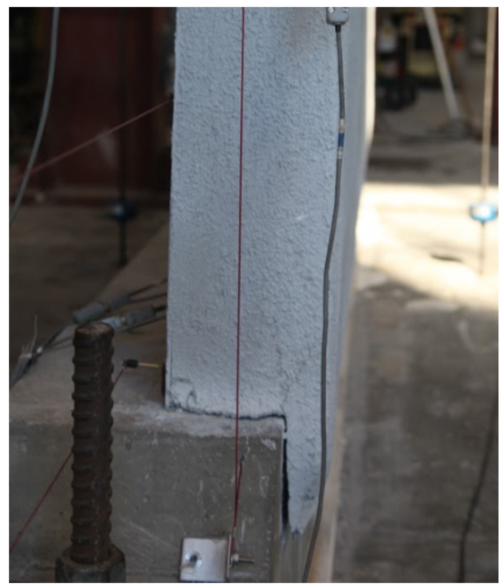
Figure 5.3 Corner and top of wall details for stucco over horizontal sheathing for top boundary condition A: (a) plan view detail; and (b) top of wall detail [Schiller et al. 2020(a)].

(a)

(b)



(c)



(d)

Figure 5.4 **Photographs of the various bottom boundary conditions: (a) bottom of end of wall for bottom boundary condition “a”; (b) bottom of end of wall for bottom boundary condition “b”; (c) bottom of end of wall for bottom boundary condition “c”; and (d) bottom of end of wall for bottom boundary condition “d”.**

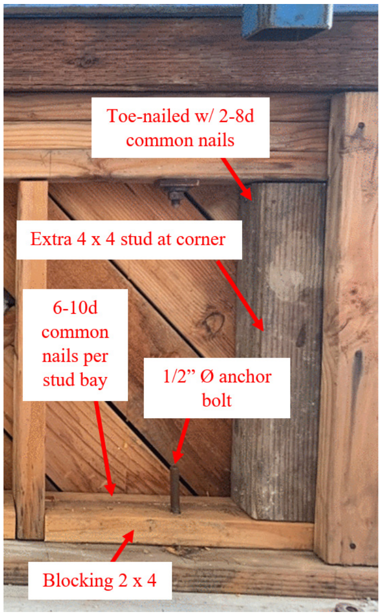
5.2.2 Damage Characteristics

The most significant observations of damage during the testing of the various small-component test specimens are itemized within this section.

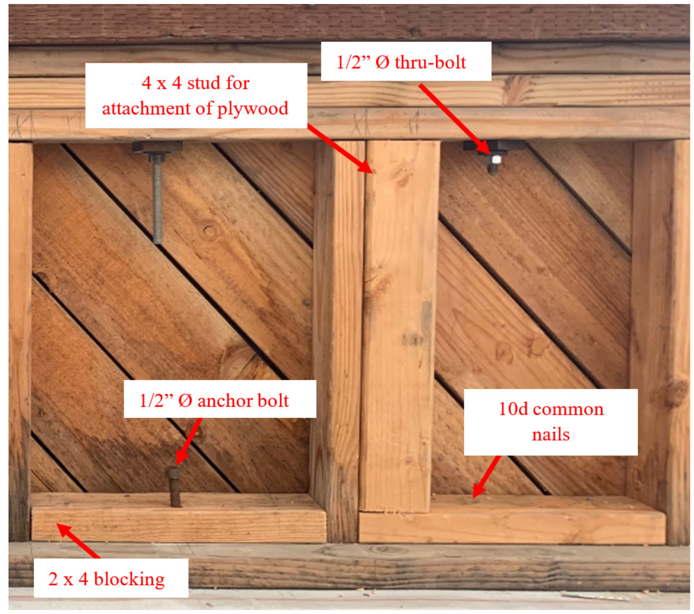
- Cracking of the stucco was minimal on the face of the cripple walls not detailed with finish around the ends (corners); denoted as top boundary condition A. When corners were present on the cripple walls, significant cracking propagated vertically and diagonally at the corners, even at low drift amplitudes

(0.2% to 1.4% drift amplitude). In addition, when corners were detailed with an additional finish (top boundary conditions B and C), vertical cracks also appeared on the exterior stucco face at the same low drift amplitudes. The extent of cracking and crack widths increased as the imposed drift increased. At large drift amplitudes, crushing and spalling of the stucco was observed, in particular at the interface with the concrete foundation, for cripple walls with corner conditions;

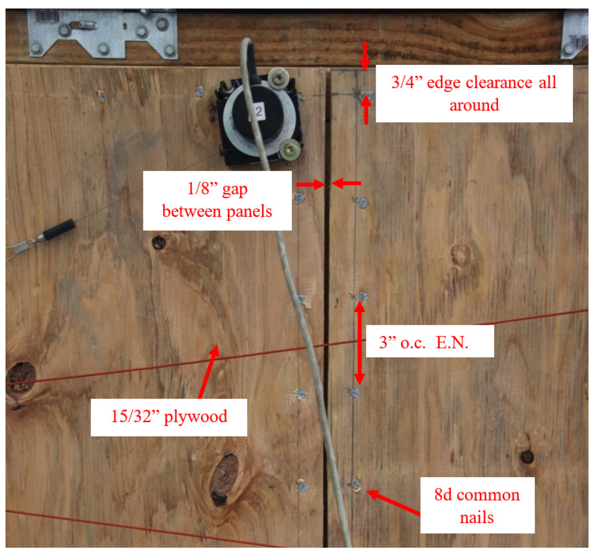
- After the strength was attained, the lateral resistance contribution from the stucco was greatly reduced due to loss of its connection to the sheathing and framing members (i.e., detachment of the furring nails). Upon continued lateral drift, however, the horizontal sheathing boards began to provide increased lateral resistance for those conditions when the sheathing boards were bearing on the footing. At very large drift amplitudes, the gaps between all sheathing boards (except for the sheathing board attached to the top plates) closed, and the sheathing boards started to bear on each other, which resisted the lateral displacement of the cripple wall, causing a significant retention of the lateral strength of the cripple wall up to large drift amplitudes;
- The stucco finish provided the majority of the stiffness and lateral strength of the cripple walls in all unretrofitted cases. Following attainment of the wall's lateral capacity, the strength of the cripple wall decreased mostly due to the detachment of the stucco from the furring nails but also from the detachment of the furring nails from the sheathing and framing members. As drift amplitudes increased, the stucco finish was pushed out laterally at the base of the cripple wall from the sheathing and framing members as the furring nails detached. At larger drift amplitudes, the stucco finish only retained its connection to the top plate, providing very little lateral strength to the wall specimen; and
- Failure of the retrofitted, cripple wall was primarily attributed to sheathing nail head pull through and/or nail withdrawal along the edges of the plywood panels, especially along the top plate and sides. At the bottom of the plywood panels, nails withdrew from the framing as added blocking split at large displacements. Some tearing of the nails through the plywood panels (edge tear-out) was observed at the corners. Photographs of the retrofit details can be seen in Figure 5.5.



(a)



(b)



(c)

Figure 5.5 Photographs of the common retrofit application details: (a) interior corner retrofit detail; (b) interior retrofit detail; and (c) plywood attachment detail.

5.3 CRIPPLE WALL SMALL-COMPONENT TEST PROGRAM: DRY SPECIMENS

This effort addresses the second phase of testing, which consisted of eight specimens, as well as half of the fourth phase of testing, which consisted of six specimens; three will be discussed herein. Although conducted in different phases, their results are combined here to co-locate observations regarding the behavior of all dry finished specimens. Experiments involved imposition of combined vertical loading and quasi-static reversed cyclic lateral load onto eleven cripple walls of 12-ft in length and 2-ft or 6-ft in height. All specimens were constructed with the same boundary conditions on the top, bottom, and corners of the walls. Parameters addressed include: dry exterior finishes (shiplap horizontal lumber siding, shiplap horizontal lumber siding over diagonal lumber sheathing, and T1-11 wood structural panels), cripple wall height, vertical load, and the retrofitted condition. Details of the test specimens, testing protocol, instrumentation, and measured and physical observations are summarized. In what follows, conclusions specific to the parameters varied within this task, i.e., all dry specimens, are summarized.

5.3.1 Impact of Exterior Finish

Horizontal Siding

- Horizontal siding was the weakest exterior finish material tested. Comparing the existing 2-ft-tall cripple wall with the horizontal siding finish had 30% of the lateral load capacity of the T1-11 plywood finished cripple wall and around 10% of the capacity of the horizontal siding over diagonal sheathing finished cripple wall. For the 6-ft-tall specimens, the horizontal siding finished specimen had 25% of the capacity of the T1-11 finished specimen;
- Horizontal siding also exhibited the largest drift capacity of any exterior finish, with little to no lateral strength degradation. The 2-ft-tall specimen dropped 25% of load from drift (4% drift ratio); strength was achieved by 12% drift. The 6-ft-tall specimen continued to gain strength until 12% drift; and
- The hysteretic response of cripple walls finished with horizontal siding was nominally symmetric. Photographs of the 2-ft-tall existing specimen with horizontal siding exterior finish are provided in Figure 5.6.

(a)

(b)

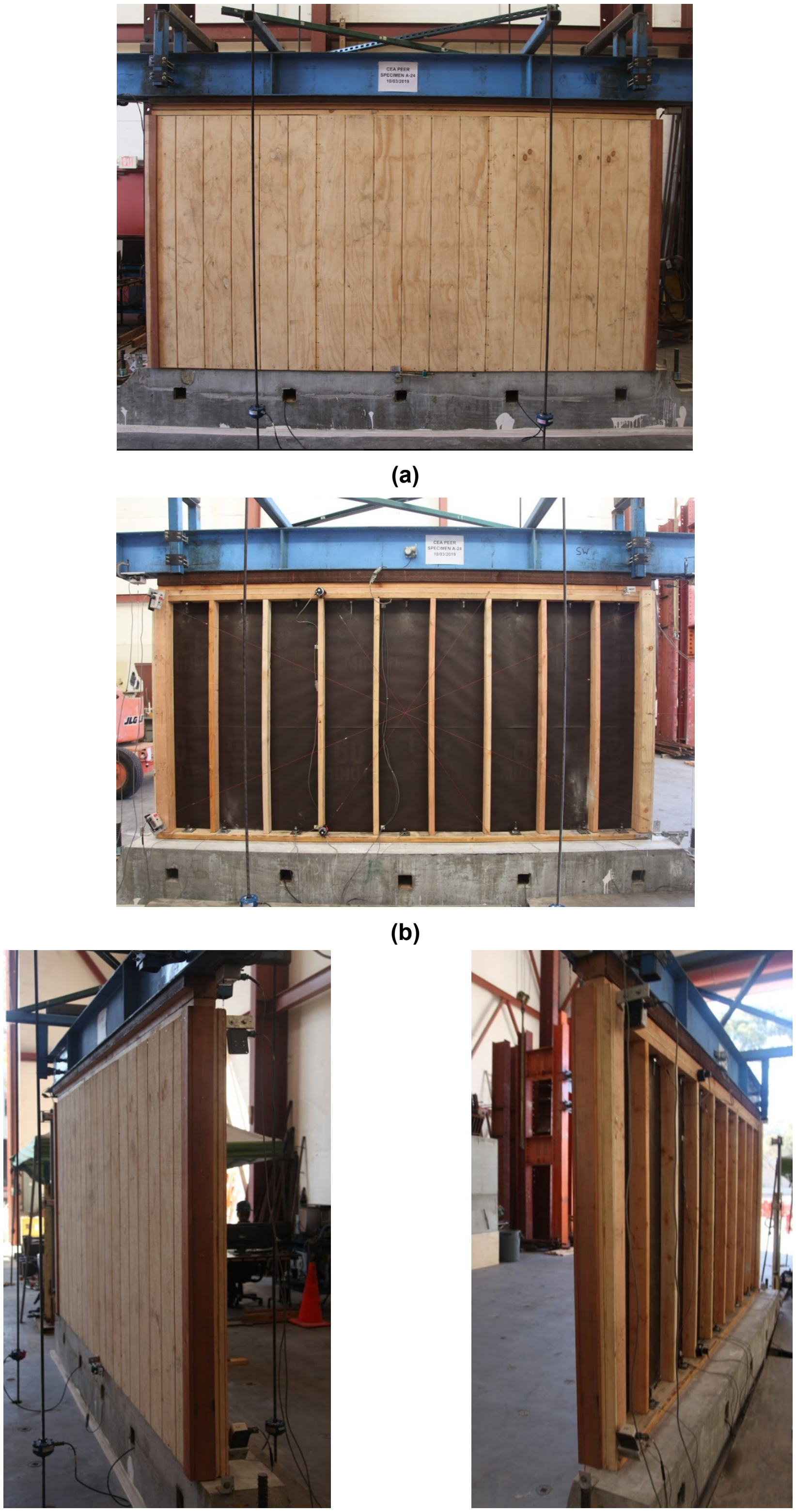
(c)

(d)

Figure 5.6 Specimen A-7: pre-test photographs for the existing 2-ft-tall cripple wall with horizontal siding exterior finish: (a) exterior elevation; (b) interior elevation; (c) north exterior corner; and (d) north interior corner [Schiller et al. 2020(b)].

T1-11 Wood Structural Panels

- T1-11 plywood panels were the second strongest exterior finish material tested; however, for a wood structural panel, the measured lateral strengths were relatively low. This can be attributed to the wide spacing of the fasteners attaching the T1-11 panels to the framing as well as the very thin T1-11 panels where the fasteners were located;
- T1-11 finished cripple walls attained their strengths at relatively low drift amplitudes, i.e., these specimens were stiffer compared with other dry finished specimens. On average between push and pull for the 2-ft-tall specimen, 80% of the strength was achieved by 1.7% relative drift ratio, while strength was achieved at 4.7% relative drift ratio; however, this finish material did not do well in maintaining its strength post-peak. This was the result of fasteners losing attachment via tearing and pulling through along the edges of the panels; and
- The hysteretic response of cripple walls finished with T1-11 plywood panels were nominally symmetric. Photographs of the existing 6-ft-tall specimen with T1-11 wood structural panel exterior finish along with the hysteretic response of the specimen are provided in Figure 5.7 and Figure 5.8, respectively.



(a)

(b)

(c) (d)

Figure 5.7 Specimen A-24: pre-test photograph for the retrofitted 6-ft-tall cripple wall with T1-11 wood structural panel exterior finish: (a) exterior elevation; (b) interior elevation; (c) north exterior corner; and (d) north interior corner [Schiller et al. 2020(b)].

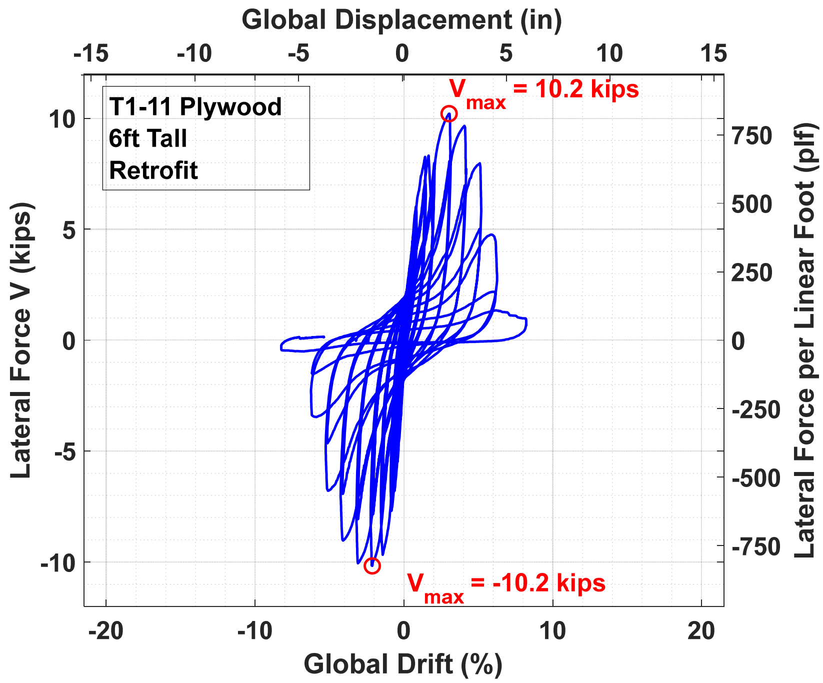


Figure 5.8 **Lateral force versus global lateral drift and displacement hysteresis of Specimen A-24 [Schiller et al. 2020(b)].**

Horizontal Siding over Diagonal Sheathing

- Horizontal siding over diagonal sheathing exhibited the strongest and stiffest existing exterior finish tested by a wide margin. Notably, the strengths of these specimens were more than 500% greater in push and 900% greater in the pull direction compared with like cripple wall specimens finished with horizontal siding, and over a 100% increase in push loading and almost a 200% increase in pull loading from the cripple wall with T1-11 wood structural panels. The significant lateral strength of diagonal sheathing finished walls was enough to cause fractures in all anchor bolts;
- The horizontal siding over diagonal sheathing cripple wall had a secant stiffness associated with relative drift at 80% pre-lateral strength, which was 130% larger than the horizontal siding cripple wall and 80% larger than the T1-11 cripple wall;
- The relative drift at strength was, on average between push and pull loading, the same as the 2-ft-tall horizontal siding cripple wall. Lateral strength was achieved at 4.5% relative drift ratio (7% global ratio) in the push direction and 3.6% relative drift ratio (10% global ratio) in the pull direction. The differences in global and relative drift are due to the large amounts of sliding of the sill plate over the foundation. The global drift ratio considers the entire imposed displacement divided by the height of the cripple wall while the relative drift ratio considers the displacement of only the cripple wall divided by the height of the cripple wall (ignoring displacement of the sill plate relative to the foundation). The drift capacity of diagonal sheathing was high, but the post-peak behavior could not be fully investigated due to fractures in the anchor bolts

or splitting of the sill plate, which caused premature termination of the test prior to significant cripple wall strength loss; and

- Different from other dry finished specimens, the response of horizontal siding over diagonal sheathing finished specimens was initially symmetric and then became highly asymmetric due to the orientation of the sheathing boards. In the push direction, the gaps between boards opened, while in the pull direction, the gaps between the boards closed. Once the gaps had closed, the sheathing boards bore on each other and acted in unison, similar to a wood structural panel. The peak strength in this direction was only 4.5% less than that of the retrofitted cripple wall with horizontal siding, demonstrating that the capacity of diagonal sheathing is similar capacity to plywood.

5.3.2 Impact of Cripple Wall Height

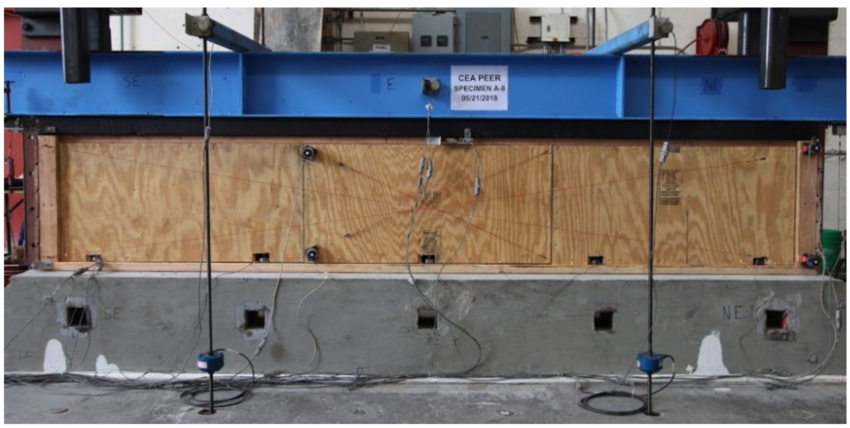
- Taller cripple walls experience more uplift and more flexure than their smaller counterparts, which are dominated by a shear response. The horizontal siding did not have the capacity to initiate any uplift of the cripple wall meaning that the lateral strength was too low for there to be any uplift of the specimen; however, it did demonstrate that taller cripple walls are more flexible by achieving peak strength at 11–12% relative drift ratio, which is 190% more than the next closest cripple wall tested in the program;
- The strength was lower for taller 6-ft-tall cripple walls when compared to the shorter 2-ft-tall specimens. For existing cripple walls with horizontal siding exterior finishes, a large amount of the capacity came from the framing due to the low strength of the finish. The framing gains capacity through the resistance to overturning of the studs carrying the vertical load as well as the withdrawal strength of the nails connecting the framing members. Since the length of the studs were decreased by four feet, the moment due to the imposed load was reduced for the 2-ft-tall specimen. This led to the capacity of the 2-ft-tall cripple wall being almost 80% greater than that of the 6-ft-tall cripple wall. For the 6-ft-tall cripple wall finished with T1-11, the capacity was around 50% less than that of the 2-ft-tall specimen; and
- For the retrofitted cripple walls with horizontal siding exterior finish, the drift at strength was reduced for the 6-ft-tall walls. The increased imposed displacement for 6-ft-tall walls (three times as high) caused the plywood to detach at a lower drift amplitude compared to the 2-ft-tall cripple wall. The same was the case for the T1-11 cripple walls, both the existing and retrofitted walls.

5.3.3 Response of Specimens Implemented with the *FEMA P-1100* Retrofit

- The *FEMA P-1100* retrofit increased the strength and stiffness for all tested cripple walls. The lowest increase in strength occurred with the horizontal siding over diagonal sheathing cripple walls, where a 110% increase in strength in the push direction and over a 50% increase in strength in the pull direction were observed. There would have been an even larger increase if the retrofitted cripple wall had reached its full capacity before the anchor bolts fractured, which is evident by the limited amount of damage to the actual cripple wall at the end of the test compared with other retrofitted cripple walls. The largest increase in strength occurred with the 6-ft-tall cripple wall with horizontal siding, which accounted for 17-times increase in lateral strength. For the 2-ft-tall counter parts, there was more than a 9.5-times increase. Photographs of the retrofitted 2-ft-tall cripple wall with horizontal siding exterior finish are shown in Figure 5.9. An overlay of the response for the existing and retrofitted specimens is given in Figure 5.10. For T1-11 specimens, the strength increase was nearly 100% for the 2-ft-tall specimens, and 125% for the 6-ft-tall specimens; and

- Overall, loss of lateral load capacity occurred when the plywood panel detached from the framing. This occurred by either the nails tearing through the edges of the panels, the nails pulling through the panels, the nails pulling out of the framing, or the nails pulling the blocking off the sill plate. Each retrofitted cripple wall experienced all these phenomena, except the T1-11 cripple wall, which did not include any blocking.

(a)



(b)

(c)

(d)

Figure 5.9 **Specimen A-8: pre-test photographs for the retrofitted 2-ft-tall cripple wall with horizontal siding exterior finish: (a) exterior elevation; (b) interior elevation; (c) north exterior corner; and (d) north interior corner [Schiller et al. 2020(b)].**

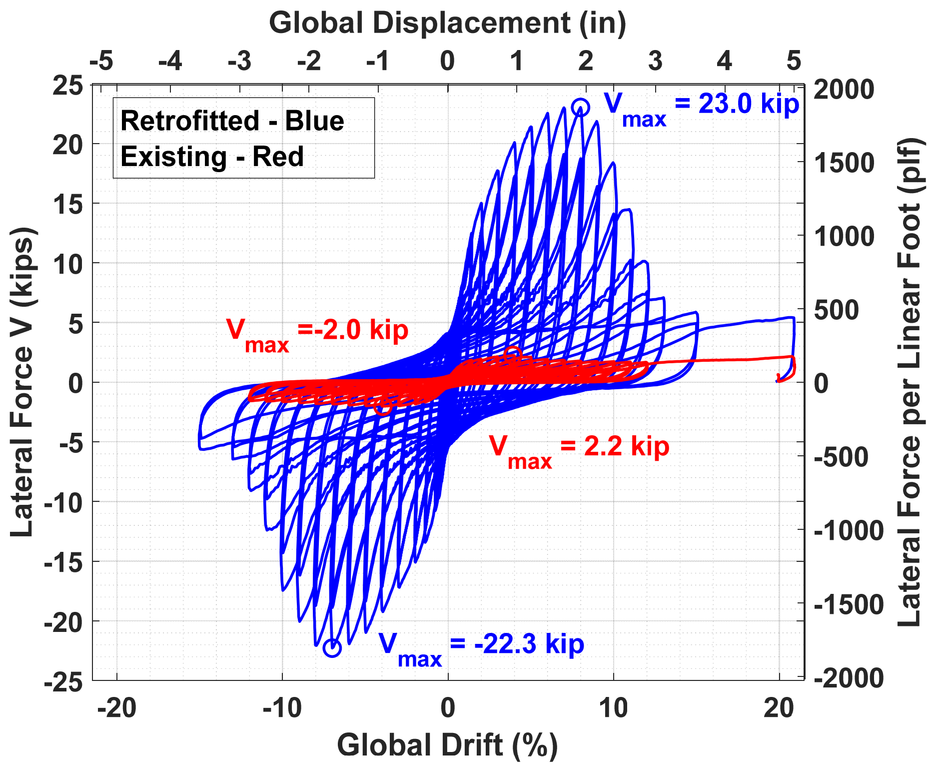


Figure 5.10 **Specimens A-7 and A-8: comparison of global drift versus lateral load hysteretic response for retrofitted and existing 2-ft-tall cripple walls with horizontal siding [Schiller et al. 2020(b)].**

5.3.4 Impact of Vertical Load

- Two vertical loads were implemented on 2-ft-tall existing cripple walls finished with horizontal siding over diagonal sheathing, namely, heavy (450 plf, emulating a two-story house) and light (150 plf, emulating a one-story house). Comparing these specimens, one notes a 50% increase in strength with the presence of the heavier vertical load condition;
- The secant stiffness associated with relative drift at 80% pre-lateral strength remained nearly unchanged; and
- The uplift at the ends of the specimen increased by 350% with the implementation of the light vertical load.

While the heavy vertical load cripple wall ended the test due to fractured anchor bolts, the light vertical load cripple wall test completion was associated with cross-grain splitting across the entire sill plate. This change in failure mode is due to the reduced vertical load decreasing the uplift resistance. Figure 5.11 shows images of the light vertically loaded cripple cross-grain cracking of the sill plate.

(a) (b)

Figure 5.11 Photographs of the sill plate failures for the cripple wall with light axial load (Specimen A-28): (a) interior view; and (b) interior view from end of cripple wall.

5.4 CRIPPLE WALL SMALL-COMPONENT TEST PROGRAM: WET SPECIMENS II

This effort addressed the third phase of testing, which consisted of eight specimens, as well as half of the fourth phase of testing, which consisted of six specimens; three will be discussed here. Although conducted in different phases, their results were combined to co-locate observations regarding the behavior of a second phase of wet finished specimens. Note that the first phase of wet specimens was largely focused on understanding the implications of various boundary conditions on the behavior of individual cripple wall specimens. Similarly, experiments summarized in this task involved imposition of combined vertical loading and quasi-static reversed cyclic lateral load onto ten 12-ft-long and 2-ft- to 6-ft-high cripple walls. One cripple wall was tested with a monotonic loading protocol. All specimens were constructed with the same boundary conditions on the top and corners of the walls while also being subjected to the same heavy vertical load condition. Parameters addressed include: wet exterior finishes (stucco over framing, stucco over horizontal lumber sheathing, and stucco over diagonal lumber sheathing), cripple wall height, loading protocol, anchorage condition, boundary condition at the bottom of the walls, and the retrofitted condition. Conclusions specific to the parameters varied within specimens are summarized below.

5.4.1 General Observations

- The hysteresis of all specimens was generally stable with no abrupt brittle failure. As anticipated, the strength and stiffness of the unretrofitted (existing) specimens was much lower than the specimen when retrofitted; and
- For all existing specimens, loss of strength occurred when the stucco detached from the furring nails at the sill plate and bottom of the studs. The only

exception to this was for the case where the stucco over diagonal sheathing specimens lost strength due to a cross-grain crack in the sill plate.

5.4.2 Impact of Exterior Finish

Stucco over Framing

- Stucco over framing was the weakest of the wet exterior finishes tested. Comparing the 2-ft-tall existing specimens, the stucco over framing finished cripple wall had around 55% of the lateral load capacity of the stucco over diagonal sheathing finished cripple wall, and around 95% of the capacity of the stucco over horizontal sheathing finished cripple wall;
- Stucco over framing had the lowest drift capacity and lowest drift at strength of any exterior finish. The average global drift ratio at strength for the, 2-ft-tall specimens was 2.3% and 1.3% relative drift ratio. For the existing 6-ft-tall specimen, these values were 1.4% and 1.1%, respectively. The 2-ft-tall existing specimens reached 40% residual strength by 4.6% relative drift, on average, while the, 6-ft-tall existing specimen reached 40% residual strength by 5.0% relative drift;
- Stucco over framing was the stiffest of any wet exterior finish tested. The secant stiffness—defined as the secant stiffness associated with the relative drift at 80% pre-strength, for the 2-ft-tall existing specimens—was 150% greater than that with stucco over horizontal sheathing and over 10% greater than that with stucco over diagonal sheathing; and
- The response was nearly symmetric for all stucco over framing finished specimens.

Stucco over Horizontal Sheathing

- Stucco over horizontal sheathing was the second strongest exterior finish tested; note, the measured lateral strengths were close to the stucco over framing finish. For the 2-ft-tall existing cripple walls with the same boundary and anchorage conditions, the stucco over horizontal sheathing only provided 3% increase in lateral strength compared with the stucco over framing specimen;
- Stucco over horizontal sheathing had the largest drift capacity of any of the finish materials tested, while the drift at strength was nearly equal to that of the stucco over diagonal sheathing finish. On average, for the 2-ft-tall existing specimens, the global drift ratio at strength was 2.8% (2.5% relative drift ratio). On average, the 2-ft-tall existing specimens reached 40% residual strength by 8.7% relative drift between both directions of loading;
- Stucco over horizontal sheathing was the most flexible exterior finish tested. For the 2-ft-tall existing specimens, the initial stiffness was 55% of the stucco over diagonal sheathing finish and 40% of the initial stiffness of the stucco over framing finish; and
- The response was nearly symmetric for all stucco over horizontal sheathing finished specimens.

Stucco over Diagonal Sheathing

- Stucco over diagonal sheathing was the strongest exterior finish tested. The average strength in both directions of loading was 75% greater than stucco over horizontal sheathing finish and 85% greater than the stucco over framing finish;
- The global drift ratio at strength was the largest of any of the finishes, 4% global drift in the push direction and 5% global drift in the pull direction; however, the relative drift ratio at strength was the nearly the same as the stucco over horizontal sheathing finish, with an average relative drift ratio at strength of 2.3% for the stucco over diagonal sheathing finish and 2.4% for the stucco over horizontal sheathing specimen;
- The stucco over diagonal sheathing finish was the only finish to fail due to either cross-grain cracking of the sill plate and/or fracturing of the anchor bolts. Thus, the response was close to symmetric for the 2-ft-tall existing specimen when it would be expected that the strength in the pull loading direction would have been greater than the strength in the push loading direction because of the orientation of the diagonal sheathing boards. When the specimen was pulled on, the gaps between the sheathing boards closed, the sheathing boards bore on each other, and then acted in unison, which would be a similar behavior for a wood structural panel; and
- Stucco over diagonal sheathing cripple walls experienced the most uplift of any of the finishes and the only 2-ft-tall existing specimen to show appreciable uplift. The largest uplift occurred at the north end during push loading due to the orientation of the sheathing boards.

5.4.3 Impact of Cripple Wall Height

- Taller cripple walls experience more uplift and more flexure than their smaller counterparts, which are dominated by a shear response. The existing stucco over framing specimens did not have the capacity to initiate any uplift, while the peak uplift for the 6-ft-tall specimen was between 0.1 and 0.2 in. In addition, there was a 60% reduction in initial stiffness for the 6-ft-tall existing specimen compared with the 2-ft-tall existing specimens;
- The strength was almost 20% larger for the taller stucco over framing finished specimen. This can be attributed to the increased number of furring nails fastening the stucco to the framing. For the retrofitted specimens, the strength capacity was same for the shorter and taller specimens; and
- For the retrofitted cripple walls with stucco over framing exterior finish, the drift capacity was reduced for the 6-ft-tall walls. The increased imposed displacement for 6-ft-tall walls (three times as high), which caused the plywood to detach a lower drift amplitude than the 2-ft-tall cripple wall. For the existing specimens, however, the drift capacity was similar regardless of height, while strength was attained at slightly lower drift amplitudes for the taller specimens (1.1% relative drift ratio versus 1.3% relative drift ratio).

5.4.4 Response of Specimens Implemented with the *FEMA P-1100* Retrofit

- The *FEMA P-1100* retrofit increased the strength, stiffness, and energy dissipation for all tested cripple walls. In addition, the retrofit increased the drift capacity for all cripple walls, except for those walls finished with stucco over diagonal sheathing, which provided almost no change;
- The lowest increase in strength occurred with the stucco over diagonal sheathing cripple walls, where an average strength capacity increase of 115% was observed. It is difficult to determine what the increase in strength would have been if the stucco over diagonal sheathing finished specimens had not lost capacity to anchor bolt fractures and cross-grain cracks in the sill plate. The largest increase in strength occurred with stucco over horizontal sheathing specimens, accounting for a 260% increase in lateral strength. Photographs and the hysteretic response of the retrofitted 2-ft-tall specimen with stucco over horizontal sheathing are provided in Figure 5.12 and Figure 5.13, respectively. For the stucco over framing specimens, the strength increase was 225% for the 2-ft-tall specimens and 180% for the 6-ft-tall specimens;
- The drift capacity increased the most with the stucco over framing finished specimens. For the 2-ft-tall cripple walls, the drift at strength increased from 1.3% relative drift ratio to 4.8% relative drift ratio, and the relative drift ratio at 40% residual strength increased from 4.6% to 9.3%. For the 6-ft-tall specimens, the relative drift ratio at strength increased from 1.1% to 2.9%, and the relative drift ratio at 40% residual strength increased from 5.0% to 7.1%. The stucco over horizontal sheathing specimens also experienced a dramatic increase in drift capacity, with the relative drift at strength increasing from 2.4% to 5.3%, and the relative drift ratio at 40% residual strength increased from 7.6% to 9.6%. The drift capacity for the stucco over diagonal sheathing cripple walls remained unchanged; and
- The increase cumulative energy dissipation was fairly consistent for the stucco over framing and stucco over horizontal sheathing specimens. For stucco over horizontal sheathing finishes, there was an 8-times increase in cumulative energy dissipated by the end of the test. The cumulative energy dissipated by the end of the test for the 2-ft-tall stucco over framing specimens increased around 7 times and increased around 8.5 times for the 6-ft-tall specimens.

(a)

(b)

(c)

(d)

Figure 5.12 Specimen A-19: pre-test photographs of the retrofitted 2-ft-tall cripple wall with stucco over horizontal sheathing exterior finish and bottom boundary condition c: (a) exterior elevation; (b) interior elevation; (c) south interior corner; and (d) south exterior corner [Schiller et al. 2020c].

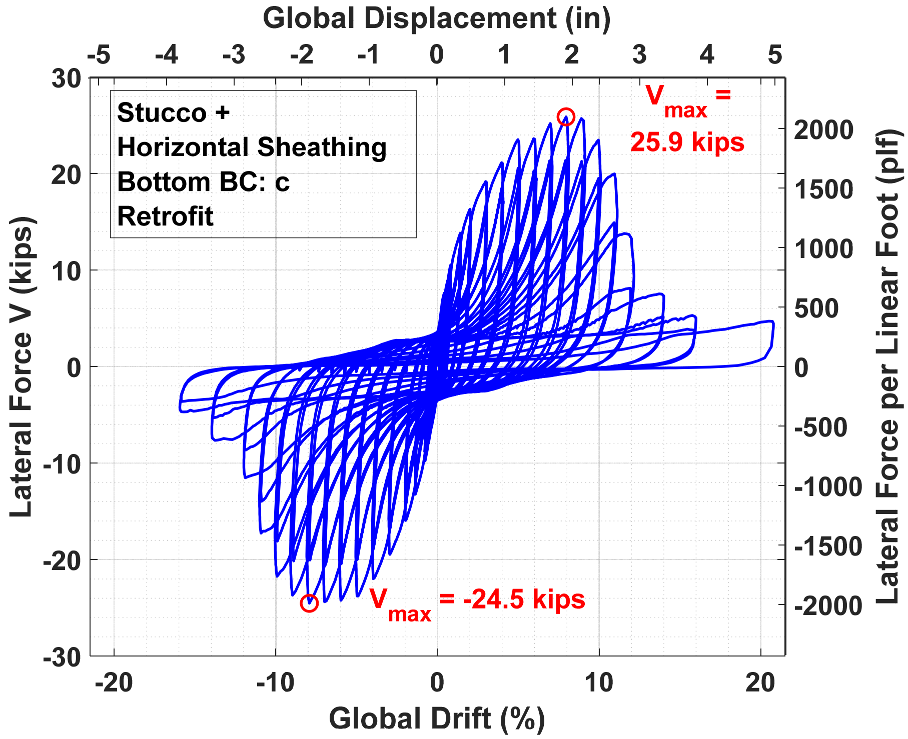


Figure 5.13 **Specimen A-19: lateral force versus global lateral drift and displacement hysteresis of [Schiller et al. 2020c].**

5.4.5 Impact of Anchorage Condition

- For the specimens with anchor bolts, denoted as typical anchorage, the oversized anchor bolt holes caused sliding of the sill plate relative to the concrete foundation and anchor bolt bearing occurred. As such, significant portions of the imposed drift were taken up by sliding of the sill plate on the concrete foundation, so much so that it became important to present the global lateral response that included the sill displacement and the relative lateral response, which omitted the sill displacement;
- The use of a wet set sill plate instead of a traditional sill plate with anchor bolts resulted in a 7% increase in strength. The wet set sill plate did not displace or damage the surrounding concrete during loading. A wet set sill plate is a sill plate that is placed into the foundation one in. while the concrete is still wet. The wet set sill plate has 2-30d nails spaced at 24 in. on center which are embedded into the concrete when the sill plate is being placed; and
- The cumulative energy dissipated was nearly identical for the global response of the two cripple walls. In terms of the relative response, the cumulative energy dissipated was around 30% larger for the wet set sill specimen.

5.4.6 Impact of Monotonic Loading Protocol

- The strength per linear foot of the monotonically loaded specimen (an unretrofitted 2-ft-tall cripple wall finished with stucco over horizontal sheathing) was 715 plf compared with 569 plf for the cyclically loaded specimen (26% increase);
- The monotonically loaded specimen achieved strength at larger displacement amplitudes than the cyclically loaded specimen as well (6% global drift ratio versus 3% global drift ratio). The relative drift ratio at 80% pre-strength was 2% versus 1.4%, at strength was 5.4% versus 2.4%, and at 40% residual strength was 9.7% versus 7.7%, for the monotonic versus the cyclic specimen; and

The secant stiffness associated with the relative drift at 80% pre-peak load, however, did decrease by 20% for the monotonically loaded specimen. However, at early drift amplitudes, the monotonically loaded specimen was stiffer than the cyclically loaded specimen.

5.4.7 Damage Characteristics

- Cracking of the stucco was minimal on the face of the cripple walls not detailed with finish around the ends (corners) (i.e., top boundary condition A). When corners were present on the cripple walls, significant cracking propagated vertically and diagonally at the corners, even at low drift amplitudes (0.2% to 1.4% drift). In addition, when corners were detailed with additional finish (i.e., top boundary conditions B and C), vertical cracks also appeared on the exterior stucco face at the same low drift amplitudes. The extent of cracking and crack widths increased as the imposed drift increased. At large drift amplitudes, crushing and spalling of the stucco was observed for cripple walls with corner conditions, particularly at the interface with the concrete foundation;
- After attainment of lateral strength, the lateral resistance contribution from the stucco was greatly reduced due to loss of its connection to the sheathing and/or framing members (i.e., detachment of the furring nails);
- At very large drift amplitudes, the gaps between all sheathing boards (except for the sheathing board attached to the top plates) closed, and the sheathing boards began to bear on each other, thus resisting the lateral displacement of the cripple wall and causing a significant retention of the lateral strength of the cripple wall up to large drift amplitudes. This phenomenon was never fully realized for diagonal sheathing boards as the loss of strength was associated with anchor bolt fractures and/or cross-grain sill plate cracks;
- The stucco over diagonal sheathing specimens tended to develop cross-grain sill plate cracks due to the uplift of the diagonal sheathing boards during loading. Since the sheathing was attached to the exterior edge of the sill plate, large uplift forces were transferred into one side of the sill plate while the center of width of the sill plate is restrained by anchor bolts causing cross-grain bending of the sill plate;

- The stucco finish provided most of the stiffness and lateral strength of the cripple walls in all unretrofitted cases. Once the wall achieved its lateral capacity, the strength of the cripple wall decreased, mostly due to the detachment of the stucco from the furring nails but also from the detachment of the furring nails from the sheathing and framing members. As drift amplitudes increased, the stucco finish was pushed out laterally at the base of the cripple wall from the sheathing and framing members as the furring nails detached. In many cases, at larger drift amplitudes the stucco finish only retained its connection to the top plates, thus providing very little lateral strength to the wall specimen. For taller specimens, the stucco remained attached to the top third of the stud height; and
- Failure of the retrofitted cripple wall was primarily attributed to sheathing nail head pull through and/or nail withdrawal along the edges of the plywood panels, especially along the top plate and sides. At the bottom of the plywood panels, nails withdrew from the framing; the added blocking split at large displacements. Some tearing of the nails through the plywood panels (edge tear-out) was observed at the corners.

5.5 CRIPPLE WALL SMALL-COMPONENT TEST PROGRAM: COMPARISONS

The focus of this final effort in the testing series performed at UC San Diego was to compare results and observations across all 28 specimens, both wet and dry exterior finishes, with additional attention towards cripple wall height and retrofitted condition. These experiments involved imposition of combined vertical loading and quasi-static reversed cyclic lateral load onto cripple walls of 12 ft in length and 2 ft or 6 ft in height. Key results compared herein include: measured force-displacement and key parameters such as strength, stiffness, displacement at strength, and post-peak strength. In addition, comparison of the evolution of physical damage amongst the wide variety of specimens is discussed. Some of the key findings are presented below.

- For 2-ft-tall existing cripple walls, horizontal siding was the weakest exterior finish tested, with an average lateral strength of 172 plf in both loading directions. Horizontal siding over diagonal sheathing was the strongest exterior finish tested, with an average lateral strength of 1435 plf;
- The strength of stucco over framing (551 plf), T1-11 wood structural panels (558 plf), and stucco over horizontal sheathing (569 plf) all had lateral strengths within 5% of each other for the 2-ft-tall existing specimens. The envelopes of the hysteretic responses for the 2-ft-tall existing specimens with the various exterior finishes are provided in Figure 5.14. The corresponding lateral strengths per linear foot are shown in Figure 5.15. The same figures for the 6-ft-tall existing specimens are given in Figure 5.18 and Figure 5.19;
- For the retrofitted 2-ft-tall cripple walls, T1-11 wood structural panels were the weakest exterior finish, with an average lateral strength of 1103 plf; horizontal siding over diagonal sheathing was the strongest, with an average lateral strength of 2550 plf;

- The strengths of stucco over framing (1815 plf), horizontal siding (1830 plf), and stucco over horizontal sheathing (2037 plf) all had lateral strengths within 10% of each other for the existing 2-ft-tall specimens. The envelopes of the hysteretic responses for all retrofitted 2-ft-tall specimens with the various exterior finishes are provided in Figure 5.16. The corresponding lateral strengths per linear foot are shown in Figure 5.17. The same figures for the retrofitted 6-ft-tall specimens are given in Figure 5.20 and Figure 5.21;
- All specimens with diagonal sheathing experienced fractures to the anchor bolts and/or cross-grain splitting of the sill plate resulting in loss of strength for the cripple walls. This was the only exterior finish material where these events occurred;
- Horizontal siding had the lowest initial secant stiffness of any of the cripple walls for both the 2-ft-tall and 6-ft-tall specimens: 10.5 kip/in. and 0.45 kip/in., respectively. Stucco over framing had the highest initial secant stiffness for both cripple wall heights, with 57.7 kip/in. for the 2-ft-tall cripple specimen and 20.5 kip/in. for the 6-ft-tall specimen. The initial secant stiffness is defined as the secant stiffness associated with drift at 80% pre-strength; and
- Stucco over framing had the lowest drift capacity of all exterior finishes tested, achieving strength at 1.2% relative drift and 40% residual strength at 4.5% relative drift. Horizontal siding had the largest drift capacity, achieving strength at 4.0% relative drift and 40% residual strength at 10.5% drift.

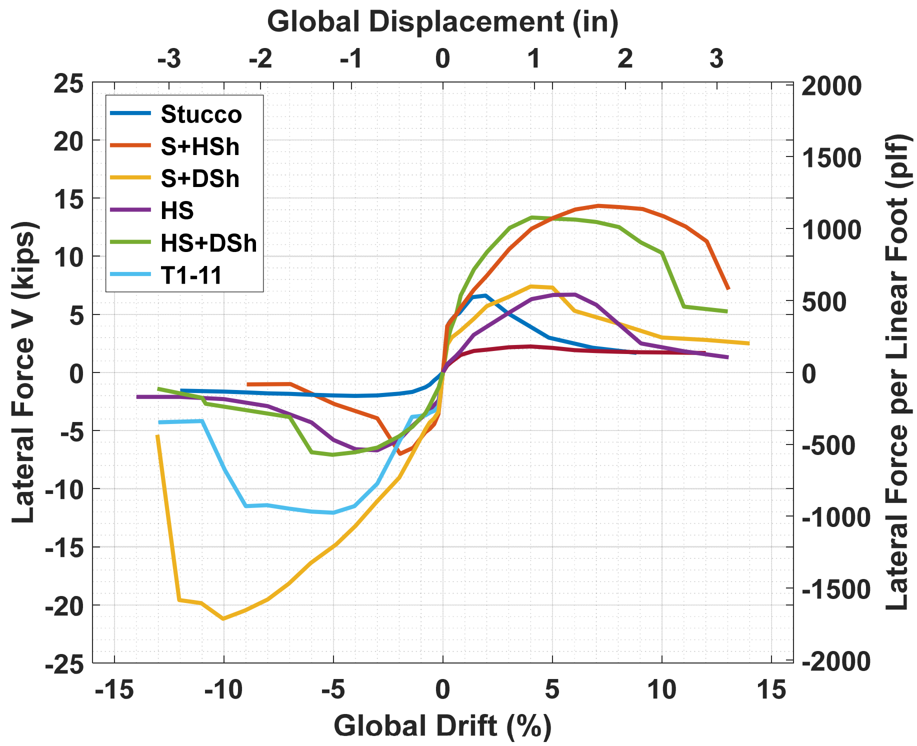


Figure 5.14 Comparison of envelopes of lateral strength: lateral displacement hysteretic response of the existing 2-ft-tall cripple walls [Schiller et al. 2020(d)].

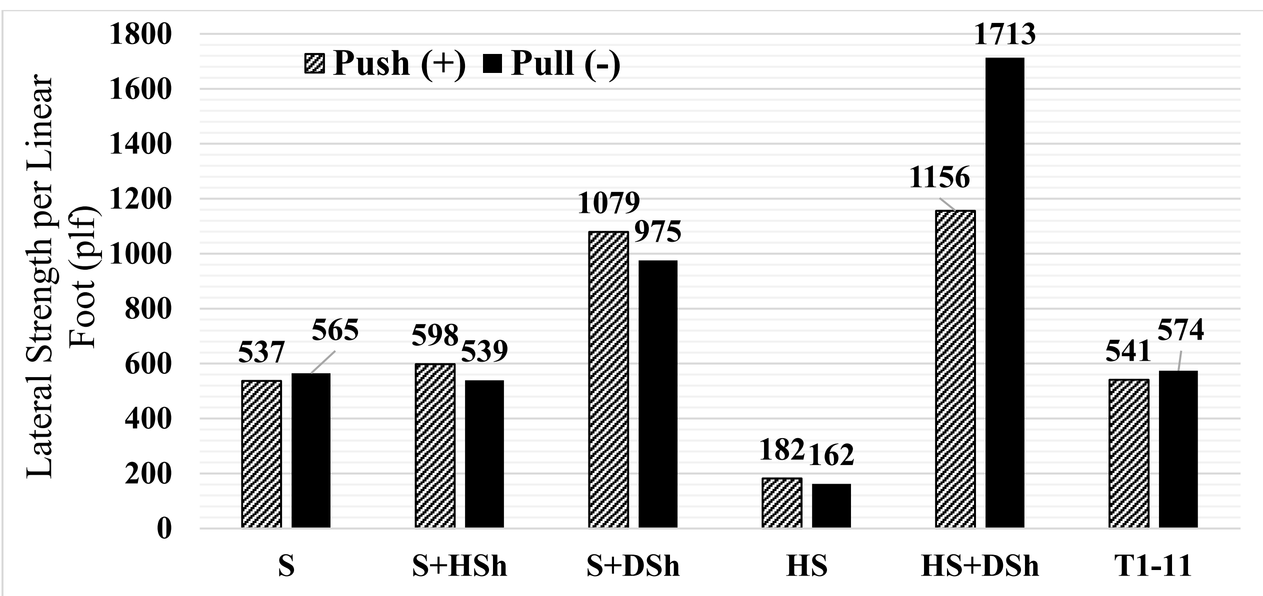


Figure 5.15 Comparison of lateral strength per linear foot of the existing 2-ft-tall cripple walls [Schiller et al. 2020(d)].

* Notes: S = stucco, HS = horizontal siding, T1-11 = T1-11 wood structural panels, HSh = horizontal sheathing, and DSh = diagonal sheathing.

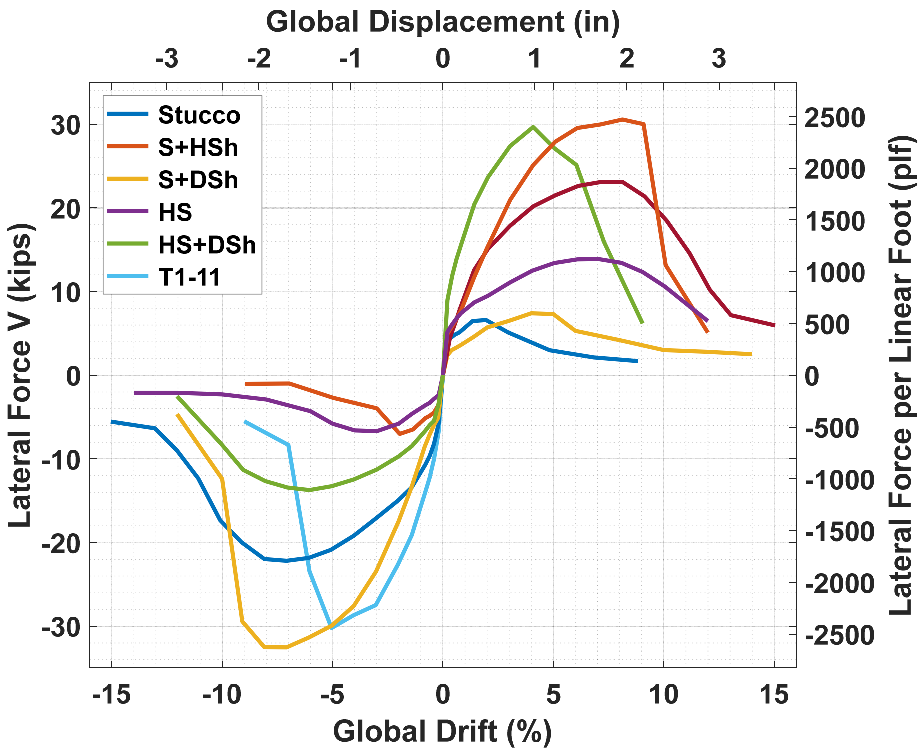


Figure 5.16 Comparison of envelopes of lateral strength: lateral displacement hysteretic response of retrofitted 2-ft-tall cripple walls [Schiller et al. 2020(d)].

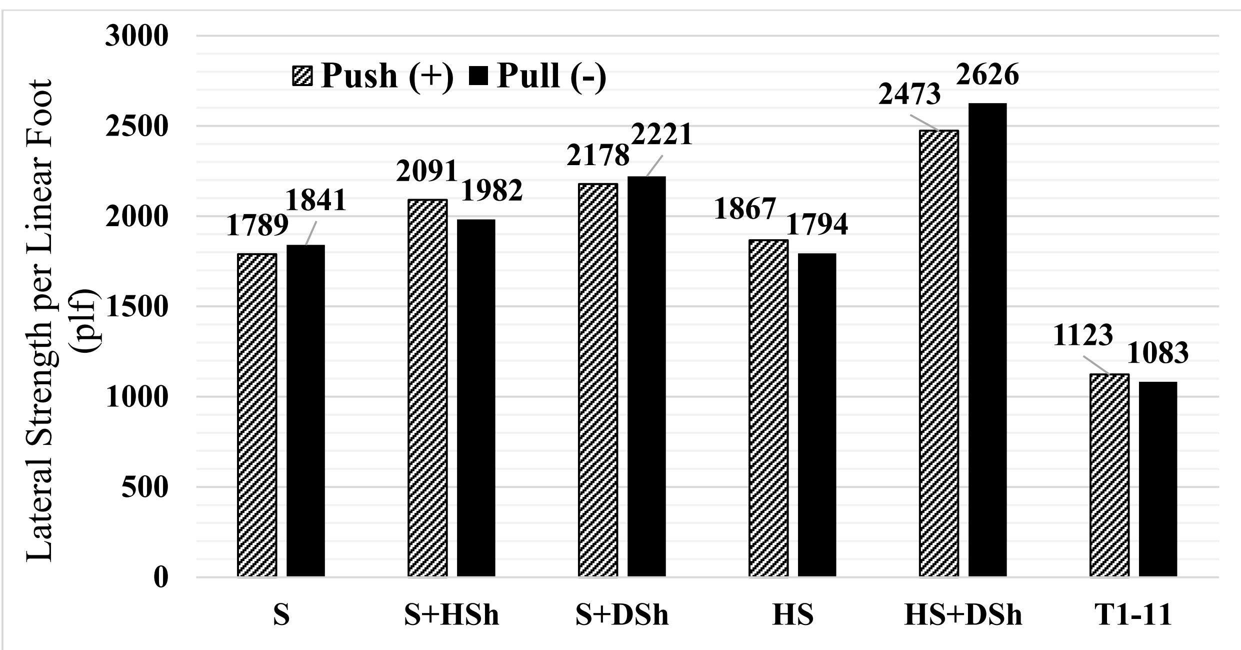


Figure 5.17 Comparison of lateral strength per linear foot of the retrofitted 2-ft-tall cripple walls [Schiller et al. 2020(d)].

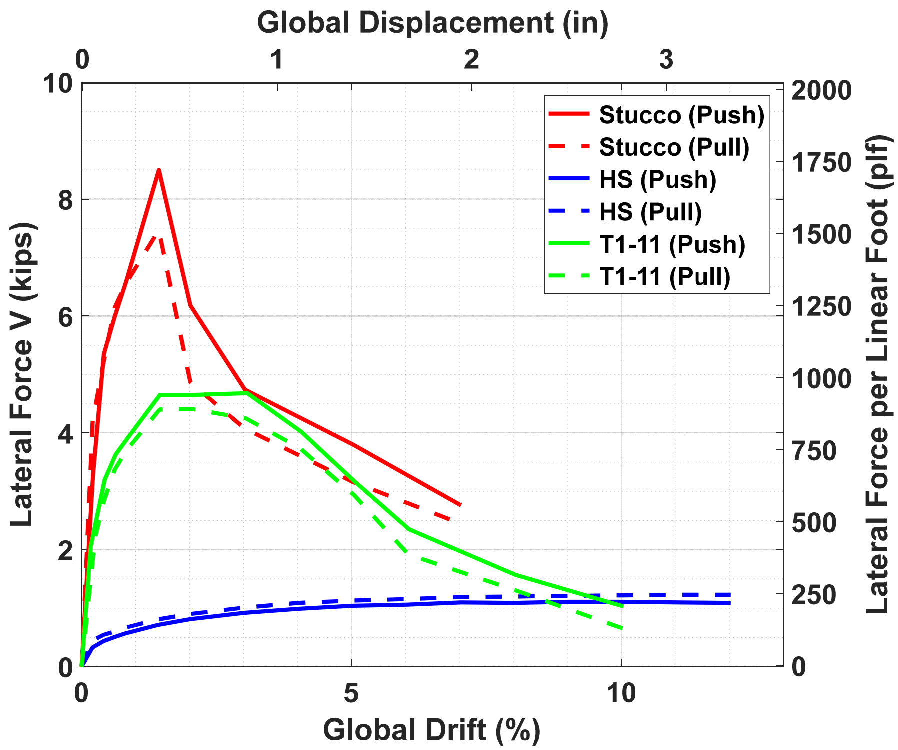


Figure 5.18 Comparison of envelopes of lateral strength: lateral displacement hysteretic response of the existing 6-ft-tall cripple walls [Schiller et al. 2020(d)].

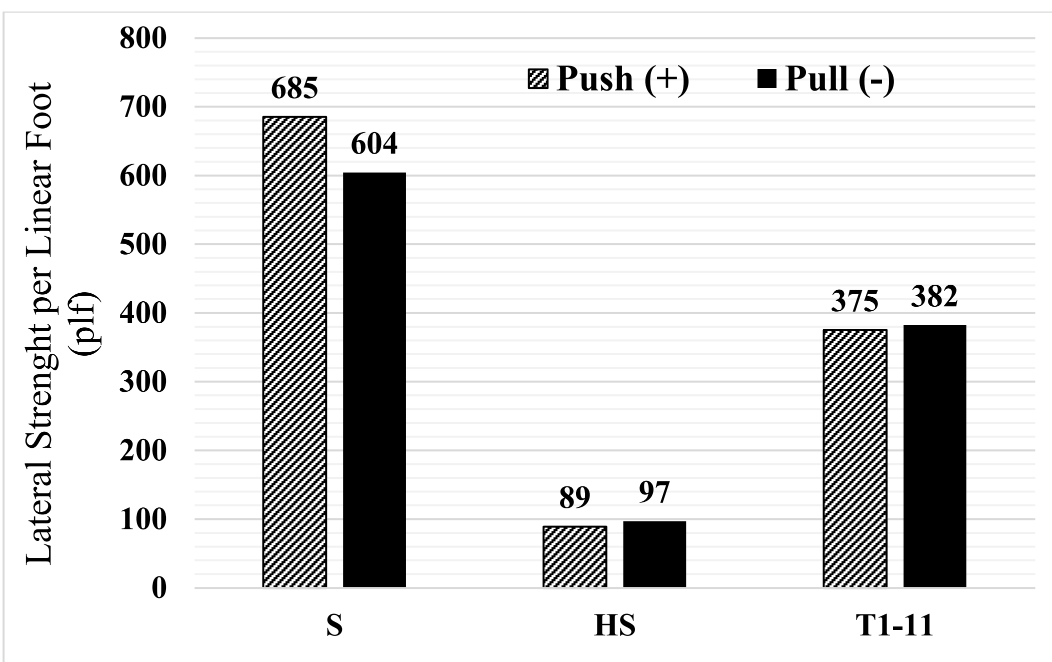


Figure 5.19 Comparison of lateral strength per linear foot of the existing 6-ft-tall cripple walls [Schiller et al. 2020(d)].

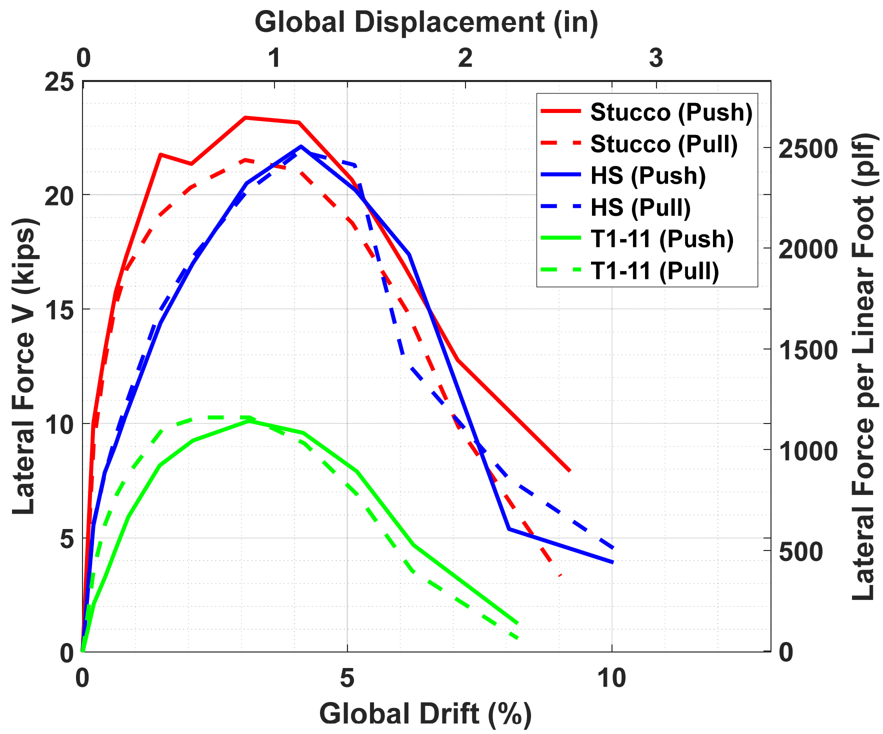


Figure 5.20 Comparison of envelopes of lateral strength: lateral displacement hysteretic response of the retrofitted 6-ft-tall cripple walls [Schiller et al. 2020(d)].

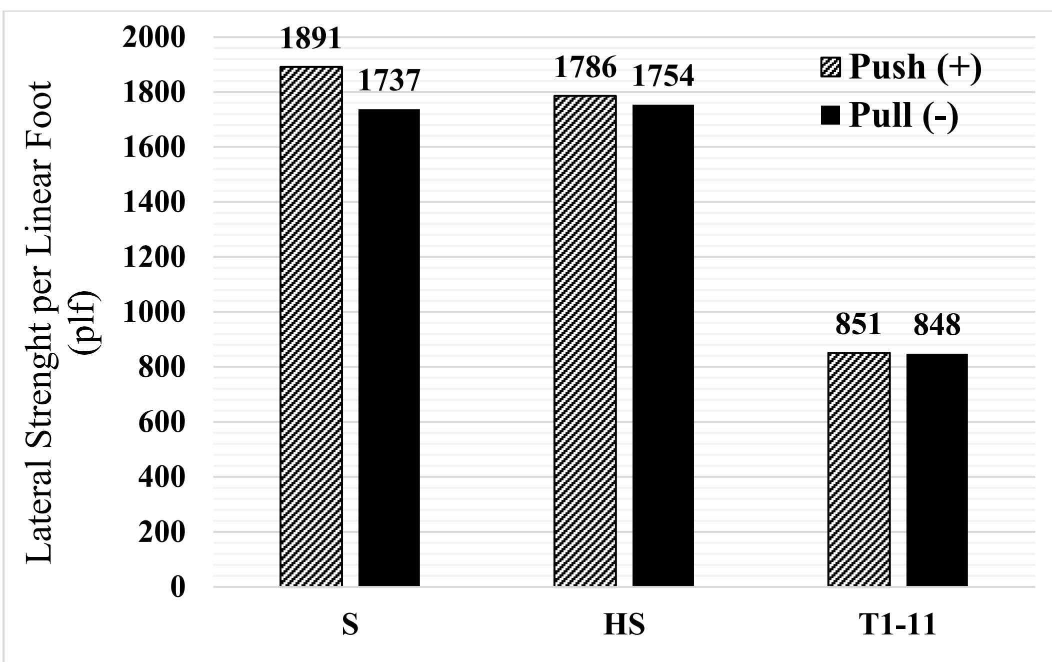


Figure 5.21 Comparison of lateral strength per linear foot of the retrofitted 6-ft-tall cripple walls [Schiller et al. 2020(d)].

6 Working Group 4b: Large-Component Testing

Working Group Leader and Participants: Kelly Cobeen, Vahid Mahdavifar, Tara Hutchinson, Brandon Schiller, David P. Welch, Grace S. Kang, Yousef Bozorgnia, Bret Lizundia, Seb Ficcadenti, Thor Matteson, and John van de Lindt

6.1 INTRODUCTION

Working Group 4B focused on portions of the Working Group 4 testing conducted at UC Berkeley: two large-component cripple wall tests (Tests AL-1 and AL-2), one test of cripple wall load-path connections (Test B-1), and two tests of dwelling superstructure construction (Tests C-1 and C-2). Included are details of specimen design and construction, instrumentation, loading protocols, test data, testing observations, discussion, and conclusions. Note: the terms "existing" and "unretrofitted" are used interchangeably in this report.

6.2 SPECIMENS AL-1 AND AL-2

Specimens AL-1 and AL-2 investigated the seismic performance of cripple walls with a stucco exterior finish installed over horizontal lumber sheathing. Both tests included a 2-ft-tall cripple wall. Specimen AL-2 included a cripple wall retrofit designed in accordance with *FEMA P-1100* [FEMA 2018(a)], while Specimen AL-1 was not retrofitted; the specimens were otherwise identical. Earlier tests [Arnold et al. 2003(a); (b)] had identified realistic boundary conditions as significantly affecting both the peak capacity and drift at peak capacity of full-story-height walls with a stucco exterior finish. Therefore, to include the most representative boundary conditions, the test specimens were three dimensional (3D) structures with plan dimensions of 20 ft × 4 ft. Each test specimen was constructed on top of a cast concrete foundation and included a 2-ft-tall cripple wall that extended to reproduce a full structure perimeter, a floor diaphragm, an 8-ft-tall superstructure, and a roof diaphragm; see Figure 6.1. This configuration allowed continuity of the stucco exterior finish around the corners, continuity of the stucco from the cripple wall into the superstructure above, and continuity of the stucco down the face of the foundation (a common detail in older stucco clad houses). Loading was applied parallel to the 20-ft-long walls.

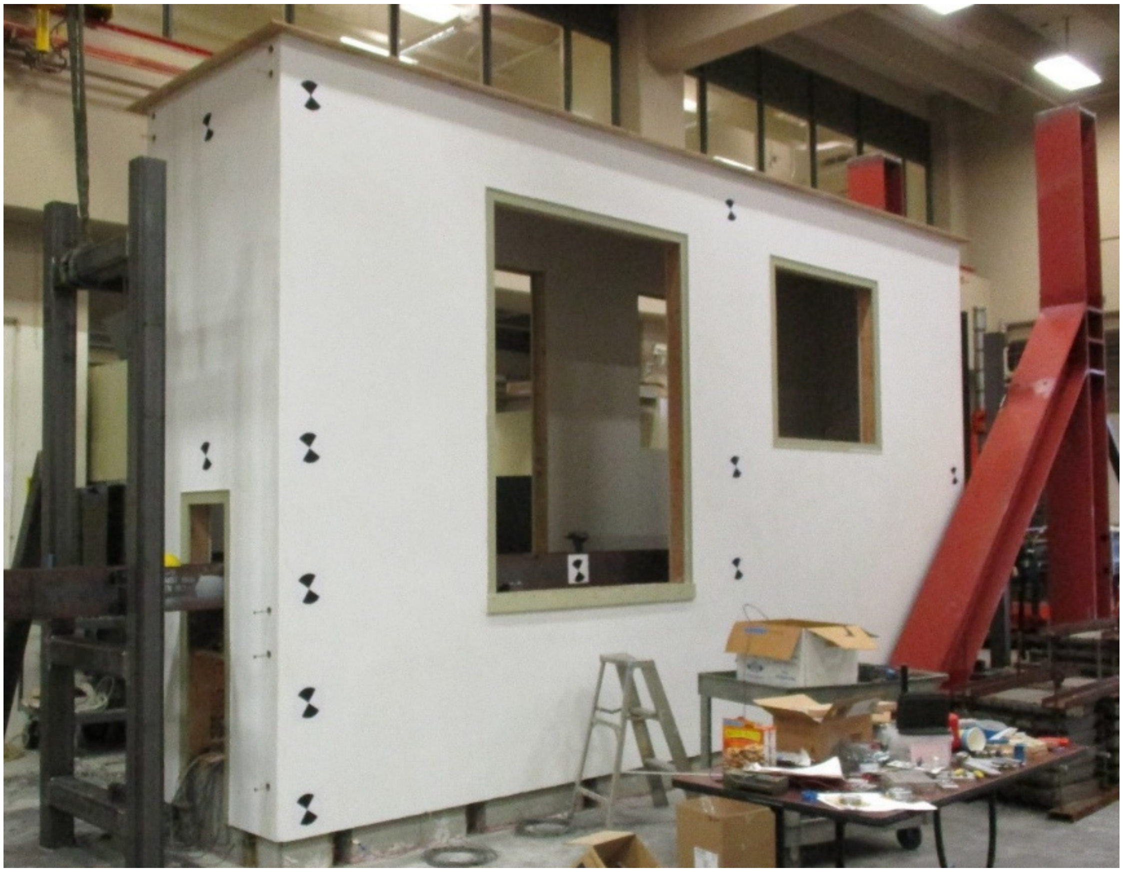


Figure 6.1 Specimen AL-1 prior to start of testing showing the superstructure wall above and cripple wall below.

In addition to the primary objectives noted above, Specimens AL-1 and AL-2 permitted direct comparison of cripple wall performance with and without retrofit, and provided data that could be compared to the PEER–CEA Project small-component tests conducted at UCSD. Comparison with the UCSD small-component tests is discussed in a separate PEER–CEA Project Working Group 4 task.

The following are highlights of the test results for Specimens AL-1 and AL-2 and conclusions based on these results:

- Specimen AL-1 reached a peak capacity (lateral strength) of 51 kips (1300 plf) at a drift ratio of approximately 2.8%;
- Specimen AL-2 reached a peak capacity of 82 kips (2100 plf) at a drift ratio of approximately 2.8%;
- These peak capacities approach the capacities in the testing reports by Arnold et al. [2003(a); (b)], which are significantly higher than other previous tests of stucco wall finishes. The peak capacities are also significantly higher than nominal capacities used to assign allowable shear for design. The capacities achieved are believed to represent an upper bound of strength due to the boundary conditions used;
- The drift ratios at peak capacity of both Specimens AL-1 and AL-2 were significantly greater than observed in previous tests of full-story-height stucco

finished walls. This continues a trend first observed in testing by Chai et al. [2002] of drift ratios at peak capacity being higher in cripple walls than in full-story height walls;

- While the capacity of Specimen AL-1 dropped off notably following cycles at peak capacity, in a final monotonic push it retained 13% of peak capacity to a drift ratio of 38% (a 9-in. drift). Note: prior testing had not extended to drift ratios this large; see Figure 6.2;

- Specimen AL-2 did not experience a drop off in capacity following cycles at peak capacity, but it retained 60% of the peak capacity to a drift ratio of 10% (2.4 in.) and 40% of peak capacity to 12%. In a final monotonic push, it retained 14% residual capacity to a drift ratio of 42% (10 in.). The increase in retained post-peak capacity demonstrates significant benefit from the retrofit; see Figure 6.3;

- The peak capacity of Specimen AL-2 (the retrofitted cripple wall) was significantly greater than Specimen AL-1, with a 60% increase in peak capacity at 2.8% drift; see Figure 6.4. Although this is a significant increase in capacity, the peak capacity of Specimen AL-2 is believed to have been reduced by the staggering of retrofit sheathing nails between the upper and lower top plates of the cripple walls. Regardless, the comparison shows that retrofitting can achieve a significant increase in peak cripple wall capacity, clearly demonstrating the benefit of installing cripple wall retrofits;

- Based on observations during testing, the continuity of the stucco around the test specimen corners had a significant effect on the peak capacity, the displacement at peak capacity, and failure mechanisms. The stucco at the corners had to be significantly degraded for cripple wall drift to occur; the inclusion of the stucco wrap around corners modified damage mechanisms, which in turn changed the load and deflection behavior;

- The continuity of the stucco from the cripple wall into the superstructure was also considered to be important in the design of the specimen. There was no indication of any significant loading or slip of the furring nails in the superstructure. Racking of wall studs and nail slip seemed largely confined to the floor level and cripple walls, without visible interaction with the superstructure framing. Based on the response of the specimens, it is believed that the superstructure played a major role in retaining the stucco's vertical and planar configuration, restraining both global in-plane rotation of the stucco and in-plane racking of the stucco;

- Based on observations during testing, the continuity of the stucco down the face of the foundation was identified to be important in the specimen design and had a significant impact on the test results as the bond between the stucco and the foundation remained intact up to peak capacity. The bond is believed to have contributed to increased peak loads and reduced the displacement at peak capacity; and

- Detailed descriptions of damage observations at each drift ratio are provided.

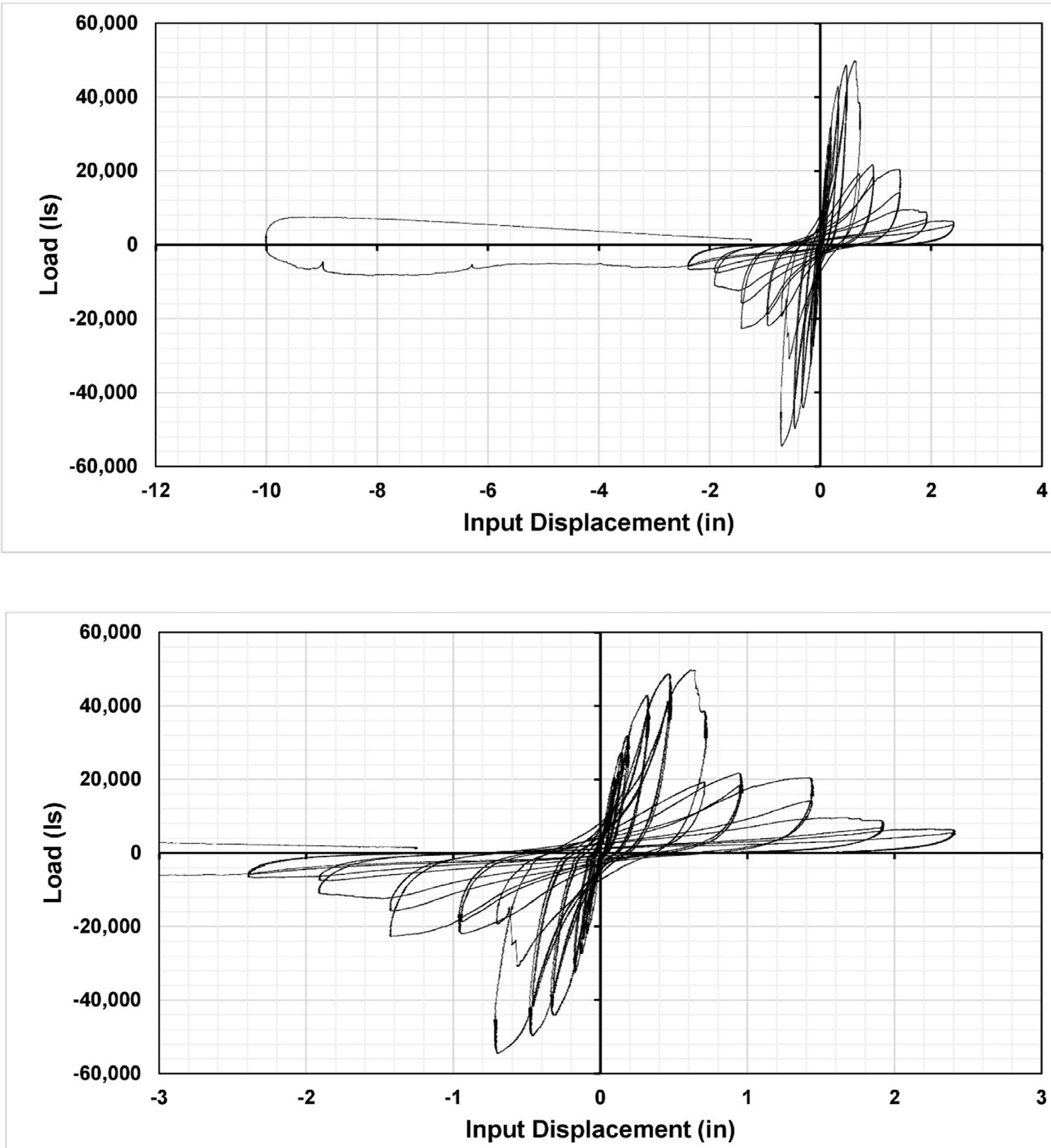
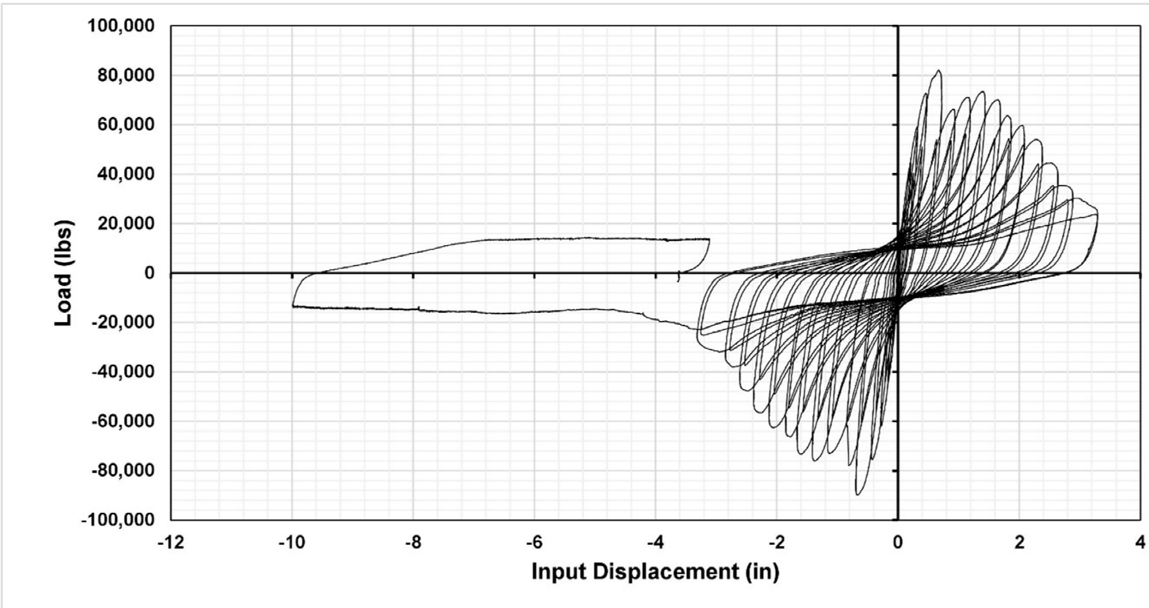


Figure 6.2 **Specimen AL-1: lateral load versus lateral displacement for (top) full hysteresis plot including monotonic push at the end; and (bottom) hysteresis plot excluding monotonic push.**



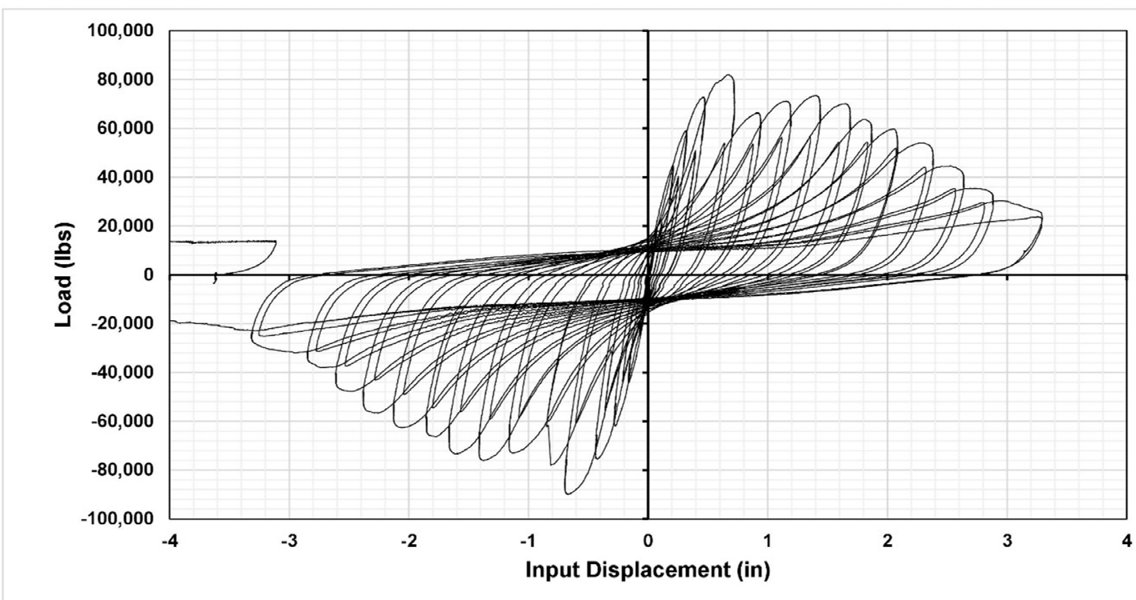


Figure 6.3 **Specimen AL-2: lateral load versus lateral displacement of (top) full hysteresis plot including monotonic push at the end; and (bottom) close-up of full hysteresis plot.**

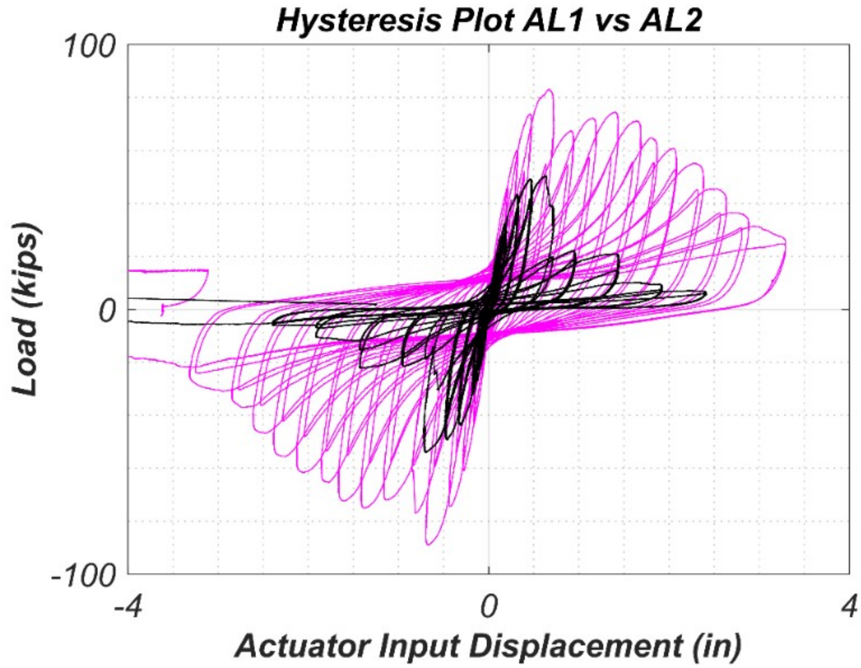


Figure 6.4 **Specimens AL-1 and AL-2: superimposed hysteresis curves. Black lines are Specimen AL-1 (pre-retrofit); magenta lines are Specimen AL-2 (with retrofit)**

6.3 SPECIMEN B-1

Specimen B-1 investigated the seismic performance of load path connectors commonly used in cripple wall retrofits. A cripple wall retrofit designed in accordance with *FEMA P-1100* was installed on 2-ft-high cripple walls with an exterior finish of horizontal wood siding. Like the other large-component tests, this test used plan dimensions of 20 ft × 4 ft. The test specimen included cripple walls at the full specimen perimeter and a high-load floor diaphragm constructed on top; see Figure 6.5. Specimen B-1 did not incorporate a superstructure story above the floor because the Project Team deemed its presence insignificant to the results.

The detailing of the cripple wall framing and relationship to the foundation was selected to be typical of construction up through the 1940s. In particular, it used a foundation sill plate that was wider that the supported studs, a detail very common in older houses in California and other western states; see Figure 6.6. The retrofit included plywood sheathing on the face of the cripple wall, shear clips (Simpson A35s) from the cripple wall top plate to the floor framing above the cripple wall, and plates (Simpson URFPs) from the 2-in. × 6-in foundation sill plate to the foundation. The use of plates is common in cripple walls that are 2 ft or less in height because of the difficulty of retrofitting a house by installing anchor bolts.

In addition to the primary objectives noted above, Specimen B-1 evaluated the ability of commonly used load path connectors to improve the strength and displacement capacity of a cripple wall with seismic sheathing. The intent of the *FEMA P-1100* provisions was to ensure that the load path connectors selected using either the prescriptive or engineered design methods of retrofit be capable of developing the combined capacity of the finish materials and sheathing retrofit. Specimen B-1 served as one data point to confirm that this is achievable.

Figure 6.5 Specimen B-1 prior to start of testing.

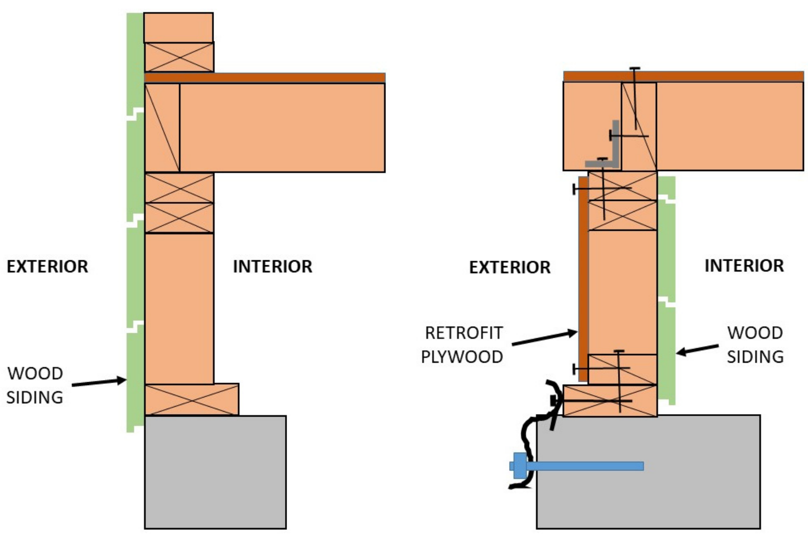
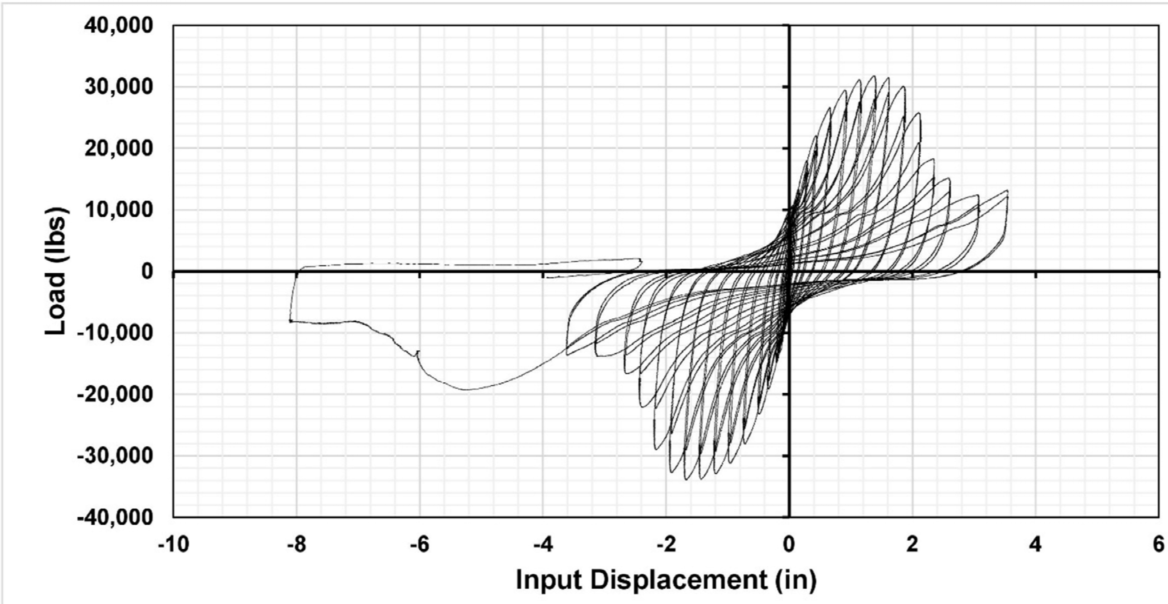


Figure 6.6 Specimen B-1 cripple wall configuration: (left) typical existing condition; and (right) with cripple wall constructed inside-out (siding on interior, retrofit on exterior).

The following are highlights of Specimen B-1 testing results and conclusions:

- Specimen B-1 reached a peak capacity (lateral strength) of 32 kips (1500 pounds per foot of plywood sheathing) at a drift ratio of approximately 6%; and
- Specimen B-1 did not experience a drop off in capacity following cycles at peak capacity, retaining 60% of the peak capacity to a drift ratio of 10% (2.4 in.) and

36% of peak capacity to 13% (3.12 in.). In a final monotonic push, it retained 24% residual capacity to a drift ratio of 33% (8 in.). The retained post-peak capacity, similar to Specimen AL-2, demonstrates significant benefit from the retrofit; see Figure 6.7.



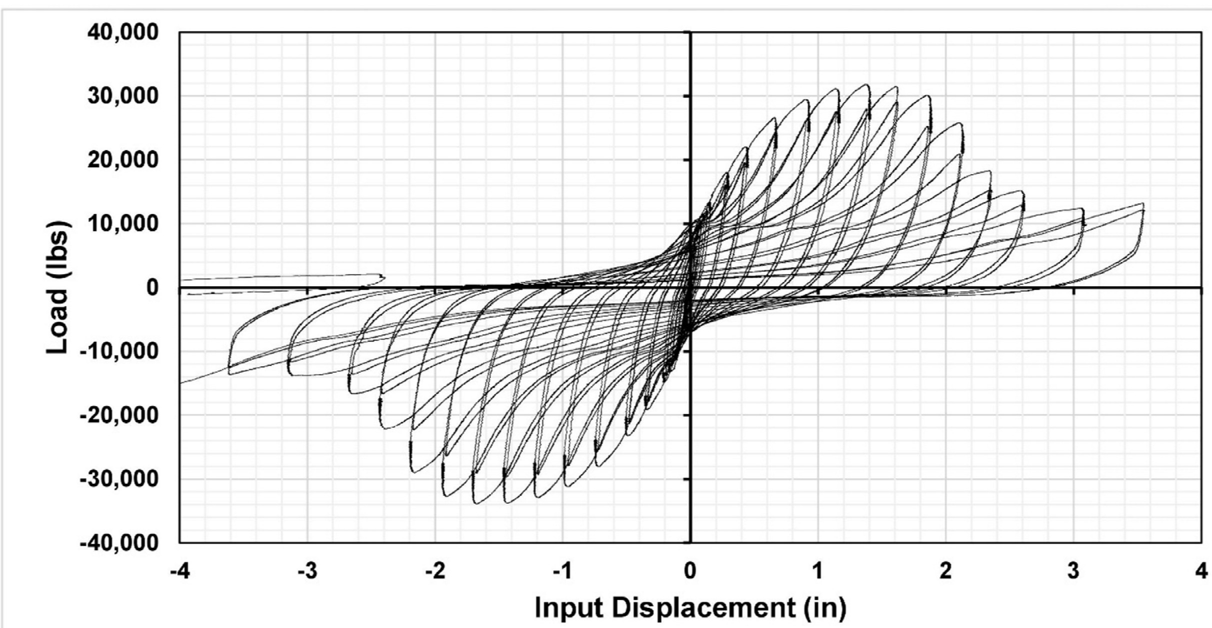


Figure 6.7 **(Top) Lateral load versus lateral actuator input displacement of Specimen B-1 with final monotonic push; and (bottom) lateral load versus lateral actuator input displacement of Specimen B-1 without monotonic push.**

- The peak capacity reached in testing is 1.25 times the sum of the tabulated nominal (wind) capacities for the horizontal wood siding and the plywood sheathing retrofit. This, coupled with observations during testing, confirm that the cripple wall was able to reach its full peak and post-peak strengths without the load path connectors (shear clips and retrofit plates) serving as weak links.
- The plywood retrofit was observed to have good strength and very high ductility and displacement capacity. The capacity of the plywood sheathing retrofit and nailing appeared to be the most significant determiner of the capacity of the cripple wall. The damage to the sheathing occurred in the common modes of sheathing slip and sheathing nail withdrawal. This confirms that the full benefit of the plywood retrofit, including both the strength and ductility, were able to be utilized.
- The load path connections designed in accordance with *FEMA P-1100* were observed to more than adequately develop the capacity of the retrofitted cripple wall. There was no observed damage and virtually no observed deformation to the connectors during testing. Similarly, there was no observed damage to the blocking installed on top of the foundation sill plate between studs. This serves as one datapoint confirming the adequacy of load path connector design in accordance with *FEMA P-1100*; and
- Detailed descriptions of damage observations at each drift ratio are provided.

6.4 SPECIMENS C-1 AND C-2

Specimens C-1 and C-2 investigated the seismic performance of wall finishes and sheathing material combinations commonly found in occupied stories of California dwellings. Test Specimen C-1 was constructed with a horizontal wood (shiplap) siding exterior finish, installed over building paper, and plaster on wood lath interior finish; see Figure 6.8. The construction was targeted to be representative of the 1930s and 1940s. Test Specimen C-2 was constructed with a plywood panel (T1-11) siding exterior finish, installed over building paper, and a gypsum wallboard interior finish (Figure 6.9); the installation of the plywood siding included a mis-installation that is prevalent in the housing stock; see Figure 6.10. Specimen C-2 used materials and construction details that were representative of housing construction practices of 1960s and 1970s. The Specimens C-1 and C-2 finish materials were specifically selected by the Project Team to supplement the limited amount of currently available data for occupied stories of dwellings. Regarding the most representative boundary conditions, the test specimens were three dimensional (3D) structures with plan dimensions of 20 ft × 4 ft. The test specimens included 8-ft-tall walls seated on the concrete foundation and a roof structure. Each of these 20-ft-long walls was constructed with one door (i.e., a sliding glass or French door) and one window, with the layout of each wall a mirror image of the other.

The configuration at the base of Specimens C-1 and C-2 was chosen to represent a dwelling in which the occupied story walls are supported on a wood-framed floor that is in turn supported on a stem wall. This base of wall configuration was reasonably common in the eras of interest and is still commonly used for new dwellings on hillsides. The adaptation of this detail for testing used a 4 × 6 nailer bolted down to the foundation, and the framed wall bottom plate nailed to the 4 × 6 nailer; see Figure 6.11. This base condition is intended to represent a lower bound but realistic condition for fastening of the wall base.

Figure 6.8 Specimen C-1 prior to start of testing.

Figure 6.9 Specimen C-2 prior to start of testing.

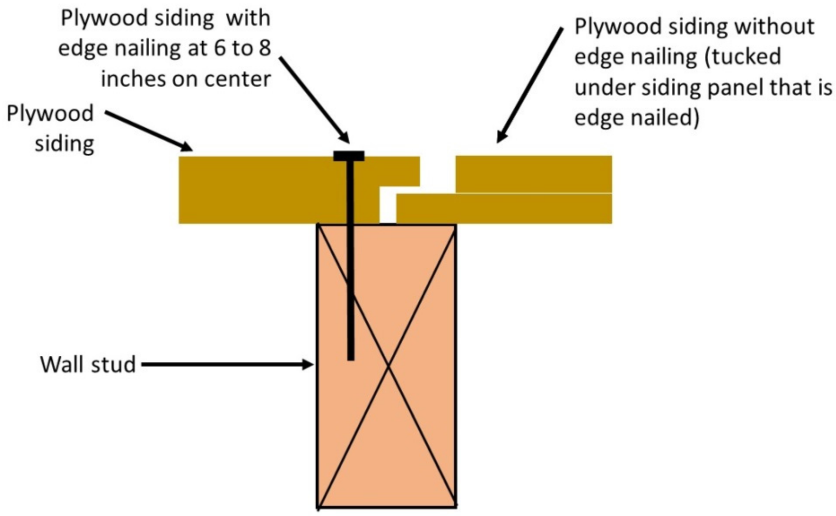


Figure 6.10 Section through stud and vertical siding joint at abutting panel edges. The mis-installation shown only includes edge nailing on one of the two abutting panels. This mis-installation was specifically included in construction of Specimen C-2.

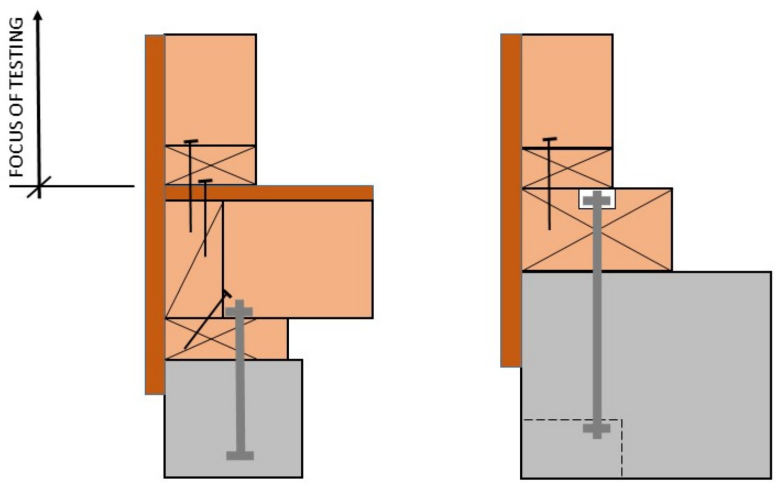


Figure 6.11 (Left) Base condition for Specimens C-1 and C-2 including the typical construction detail being represented and (right) the configuration used in the specimens.

The following are highlights of the test results for Specimen C-1:

- Specimen C-1 reached a peak capacity (lateral strength) of 11.4 kips (520 plf) at a drift ratio of approximately 1.1% in the negative quadrant (first displacement direction) and a peak lateral strength of 9.5 kips (430 plf) in the positive quadrant at a drift ratio of approximately 0.6%;
- Although the capacity of Specimen C-1 dropped off notably following cycles at peak capacity, the retained capacity stabilized at a drift ratio of 2% (1.9 in.) with a residual capacity of 6.5 kips (two-thirds of peak capacity), and

substantially maintained this capacity out to at drift ratio of 14% (13.4 in.). The testing was stopped when the test setup displacement range had been exhausted; at the conclusion of testing there was no indication that the test specimen would not be able to continue retaining capacity to higher drift ratios. This retention of capacity is more consistent with available test data for horizontal wood sheathing and siding tested alone. Regardless, the retained capacity to 14% drift far exceeds published test information for any of these materials; see Figure 6.12;

- Specimen C-1 was observed to have a reached a high percentage of the peak capacity in the very first displacement cycle to 0.2% drift, accompanied by significant popping noises and cracking and spalling of the plaster occurring in the very first displacement cycles. This suggests that the plaster would require repair at much lower drift levels than the materials tested in other PEER–CEA Project specimens;

- Available data from previous tests show significant variation in peak capacity of specimens with plaster on wood lath. The peak unit shear capacity of Specimen C-1 falls at the lower end of the range of observed strengths. There are a number of aspects that could have contributed to the strength for Specimen C-1 falling towards the bottom end of available data, even though constructed in controlled laboratory conditions. Two primary aspects that bear consideration are the materials and workmanship, and the age of the plaster at testing. Plaster on wood lath is an archaic construction type that is rarely used today. Although utmost care was taken to use representative materials and installation techniques, and work was performed by contractors that are regularly involved in installing similar systems, the materials or workmanship could have varied from that used in the 1930s and 1940s in a way that affected performance. These tests are being compared to tests conducted on wall assemblies constructed circa 1930, and test results using plaster constructed in the laboratory with very non-typical materials and techniques. Another aspect that bears consideration is that Specimen C-1 was tested approximately one month after installation of the plaster finish coat. Although this was decided to be a reasonable age to allow curing to near target strength, it is possible that due to age, the physical properties of the plaster varied from plaster in place since the 1930s. This is a recognized and unavoidable limitation of laboratory testing;

- Specimen C-1 displacement at peak capacity can be compared to prior testing results of stucco finishes with attention given to boundary conditions and prior plaster on wood lath testing where boundary conditions were not considered. While the peak capacities varied widely, it is notable that the displacement at peak capacity is very uniform, with a range of 1.0 to 1.1 in.;

- It is notable that there was no evidence of any significant uplift behavior involving separation of the 2 × 4 bottom plate from the 4 × 6 nailer bolted to the foundation. This is in significant contrast to the response of Specimen C-2; and

- Detailed descriptions of damage observations at each drift ratio are provided.

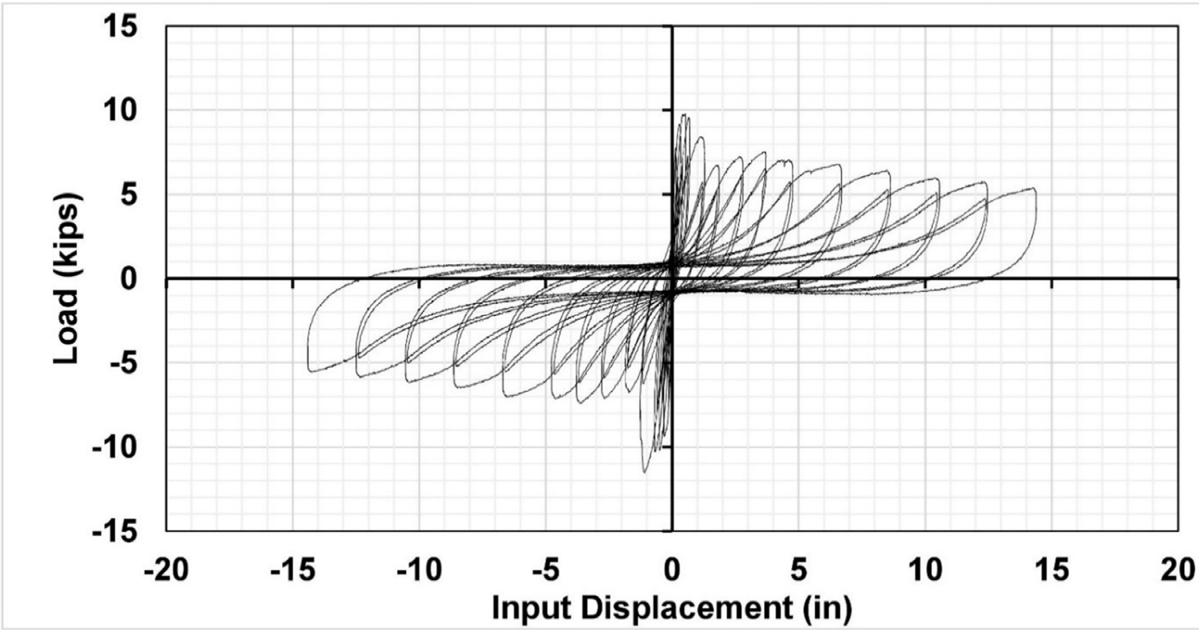


Figure 6.12 Specimen C-1: lateral load versus lateral actuator input displacement.

The following are highlights of test results for Specimen C-2:

- Specimen C-2 reached a peak capacity (lateral strength) of 18.4 kips (830 plf) at a drift ratio of approximately 2.7% in the negative quadrant (first displacement direction) and a peak lateral strength of 19.1 kips (870 plf) in the positive quadrant at a drift ratio of approximately 2.9%;
- Specimen C-2 was only able to be tested to drift ratios of approximately 6% due to significant deterioration and concerns regarding stability. At stop of testing, Specimen C-2 retained approximately 30%; of peak capacity, as shown in Figure 6.13; and
- The final failure mechanism of Specimen C-2 involved the withdrawal of practically all the nails fastening the wall bottom plates to the 4 × 6 nailers at the base of the structure. The combined withdrawal and shear created significant demands on the nails. As uplift of the specimen increased, the nails had to resist the combined withdrawal and shear loads, while also experiencing partial withdrawal reducing their capacity. Most of the nails eventually withdrew completely from the 4 × 6 nailers. Many were bent flat between the bottom plates and the nailers during load reversals. Eventually, the wall structure could be pushed back and forth across the 4 × 6 nailers and foundation fairly easily. Restraints were added during testing to 4% drift, with a second restraint strap added during loading to 6% drift. Without the restraints, it is anticipated that the base of the wall piers at the doors would have spread and fallen off the foundation. Although there was no clearly observed brittle failure, the failure mechanisms observed were severe and should be avoided. These mechanisms started to be observed at or just beyond peak capacity. Unlike engineered plywood shear walls, for which post-peak loading is thought to be

acceptable in a maximum considered earthquake, it is suggested that it would not be acceptable for the construction details used in Specimen C-2.

- Specimens C-1 and C-2 provide two examples of dwelling superstructure construction representative of two different eras of construction, both prevalent in California's housing stock. While the tests were not developed for the purpose of direct comparison, it is worthwhile to discuss a few aspects of the specimens and testing results. The base detail with a 2 × 4 bottom plate nailed to a 4 × 6 nailer was the same for both tests. Although this did not prove to be a weak link for Specimen C-1, it did act as a weak link for Specimen C-2. This could be a function of both the lower capacity of Specimen C-1 and changes in mechanics of uplift and overturning between the two different specimens;
- The peak capacity of Specimen C-2 was nearly double that of Specimen C-1, suggesting significantly better seismic performance; however, by the time it reached peak capacity, significant damage was observed and in the next displacement cycle, restraints were added to the specimen to avoid premature failure. Compare this performance to Specimen C-1, which had a lower peak capacity but maintained nearly 50% of the peak capacity out to a drift ratio of 16%. When put in terms used to classify vertical elements of seismic force-resisting systems, Specimen C-2 would be categorized as non-ductile in that catastrophic damage could occur just past peak capacity, while Specimen C-1 could be categorized as highly ductile, while having moderate capacity. Figure 6.14 captures this difference in response. This pattern is counter-intuitive, in that it would be common to categorize Specimen C-2 as ductile based on the performance of wood structural panel shear walls and categorize Specimen's C-1 configuration with plaster on wood lath as non-ductile; however, observed behavior suggests the opposite; and
- Detailed descriptions of damage observations at each drift ratio are provided.

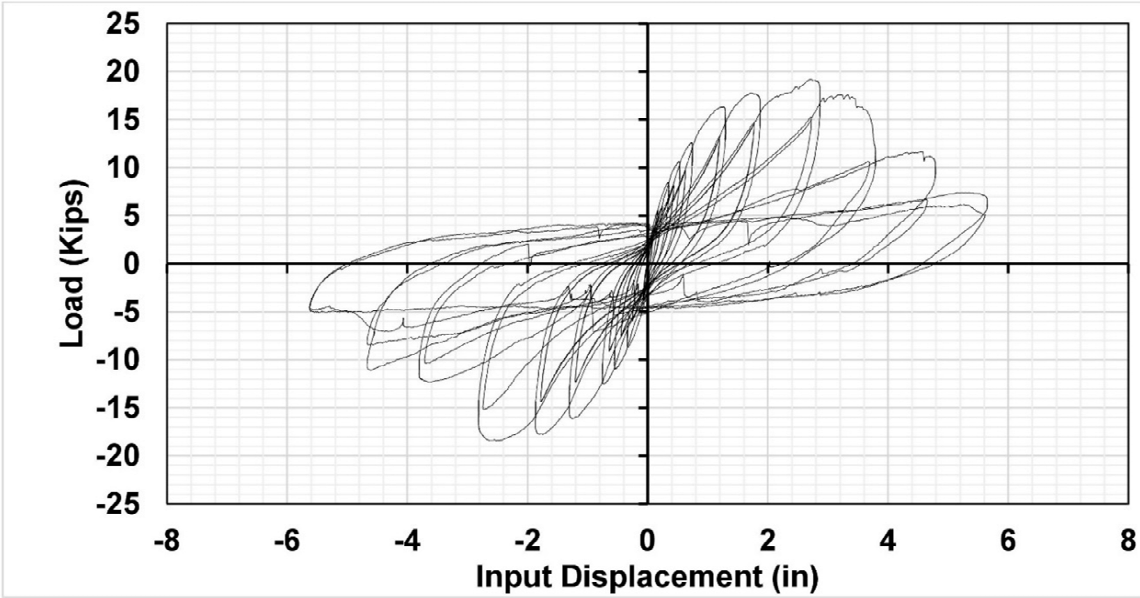


Figure 6.13 Specimen C-2: lateral load versus lateral actuator input displacement.

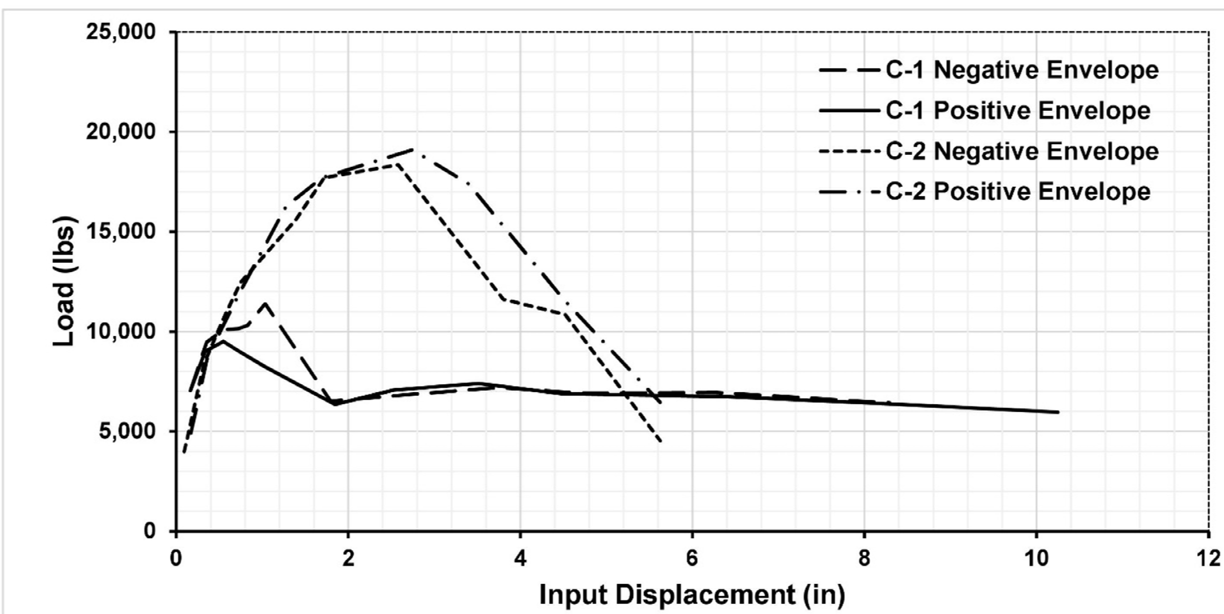


Figure 6.14 Specimens C-1 and C-2: superimposed envelope curves. Note that Specimen C-1's response is truncated at 10% drift while the testing continued to 16%.

7 Working Group 4c: Comparison of Large- and Small-Component Test Results

Working Group Leaders and Participants: Kelly Cobeen, Tara Hutchinson, and Brandon Schiller

7.1 INTRODUCTION

Working Group 4c compared the response of small-component cripple wall specimens testing conducted at UCSD with the results of those obtained from large-component testing of cripple wall specimens under simulated seismic loading conducted at UC Berkeley. The specimens being compared were all constructed with a stucco exterior finish installed over horizontal lumber sheathing; see Table 7.1. Specimens constructed with other materials were included in the Working Group 4 tests but are outside of the scope of this task. The intent of the UC Berkeley large-component cripple wall tests (Specimens AL-1 and AL-2) was to capture the effect of boundary conditions as near as possible to those that would occur in a complete house. This included providing continuity of the exterior stucco around corners, continuity of the stucco into the superstructure story above, and continuity of the stucco down the face of the foundation (a common detail in existing homes). The UC San Diego small-component tests (Specimens A-1, A-2, A-3, A-4, A-5, A-19, and A-20) explored the effects of a range of boundary conditions, applied to 12-ft-long cripple wall (only) components. Note: the terms "existing" and "unretrofitted" are used interchangeably in this report.

This pairing of small- and large-component tests was included in the Project testing plan so that a direct comparison could be made to determine the following: (1) how closely could small-component specimen response emulate that seen in the response of large-component specimens; and (2) what boundary conditions in the small-component specimens led to the best match with the response of large-component specimens. The answers to these questions are intended to help identify best practices when designing specimens in the future, with particular interest in supporting realistic design of small-component specimens that may capture large-component specimen response, and to qualitatively understand where the small-component tests fall in the range of lower-bound to upper-bound estimation of strength and deformation capacity for the purposes of numerical modelling.

Table 7.1 Pairing of small- and large-component test specimens.

Cripple wall description	Retrofit included	UC Berkeley large-component specimen	UC San Diego small-component specimens
Stucco over horizontal lumber sheathing	No	AL-1	A-1 A-2 A-3 A-4 A-20
Stucco over horizontal lumber sheathing	Yes	AL-2	A-5 A-19

7.2 SUMMARY OF CONCLUSIONS

The following are overarching conclusions regarding the load-deflection response of cripple walls with stucco over horizontal lumber sheathing:

- A commonality seen in the small- and large-component test specimens is that the drift ratios at lateral strength were significantly higher in cripple walls than in similar full-story-height walls. This confirms the pattern previously suggested by the cripple wall tests of Chai et al. [2002]; and
- Another commonality seen in the small- and large-component test specimens is an ability to retain between 10–20% of their lateral strength out to drift ratios between 10–40%. This is a significant new finding as past testing was not conducted to such large drift ratios. This information is important for numerical modeling intended to identify the probability of collapse. It is recommended that future testing extend to incipient collapse or as close to incipient collapse as permitted by the test setup.

The following are the primary conclusions regarding the specimens *without retrofit*:

- For existing specimens, the lateral strength (peak capacity) of the large-component specimen was significantly higher than that of the small-component specimens; see Figure 7.1. This is largely attributed to a combination of the continuity of the stucco from the cripple wall to the first floor, the excellent construction quality of the bond between the stucco and the foundation, and the continuity of the stucco around the entire large-component specimen. For these reasons, the large-component specimen is viewed as representing a high but achievable upper bound of lateral strength;
- For existing specimens, the drift at lateral strength of the small-component specimens had notable variation; see Figure 7.1. Small-component Specimen A-3, with 2-ft-long return walls observed its drift at lateral strength at a 2.5% drift ratio (the best match to the large-component specimen) and reached lateral strength at a 2.8% drift ratio, with comparable damage mechanisms. Specimen A-20 (which best matched the boundary conditions of Specimen AL-1) was also a close match to the large-component specimen, with a drift ratio at lateral strength of 2.4%, in addition to having similar damage mechanisms; and

- For specimens without retrofit, the extent and location of damage varied between the small- and large-component specimens; however, the mechanisms of damage remained similar.

The following are the primary conclusions regarding the specimens *with retrofit*:

- For retrofitted specimens, the lateral strength of the small- and large-component specimens were quite similar; see Figure 7.2. This may in part be due to plywood detailing of the large-component specimen retrofit, which may have prevented the sheathing from achieving full capacity;
- For retrofitted specimens, the drift at lateral strength varied notably, as seen in Figure 7.2, with the drifts of the small-component specimens being larger than that of the large-component specimens. Significant uplift at the ends of the small-component specimens may be one source of the larger drift. This uplift may be limited for the large-component specimens due to the size of these specimens and presence of the upper floors. Another source of the discrepancy might be due to the plywood panels in the small-component tests being free to move upward, thus not being partially constrained against rotation and uplift as was the case with the large-component tests; and
- For retrofitted specimens, the physical damage at a drift ratio of 1.4% was more extensive in the large-component specimen than the small-component specimen. At lateral strength, the small-component specimens experienced notably more deterioration in the plywood retrofit than the large-component specimen. At 20% residual strength, the damage to the small- and large-component specimens were similar.

The following are overarching conclusions regarding the *benefits of retrofit*:

- The increase in lateral strength with the addition of cripple wall retrofit was significant for both small- and large-component specimens as seen in Table 7.2 and Figure 7.3 through Figure 7.6. The ratio of lateral strength with retrofit to without was higher for the small-component tests; and
- At 1.4% drift, both the small- and large-component specimens with retrofit experienced less damage than the corresponding specimens without retrofit.

Table 7.2 Comparison of cripple wall lateral strength with and without retrofit.

Specimens	Specimen type	Average lateral strength without retrofit (plf)	Average lateral strength with retrofit (plf)	Ratio of lateral strength with to without retrofit
A-2 and A-5	Small	730	1965	2.7
A-2 and A-19	Small	730	2035	2.8
AL-1 and AL-2	Large	1300	2150	1.7

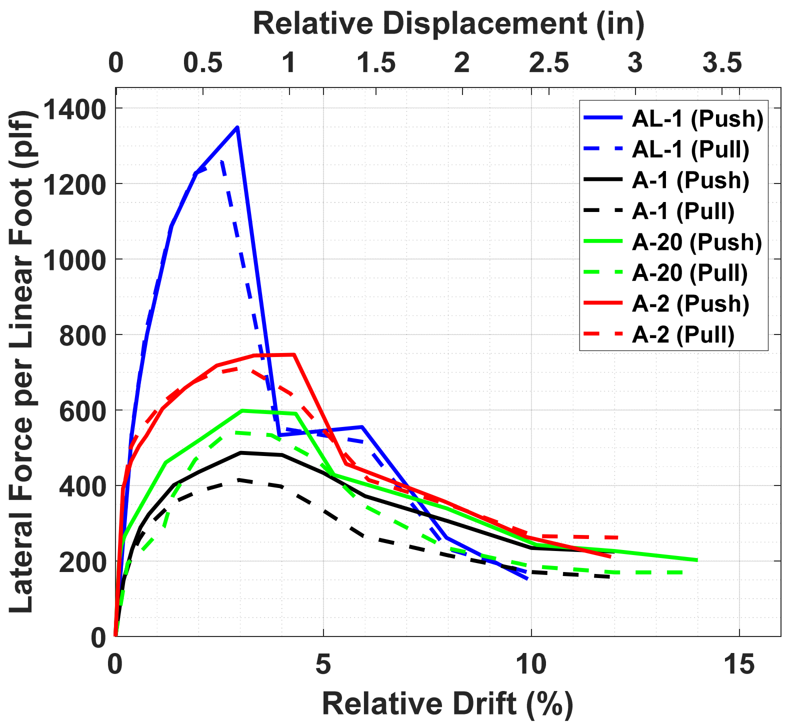


Figure 7.1 Envelope of lateral force: *relative* lateral displacement hysteresis for existing specimens.

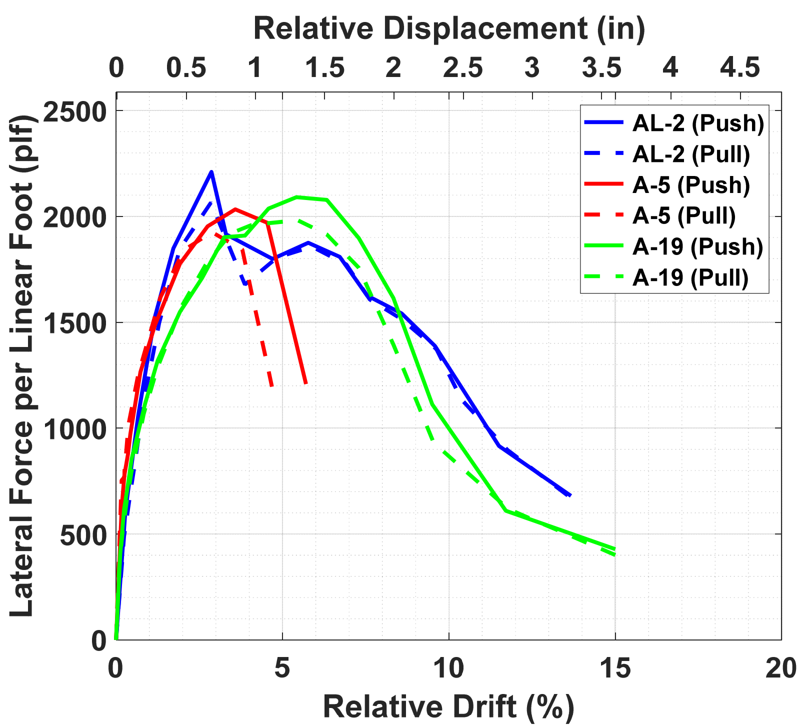


Figure 7.2 Envelope of lateral force: *relative* lateral displacement hysteresis for retrofitted specimens.

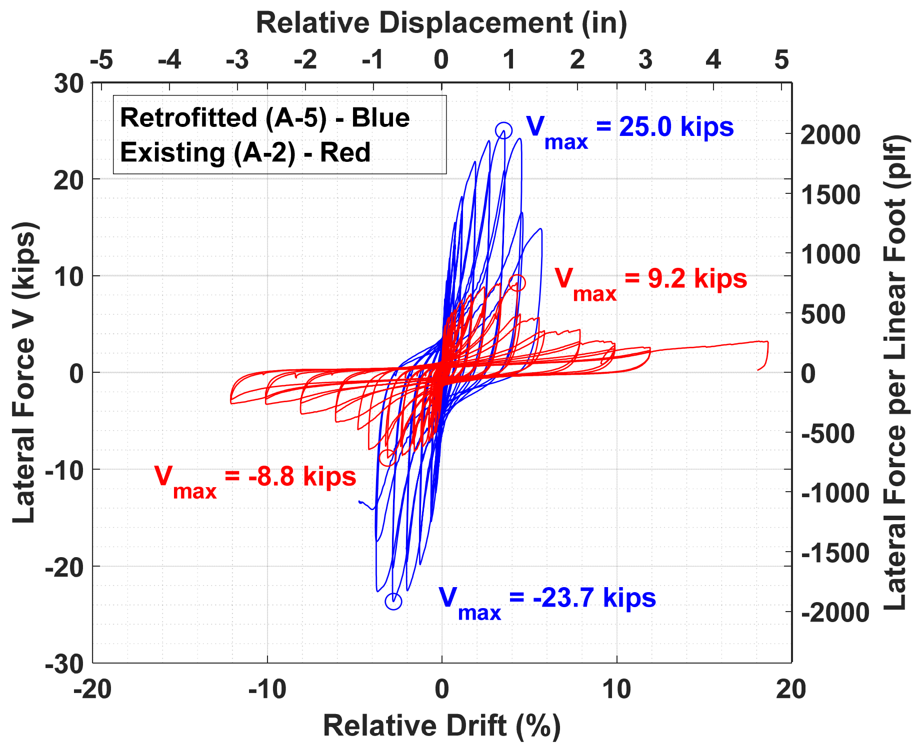


Figure 7.3 Specimens A-2 and A-5: superimposed hysteresis curves.

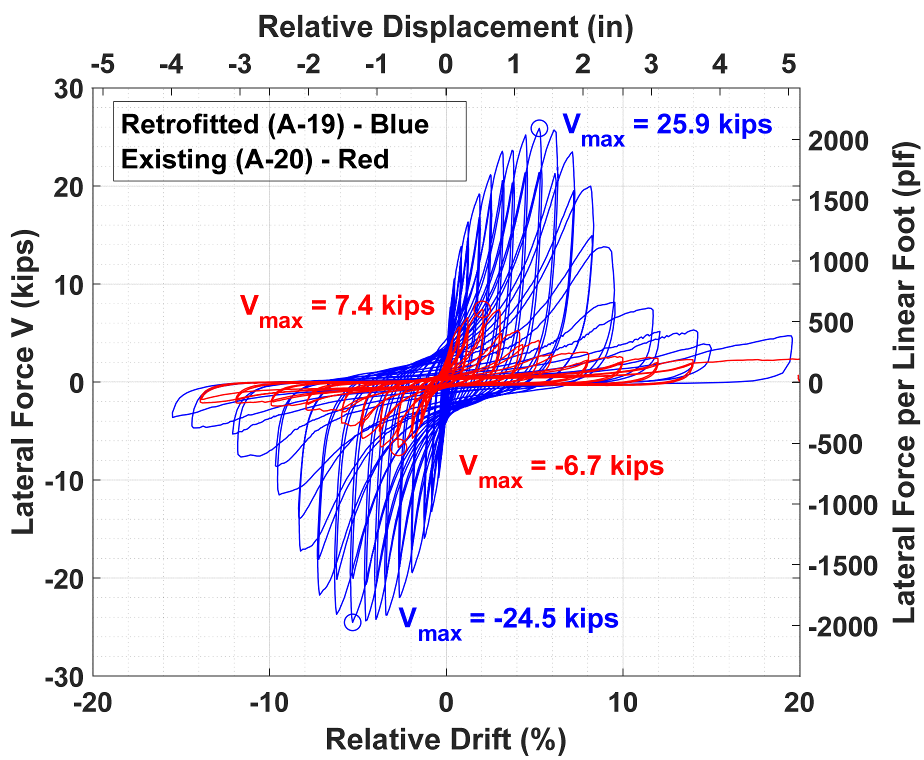


Figure 7.4 Specimens A-19 and A-20: superimposed hysteresis curves.

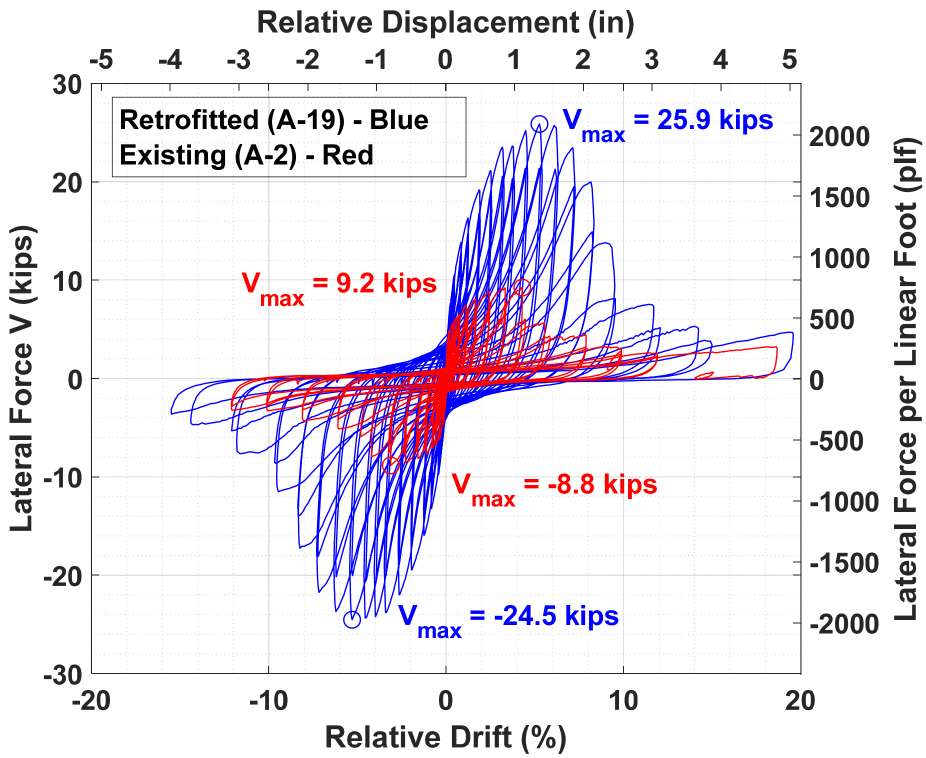


Figure 7.5 Specimens A-19 and A-2: superimposed hysteresis curves.

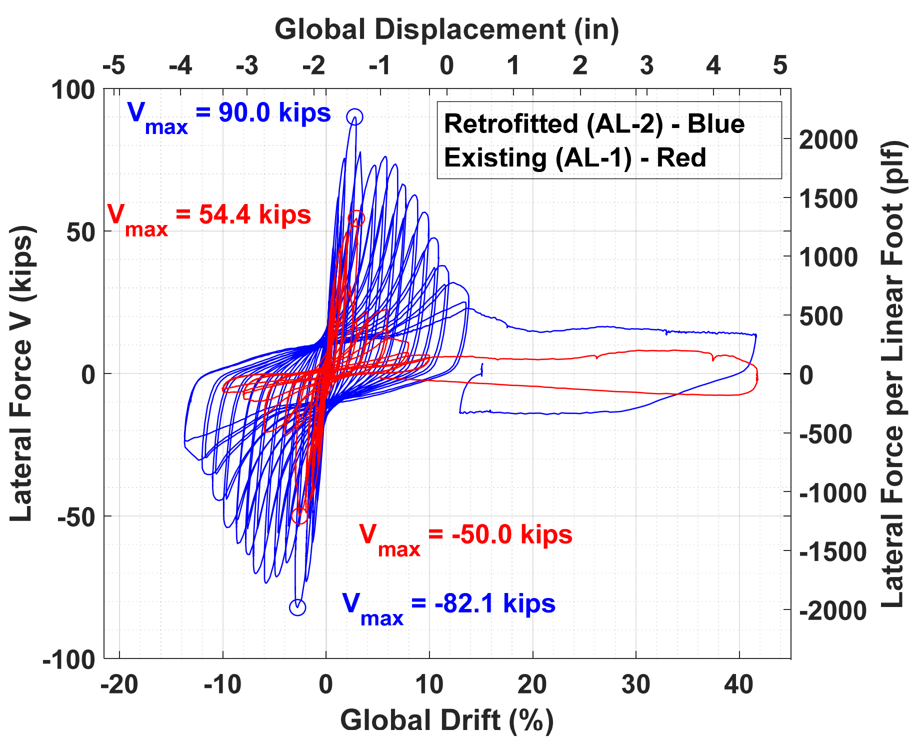


Figure 7.6 Specimens AL-1 and AL-2: superimposed hysteresis curves.

7.3 RECOMMENDATIONS

The PEER–CEA Project experimental group set out to conduct parallel small- and large-component testing to facilitate comparison and determine what small-component configurations best compared to the large-component results. To this end, the following are recommendations based on knowledge gained from this comparison.

- For future testing, the pairing of small- and large-component specimens is encouraged where practical because the results can complement each other and provide a more complete picture. Note that the large-component specimen provided upper bound lateral strength. Moreover, the complementary large-component specimens offered a more complete system, identifying additional damage patterns and weak links (such as the slip between upper and lower top plates experienced by Specimen AL-2);
- For future testing, incorporating the boundary conditions used in the PEER–CEA Project for small-component test specimens may be useful when attempting to capture conditions occurring in complete houses. These boundary conditions include: (1) additional fasteners between the stucco and the cripple wall top plates; (2) finish materials oriented outboard of (overhanging) the foundation; and (3) wrapping of the stucco around the wall end for stucco returns of between 6 in. and 2 ft; and
- Lastly, for a small-component specimen, the foundation could be modified such that finish materials on return walls are placed outboard of the foundation (i.e., overhanging), essentially matching the orientation on the long face of the cripple wall. This would better mimic typical house construction and a configuration that could be achieved with a larger-component specimen.

8 Working Group 5: Analytical Modeling

Working Group Leaders and Participants: Gregory G. Deierlein, Henry Burton, David P. Welch, Bret Lizundia, Evan Reis, Curt Haselton, and Charlie Kircher

8.1 INTRODUCTION

Working Group 5 focused on the development of numerical frameworks for the seismic performance assessment of single-family wood-frame buildings with cripple walls and sill anchorage deficiencies both with and without seismic retrofit. The seismic performance assessment framework is based on the *FEMA P-58* methodology, which represents the state-of-the-art in building-specific loss assessment. The framework is termed building-specific: it incorporates specific information for a given structure in a multi-staged framework that includes building definition, seismic hazard analysis, structural response analysis, damage assessment, and, finally, consequence (loss) assessment to arrive at the particular decision variables of interest to represent the estimated seismic performance. An illustration of the performance assessment framework is shown in Figure 8.1. Note: the terms "existing" and "unretrofitted" are used interchangeably in this report. Details of the analytical modeling are provided in the full Working Group 5 technical background report [Welch and Deierlein 2020].

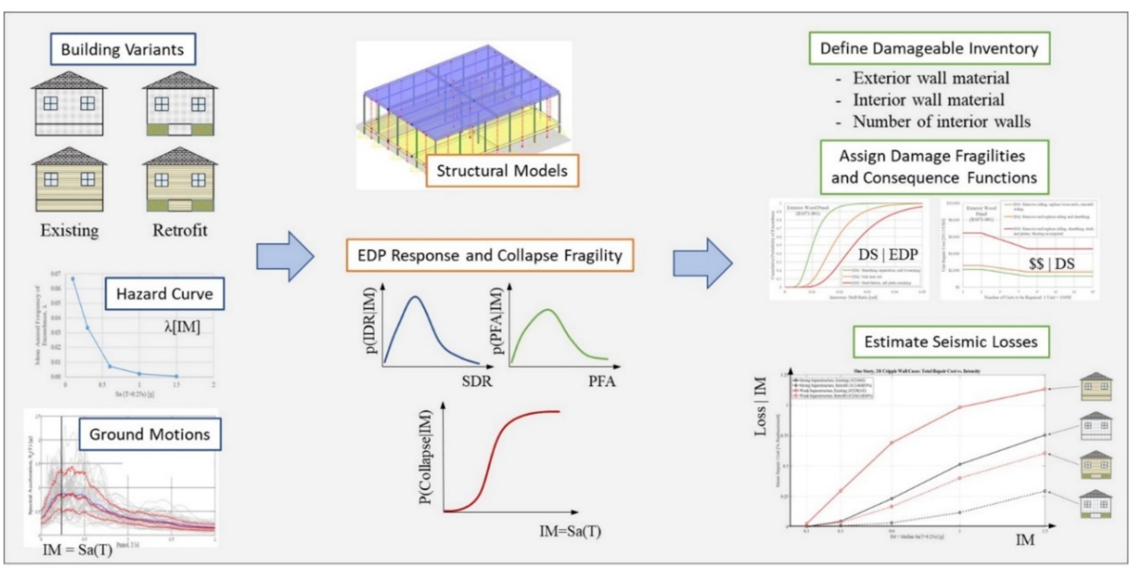


Figure 8.1 **Illustration of the building-specific seismic performance assessment process for cripple wall dwellings.**

8.2 SCOPE OF BUILDING VARIANTS FOR NUMERICAL ANALYSIS

The development of the building variant scope for numerical analysis considered available information within the literature regarding residential inventory and construction trends in California, as well as typical considerations within the catastrophe modeling industry. The initial collection of possible variants focused on those variants having a significant impact on seismic damage and, more importantly, having the differential in seismic losses due to retrofitting be affected by the presence of the variant. All retrofitting details were designed according to the recent *FEMA P-1100* prestandard developed in conjunction with the Applied Technology council [FEMA 2018(a)]. Preliminary variants included both primary and secondary modifiers applicable to the catastrophe modeling industry.

Primary variants included easily accessible and documented information for residential homes, such as the number of stories and era of construction. Secondary variants included the "observable," such as the exterior material of the home or "unobservable," such as the presence of horizontal or diagonal wood sheathing beneath the exterior finish. Initial development of the building variant list also drove the scope for experimental testing, with subsequent test data and accompanying numerical studies informing decisions on important variants to maintain throughout the course of the project. Details of building variants used for numerical analysis are provided in Section 3.1 and the full Working Group 2 report [Reis 2020].

In addition to assumed materials and cripple wall details, the configuration and layout of the variant dwellings is also important. Consistent with the ATC-110 project, the current project targeted a moderate-sized plan area of 1200 ft^2 with an aspect ratio of 0.75 (i.e., 40 ft × 30 ft). Geometrical data from the ATC-110 project was collected for many one- and two-story homes from archived housing catalogs ranging in construction era from 1900 to 1969. This data was used to maintain realistic interior and exterior wall densities as well as relative wall densities from first to second stories of two-story dwellings. Baseline configurations were selected and developed using this information. The 1200 ft^2 CUREE-Caltech small house configuration was used as the basis for variant wall configurations in the PEER–CEA study due to its agreement with the average wall density statistics obtained from the configuration study. This configuration was originally designed and analyzed within the CUREE-Caltech Woodframe Project [Isoda et al. 2002; Porter et al. 2002; Reitherman and Cobeen 2003].

Characterization of the ground-motion hazard considered ten sites around California. Through preliminary structural analysis and in direct collaboration with Working Group 3, the baseline sites and record selection procedures were established to produce the most applicable and meaningful results. Based on a review of the seismicity of the ten sites provided by Working Group 3, four sites were selected (Bakersfield, San Francisco, Northridge, and San Bernardino) to represent a range of seismicity conditions and cover the range of seismicity classifications outlined in the *FEMA P-1100* guidelines for the design of the cripple wall retrofits. Hazard representation through ground-motion selection was selected to be the most consistent with state-of-the-art practices; the use of intensity measures was consistent with the catastrophe modeling industry, and a balanced approach considering the scope limitations of the project was maintained. Through interaction with Working Group 3 and sensitivity analyses, the seismic hazard is represented by conditional spectra using a conditioning period of 0.25 sec.

Structural analysis of building variants required the development of finite-element models that capture the appropriate dynamic behavior of various house configurations and building materials. This required quantifying the appropriate mass, strength, stiffness, and deformation capacity of individual materials. Development of these models required an extensive review of existing experimental data within the literature. Interaction with the experimental Working Group 4 was an iterative process of testing decisions based on current knowledge and new knowledge gained through testing. The structural modeling of building variants uses a 3D macro-element approach using the OpenSees v2.5.0 analysis program. The modeling concept for a residential house with cripple walls is illustrated in Figure 8.2.

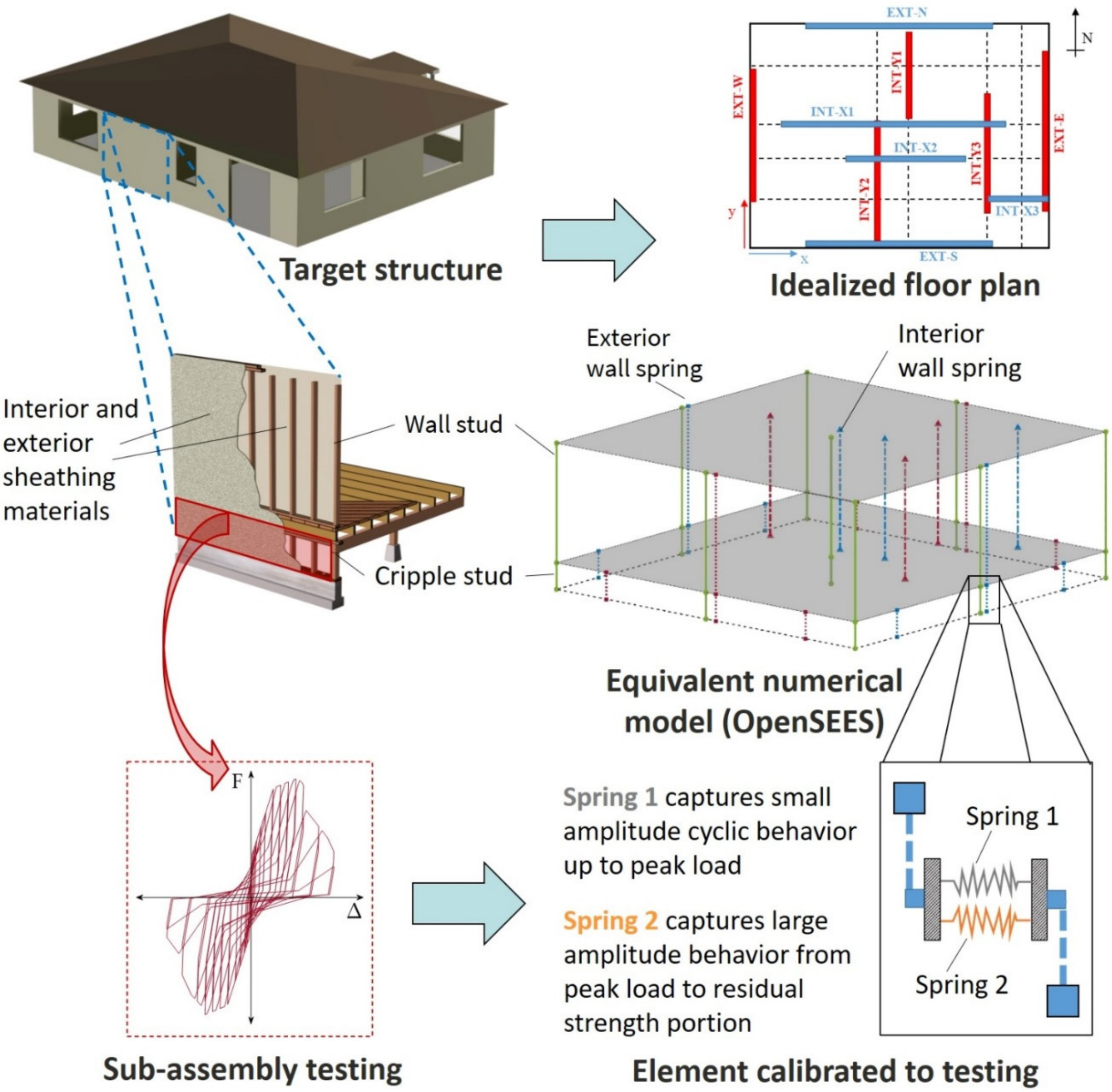


Figure 8.2 **Illustration of the three-dimensional modeling concept used to represent building variants.**

8.3 NONLINEAR STRUCTURAL ANALYSES

Geometrical considerations were included in the model by creating rigid diaphragms for the floor and roof levels that are supported by elastic co-rotational truss elements applied using vertical gravity loads to capture second-order (P-delta) effects. The strength and stiffness of the structure are captured by nonlinear shear spring elements that represent the location, material, and effective length of walls or connections located within the structure. The nonlinear wall springs used the

Pinching4 material in OpenSees, with a two-spring in parallel approach to capture the difference in cyclic behavior of materials under small displacements (onset of damage) and large displacements (collapse). Modeling of stem-wall dwellings used the same approach for modeling of the superstructure with the existing stem-wall connection (i.e., floor joist–sill connections) using a combination of nonlinear springs and friction elements to capture the strength and stiffness of toe-nail connections and sliding resistance provided by the weight of the structure, respectively.

Nonlinear dynamic structural analyses were run to determine relevant EDPs, primarily story drift ratios and peak floor accelerations, statistics that are used to calculate structural and nonstructural damage. The collapse fragility of the structure was also estimated. An illustration of how collapse and non-collapse analysis realizations were calculated is shown in Figure 8.3(a). Collapse fragilities were created by fitting a lognormal distribution using the maximum likelihood approach to collapse data at multiple intensities of ground motions. Both the collapse fragilities and EDPs were conditioned on a no-collapse scenario and included an additional modeling uncertainty of 0.35 (β_{mod}) that was combined using SRSS with the calculated ground-motion record-to-record variability (β_{RTR}). The concept of including additional modeling uncertainty within a collapse fragility is shown in Figure 8.3(b).

8.4 DAMAGE AND LOSS MODELS

The development of damage and loss models for individual components (termed component fragility and consequence functions, respectively) is a critical portion of the loss assessment process. An important investigative effort was conducted to review available component damage fragility information for wood-frame construction to determine appropriate adjustments for older crawlspace dwellings. Although all materials used within the analysis scope were reviewed, only two proposed adjustments are mentioned here for brevity; note that both proposed adjustments were supported by observations from recent testing by Working Group 4. One assumption within the Project was the use of a height-dependent relationship to relate the damage state of full-height stucco walls to that of shorter cripple walls in terms of drift ratio; see Figure 8.4(a). Another key adjustment is revised interior finish fragilities that capture the more brittle nature of older plaster on wood lath when compared to modern gypsum wallboard; see Figure 8.4(a).

To gain a better understanding of repair costs, a claims adjuster damage workshop was organized by Working Group 6 (see Section 9, Vail et al. [2020]) to collect feedback from experts with experience in assessing damage and repair costs for wood-frame houses following natural disasters. Damage description packages were developed for three case study buildings that provided photographs, drawings, and textual descriptions of different materials and sub-assemblies at various damage states within a home. Case study buildings were purposefully devised to provide comparisons to available *FEMA P-58* materials (e.g., exterior stucco with gypsum wallboard interior finish) as well as gain much needed information on the repair costs for sheathing materials that are *not* included in the *FEMA P-58* fragility database (e.g., plaster on wood lath). The results of the damage workshop allowed for cross-comparison with existing *FEMA P-58* functions as well as expanding the range of damage and loss functions for older wood-frame dwellings. Further, the assumed building replacement cost of \$200/ft^2, which was based on information assembled for this project, was supported by similar estimates provided by claims adjustors.

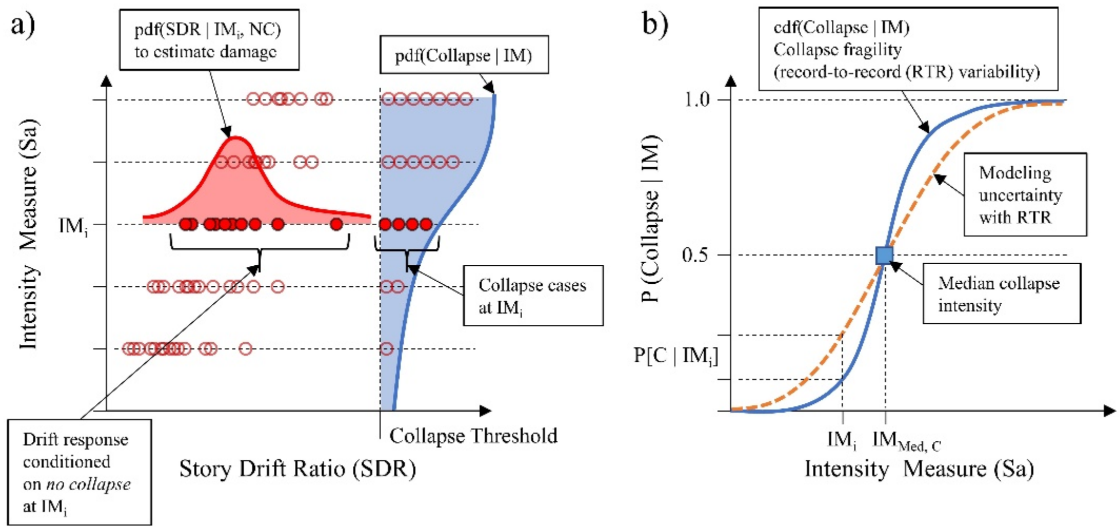


Figure 8.3 **Important concepts for treating structural analysis data: (a) separation of non-collapse and collapse responses for statistics for damage and collapse assessment; and (b) collapse fragility considering record-to-record variability (solid line) and additional modeling uncertainty (dashed line).**

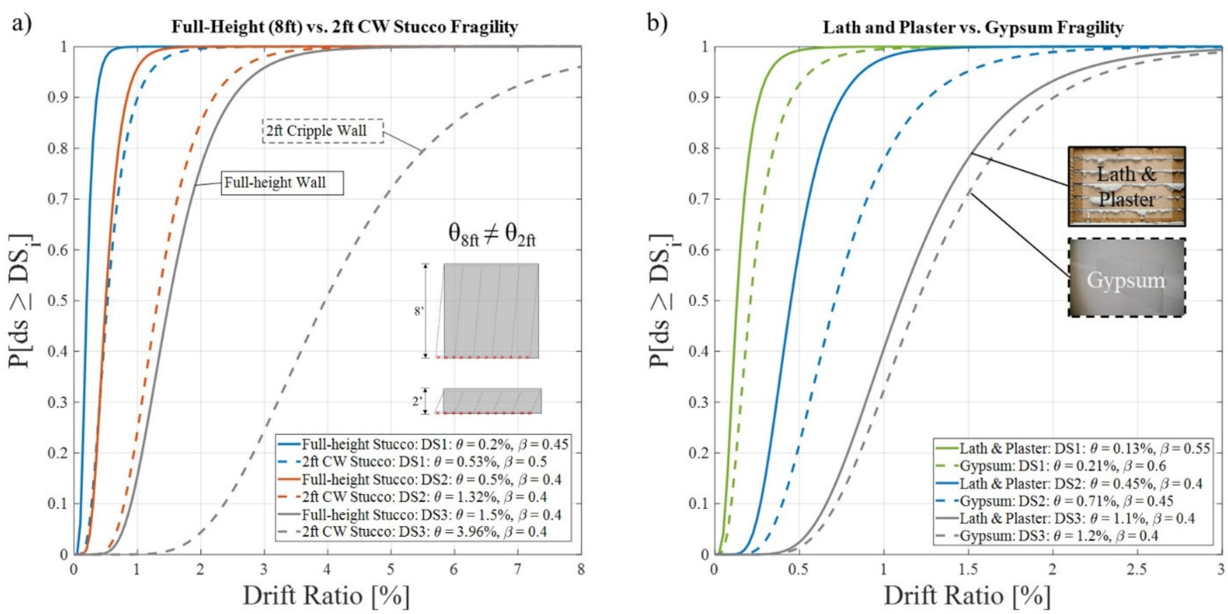


Figure 8.4 **Damage fragility adjustments: (a) height-dependent relationship to capture damageability of shorter stucco walls; and (b) revised lath and plaster fragilities compared to gypsum wallboard.**

An important consideration for the assessment of cripple wall dwellings is the economic treatment of the extent of damage due to cripple wall failure. Supported by reconnaissance reports following earthquakes, the economic consequences due to cripple wall failure can vary widely. In the best-case scenario, the cripple wall fails without significantly damaging the flooring or occupied stories, requiring that the structure be lifted back to position, and the cripple wall or foundation be rebuilt. In other cases, the failing cripple wall causes significant racking damage to interior flooring and finishes, perhaps resulting in a total loss. Based on previous studies [Porter et al. 2002], practitioner surveys [Grossi 1998], and reconnaissance review, the cost of repairing a failed cripple wall is estimated to range from approximately 33% to 100% (total loss) of building replacement cost. The Project has assumed that cripple wall failure incurs 67% of replacement cost, which is judged to be a reasonable expected value by the PEER team and reviewers. The ability to vary this assumption and investigate the sensitivity to other loss ratios is maintained within Project documentation to include improved costing information in future studies.

The performance results for the index house variants are expressed in terms of three outputs:

- Mean loss versus intensity curves: The average loss, expressed in percent of replacement cost, as a function of ground motion shaking intensity, described in terms of 5% damped RotD50 spectral acceleration at 0.25 sec;
- Expected annual loss (EAL): The expected (mean) loss due to the risk of earthquake damage on an annualized basis. This value is obtained through integration of the mean loss versus intensity curve with the site hazard curve that relates ground motion shaking intensity measure to an annual probability of exceedance; and
- Expected (mean) repair cost for earthquake shaking with a return period of 250 years (RC250); this is an intensity-based metric that represents the average loss for earthquake ground shaking with a specified return period. The 250-year return period was selected as a representative point of comparison based on discussion with the catastrophe modeling groups.

An illustration of the use of the primary performance outputs is shown in Figure 8.5. The figure shows two existing and retrofitted pairs of one-story houses with 2-ft-tall cripple walls with wood siding and stucco exteriors. The structures represent the later 1956–1970 construction era with gypsum wallboard interior walls; they are assumed to be located in San Francisco. The loss curves (e.g., ground up repair costs versus intensity) in Figure 8.5(a) show the effect of seismic retrofitting of the cripple wall by comparing the solid (existing) and dashed (retrofit) lines across a range of seismic intensities. The influence of the different exterior materials (stucco versus wood siding) is shown through comparison of the different colors within the figure. In order to express benefits of seismic retrofitting, the key loss metrics are shown as bar charts in Figure 8.5(b) and Figure 8.5(c) for expected annual loss and mean loss at the 250-year return period, respectively. The existing and retrofitted values are overlaid such that the difference between the solid bars and hatched bars illustrate the benefits due to retrofitting for that particular building variant and loss metric. Figure 8.5 shows a much larger expected benefit for the house with wood siding (orange) compared to stucco (green), largely because the cripple wall with wood siding is much weaker than the one with stucco. In both cases, the large reduction in loss is achieved by retrofitting the cripple wall, which greatly reduces the risk of cripple wall collapse. For example, in the wood siding house, retrofitting the cripple wall reduces the expected loss for ground shaking with a 250-year return period from 45% of the house replacement value to just 5%.

Losses for the primary set of cripple wall dwellings (i.e., a 2-ft-tall cripple wall, either one- or two-story) are compared in terms of expected annual loss and mean loss at the 250-year return period in Figure 8.6 and Figure 8.7, respectively. The figures present wood siding and exterior stucco with one- and two-story variants grouped by construction era. The two bounding eras are represented (i.e., pre-1945 and 1956–1970) in the figures, where the cut away for the pre-1945 era signifies that these variants have plaster on wood lath as an interior material, whereas the more modern 1956–1970 era variants have gypsum wallboard interiors. This is an important distinction since the different interior materials affect the mass, strength, stiffness, damageability, and associated repair costs. The groups of colored bars represent the four baseline sites of Bakersfield, San Francisco, Northridge, and San Bernardino where the general seismicity associated with the site increases in that order, with Bakersfield representing a much lower seismicity when compared to the other three.

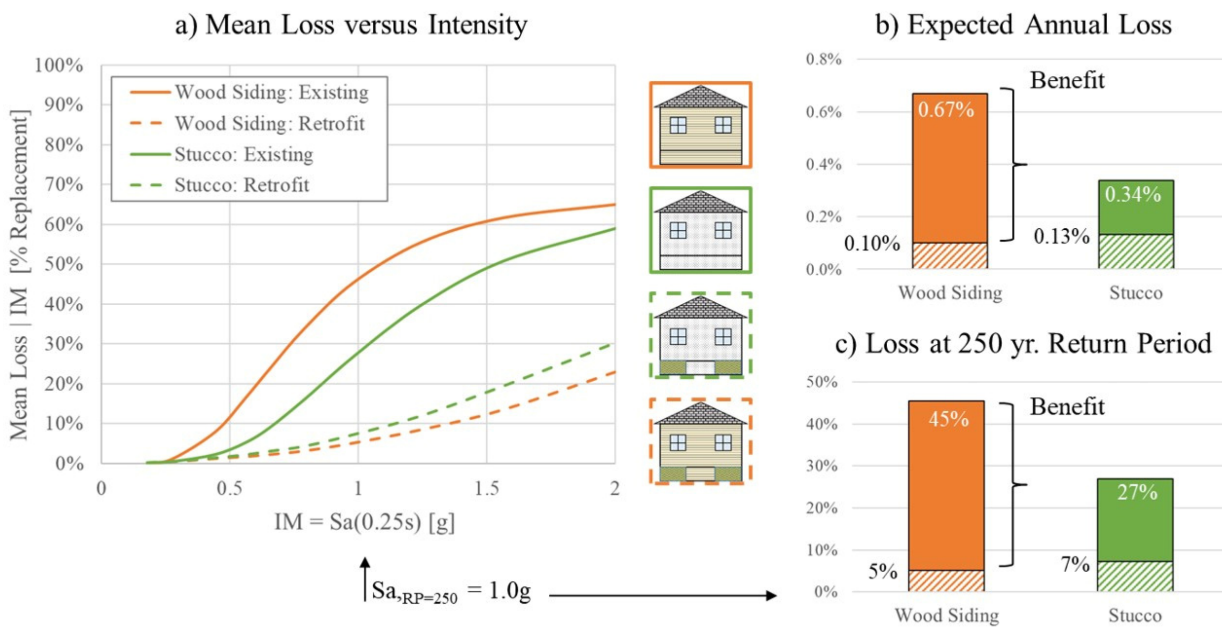


Figure 8.5 **Example of primary performance outputs for 1956–1970 era one-story dwellings with 2-ft-tall cripple walls located in San Francisco showing the effect of seismic retrofit: (a) mean loss versus intensity curves; (b) expected annual loss; and (c) mean loss at the 250-year return period.**

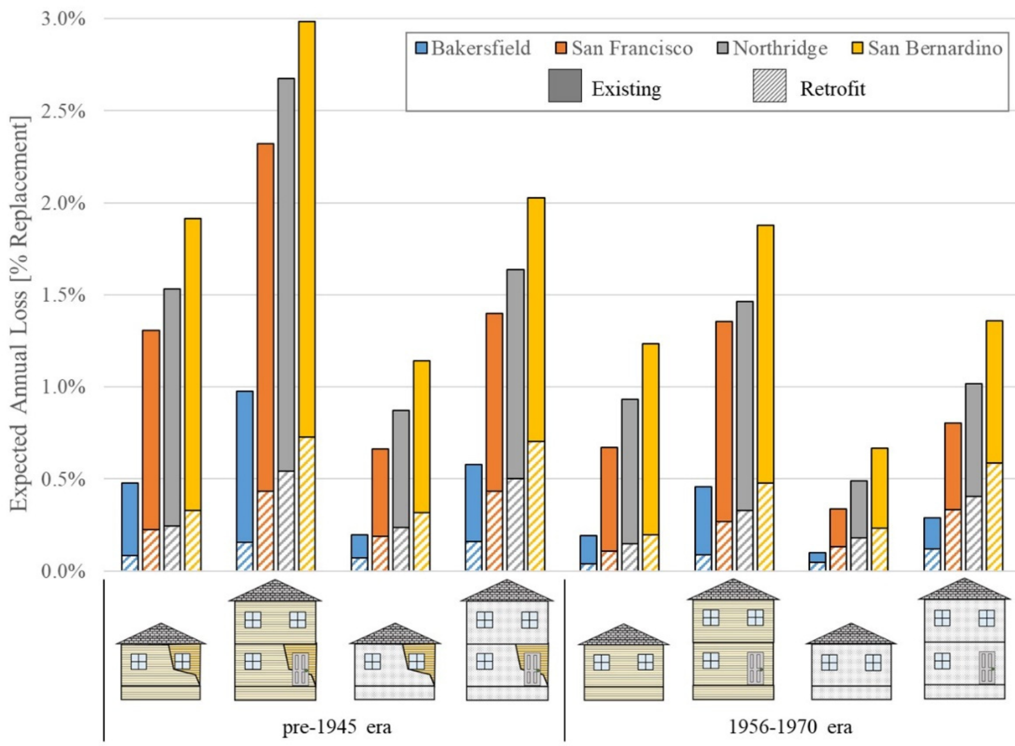


Figure 8.6 Expected annual loss results for houses with 2-ft-tall cripple walls for the pre-1945 and 1956–1970 construction eras.

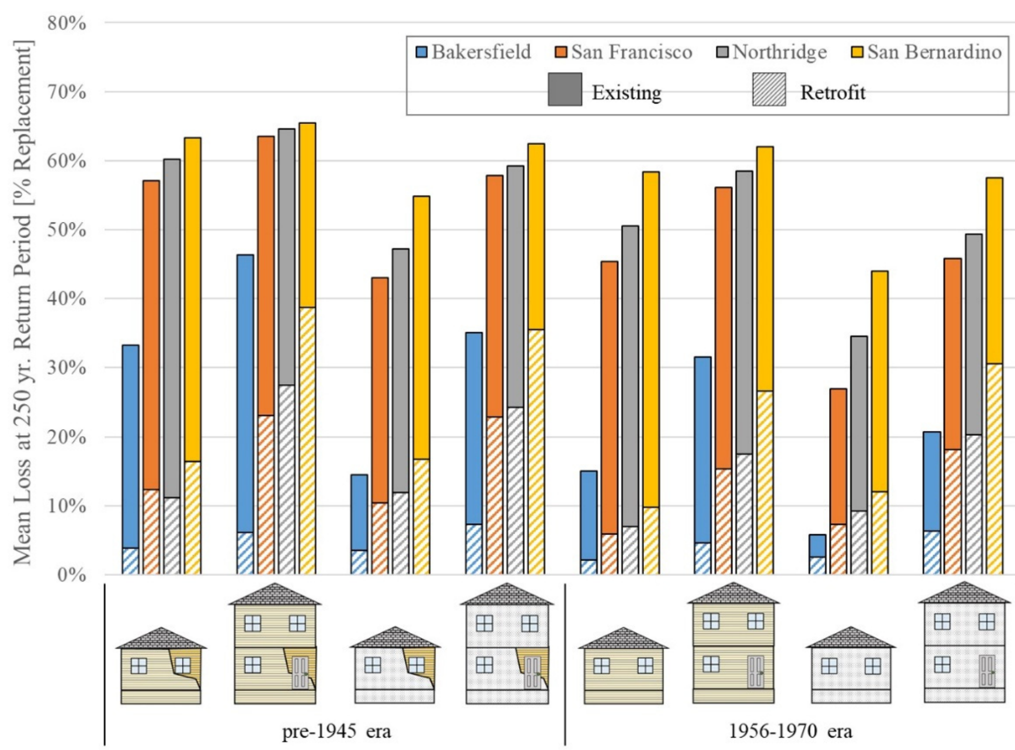


Figure 8.7 Mean loss at the 250-year return period hazard level for houses with 2-ft-tall cripple walls for the pre-1945 and 1956–1970 construction eras.

8.5 GENERAL FINDINGS

The general findings and observations for cripple wall dwellings can be summarized as the following:

- **Influence of Exterior Material:** Wood-siding cripple wall dwellings without retrofit are more susceptible to damage and losses than equivalent stucco exterior cases. This due to the lower strength of wood-siding cripple walls. Accordingly, houses with wood siding generally benefit the most from retrofitting the cripple walls. For houses with retrofitted cripple walls, the damage and losses are comparable for wood siding and stucco houses. In some cases, the retrofitted stucco houses experienced slightly higher losses due to the more brittle nature and more expensive repair costs for stucco as compared to wood siding; however, these slight differences are much less than the overall reduction in losses achieved by retrofitting houses with either type of finish;
- **One-Story versus Two-Story Houses:** As expected, the two-story houses perform poorly compared to the one-story houses. The increased weight of the second story effectively doubles the mass of the house and the imposed earthquake forces on the cripple walls. For the existing (non-retrofitted) cripple wall cases, the two-story houses begin to experience cripple wall damage and losses at much lower seismic intensities (i.e., accelerations) compared to equivalent one-story houses. The two-story houses with retrofitted cripple walls also experience higher losses as compared to one-story cases, although the differences between the two vary more depending on the wall materials and level of seismicity. Since the retrofitted cripple wall design considers differences in building weight, the retrofitted cripple walls are much stronger for the two-story home compared to one-story configurations. This stronger retrofit allows higher forces to be developed in the first occupied story of the superstructure, with the net effect being that displacements and damage shift from the cripple wall into the first story of the superstructure; however, the damage in the first story of the retrofitted houses initiates at much higher seismic intensity compared to damage and collapse in the cripple walls of non-retrofitted houses;
- **Influence of Interior Material:** Older variants with plaster on wood lath interior representing the pre-1945 era generally experience more damage and losses than 1956–1970 era dwellings with modern gypsum wallboard interiors. While plaster on wood lath interior is stronger and stiffer than gypsum wallboard, it is significantly heavier, more easily damaged, and more expensive to repair than gypsum wallboard. The increased mass of houses with plaster and wood lath leads to larger seismic forces in the cripple walls. Similar to the effect in houses with a second story, the larger forces lead to cripple wall damage and collapse at lower ground-motion intensities for non-retrofitted cripple walls. The differences are reduced for retrofitted houses since the design of the cripple walls considers the larger mass and seismic forces associated with the heavier plaster interior walls. Thus, the increase in damage and losses for wood lath and plaster compared to gypsum wallboard is more significant for existing cripple wall cases as compared to the retrofitted cases;

- **Site Seismicity:** In general, the overall losses and the benefits of cripple wall retrofit increase with increased seismicity. In particular, the overall losses and the benefit of the retrofit are much higher in the San Francisco, Northridge, and San Bernardino sites compared to the Bakersfield site. But, even in Bakersfield, the benefits of the cripple wall retrofit are significant, with the exception being for the one-story 1956–1970 stucco house, where the overall losses are insignificant enough that the benefit of retrofit rather low.

The Project also investigated the benefits of anchorage retrofit to older houses with stem-wall foundations. These houses have a crawlspace below the first-floor framing, which is created by a concrete or masonry "stem" wall, where there is a potential vulnerability at the connection between the first-floor framing (i.e., floor joists) to the wooden sill plate attached to the stem wall (i.e., foundation) or an inadequately bolted or unbolted sill plate. Retrofitting of stem-wall dwellings eliminates the vulnerability of stem wall to floor framing connection by installing framing-to-sill clips and foundation anchor bolts (or other anchorage devices) to ensure seismic forces are transferred to the foundation. The main observations for seismic damage and losses related to retrofit of stem-wall connections are summarized as follows:

- **Stem-Wall versus Cripple Wall**: Existing stem-wall dwellings are found to be significantly less vulnerable than equivalent existing cripple walls with the same superstructure. This is due to the assumed existing stem-wall connections (i.e., toenails and friction) being more resistant to failure than unbraced cripple walls resisting the same lateral seismic forces. In general, the consequence of damage to the stem-wall connections is much less than that associated with the high risk of collapse of vulnerable cripple walls. In most cases, damage to the stem-wall connection is limited to small sliding displacements, repairs of which are less complicated and expensive compared to cripple wall damage and collapse. Even in the most extreme cases where the house slides off the stem walls, the repairs are assumed to cost less than the 67% replacement cost assumed for cripple wall collapses based on the expert judgment of PEER team members and project reviewers. Stem-wall repair costs are assumed to be considerably lower than those associated with cripple wall failure since the effects of impact and differential vertical displacements of flooring are not expected to control for stem-wall dwellings; and
- **One-Story versus Two-Story Stem Wall**: Owing to less vulnerability in existing stem walls as compared to cripple walls, the expected benefits for retrofitting of stem walls are significantly less than for retrofitting equivalent houses with cripple walls. The one-story houses with stem walls are observed to show benefits due to retrofitting that range from almost no benefit for the Bakersfield site with lower seismicity to slight benefits for the higher seismicity sites. For example, at the San Francisco site, retrofitting of the stem-wall connection reduced the mean repair cost for the 250-year return period hazard from about 8–14% (of house replacement value) for the non-retrofitted case to 4% to 6% with the retrofit. Results for two-story houses with stem walls show mixed results; in some cases, the stem-wall connection retrofit slightly increases the losses compared to existing stem-wall cases. For example, at the San Francisco site, the losses for the two-story houses at the 250-year return period hazard change from about 15–16% for the unretrofitted cases to 15–23%

for the retrofitted cases. This is explained by the fact that much of the damage and losses (i.e., ground-up repair costs) calculated for the two-story stem-wall houses occur in the first story. In some cases, the non-retrofitted cases experience connection failure and sliding that resulted in a "base isolation" effect for the superstructure, such that the repair costs for the stem-wall connection failure are offset by reduced repairs in the superstructure. Note: the net differences in these cases are small and subject to assumptions made in the analysis models. Should the stem-wall connections be weaker than the expected strength assumed, leading to larger sliding displacements of the unretrofitted cases or should the superstructure be stronger than assumed, then the retrofitted cases would likely show lower losses.

9 Working Group 6a: Interaction with Claims Adjustors

Working Group Leader and Participants: Bret Lizundia, Kylin Vail, David P. Welch, and Evan Reis

Introduction

Working Group 6a focused on a damage workshop effort undertaken to provide repair estimates of representative damaged single-family wood-frame case study buildings to compare the differences in costs between houses with and without retrofits to cripple walls and sill anchorage. At the request of the CEA, 11 experienced claims adjustors from insurance companies volunteered to provide the estimates. Electronic cost estimation files for each case study building were developed by the Project Team using the Verisk Xactware Xactimate X1 platform and provided to the claims adjustors to complete their estimates. The Xactimate platform is commonly used in the insurance industry. These adjustor estimates served as the baseline for comparison against the *FEMA P-58* [FEMA 2018(b)] methodology used on the project for loss estimation. Note: the terms "existing" and "unretrofitted" are used interchangeably in this report.

Three building types were investigated, each with an unretrofitted and a retrofitted condition; an example is shown in Figure 9.1. These were then assessed at four levels of damage, resulting in a total of 24 potential scenarios. Because of similarities, only 14 scenarios needed unique Xactimate estimates. Each scenario was typically estimated by three to five adjustors, resulting in a final total of 74 different estimates.

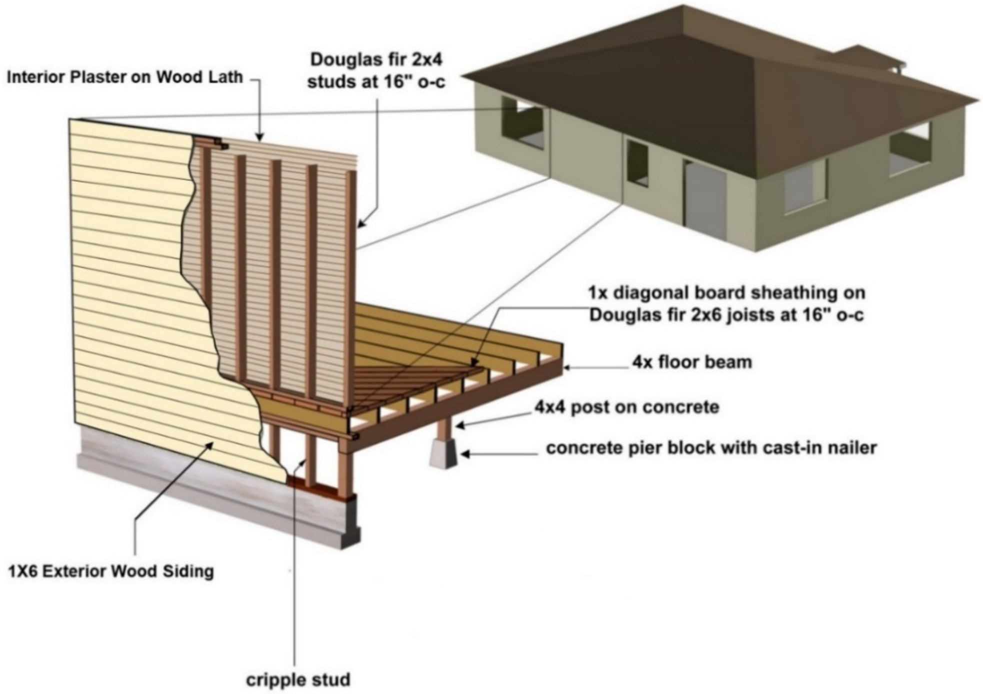


Figure 9.1 Typical framing and finish components for Case Study Building 2 (image adapted from CUREE [2010]).

9.1 SCOPE

The scope of the damage workshop effort included the following tasks:

- A preliminary document, entitled *Earthquake Damage Workshop: Damage Description Package*, was developed by Working Group 6 members. The *Damage Description Package* provided a set of case study examples of damaged buildings to be estimated and a detailed list of estimating assumptions. It also included a list of survey questions to be answered by estimators;
- At the request of the CEA, experienced claims adjustors from insurance companies volunteered to provide estimates of the case study buildings;
- The *Damage Description Package* was provided to all adjustors for their review;
- A conference call was held to introduce the Project's overall goals and to answer any questions from adjustors prior to completing their initial round of estimates;
- Each adjustor prepared estimates on a subset of the total set of case studies. The Verisk Xactware Xactimate X1 platform was used, which is commonly used in the insurance industry. Note that PEER, the CEA, and the participants of this task provide no endorsement of this platform;
- The adjustor estimates were received and reviewed for areas of improvement, and for issues to be discussed as a group;

- An online workshop was held to discuss issues identified in the first round of estimates and answers to survey questions. Standards of practice used by claims adjustors were discussed. Consensus was reached on additional refinements in the estimating assumptions in the final *Damage Description Package*. For ease of use to readers, revisions were made in a track change format. Survey questions were updated;
- Adjustors revised their estimates based on the changes agreed upon in the online workshop and submitted their final estimates and answers to survey questions for review; and
- Review of the estimates indicated that not all instructions in the *Damage Description Package* were followed. For improved consistency, the final estimates were filtered and organized for data comparison.

Adjustors provided the full Xactimate estimate results that include the details on how the estimate was done on a line-by-line level. The collective body of the estimates runs to several thousand pages. By prior agreement, estimating information has been kept confidential, and only results are shown. Similarly, the survey respondent answers have been kept anonymous. Figure 9.2 shows one of the summary tables containing adjustor provided Xactimate cost estimates. Figure 9.3 compares the claims adjustor and *FEMA P-58* results for two of the cases considered.

Adjustor #	Code	Total RCV	Cripple Wall and Foundation	Exterior Damage	Interior Damage	Windows and Doors	Ceilings	Floors	Roof	Miscellaneous	Chimney	Stairs and Porch	MEP	Indirect Cost	RCV Mean	RCV Standard Deviation	Total RCV ($/SF)	RCV Mean ($/SF)
1	CS1RDS3	$138,579	$10,296	$12,521	$23,382	$4,088	$6,517	$4,411	$971	$13,374	$23,868	$2,512	$2,325	$34,314	$136,346	$5,012	$115	$114
3	CS1RDS3	$130,606	$5,535	$11,525	$40,007	$5,316	$0	$0	$1,634	$7,689	$22,151	$2,583	$4,190	$29,976			$109	
5	CS1RDS3	$139,853	$35,327	$20,253	$51,658	$4,435	$0	$0	$2,577	$6,863	$11,346	$2,867	$4,529	$0			$117	
1	CS1UNDS1	$13,824	$1,221	$3,183	$1,830	$0	$0	$0	$0	$5,805	$0	$0	$0	$1,784	$17,774	$6,469	$12	$15
2	CS1UNDS1	$9,835	$3,307	$1,606	$3,215	$0	$0	$0	$0	$0	$0	$0	$0	$1,707			$8	
3	CS1UNDS1	$16,477	$1,117	$3,234	$3,715	$0	$0	$0	$0	$6,025	$0	$0	$0	$2,386			$14	
4	CS1UNDS1	$24,630	$618	$3,511	$2,785	$0	$0	$0	$0	$14,256	$0	$0	$0	$3,460			$21	
5	CS1UNDS1	$24,105	$2,463	$7,387	$13,127	$0	$0	$0	$0	$1,128	$0	$0	$0	$0			$20	
1	CS1UNDS2	$29,402	$1,464	$3,895	$2,064	$0	$624	$0	$0	$7,367	$9,192	$0	$0	$4,796	$41,262	$12,834	$25	$34
2	CS1UNDS2	$40,471	$2,609	$6,804	$2,851	$0	$430	$0	$0	$813	$21,564	$0	$0	$5,401			$34	
3	CS1UNDS2	$30,541	$3,101	$5,310	$5,264	$0	$833	$0	$0	$5,956	$1,424	$0	$0	$8,654			$25	
4	CS1UNDS2	$60,994	$14,439	$5,529	$4,104	$0	$0	$0	$0	$14,201	$11,647	$0	$0	$11,073			$51	
5	CS1UNDS2	$44,901	$3,044	$8,931	$20,249	$0	$0	$0	$0	$1,331	$11,346	$0	$0	$0			$37	
1	CS1UNDS3	$110,089	$7,414	$10,117	$16,954	$3,385	$6,725	$0	$971	$5,847	$23,868	$2,512	$2,325	$29,970	$127,344	$30,330	$92	$106
2	CS1UNDS3	$106,808	$38,442	$16,257	$19,814	$0	$0	$0	$1,224	$8,441	$0	$0	$0	$22,630			$89	
3	CS1UNDS3	$110,230	$5,745	$11,564	$19,360	$5,316	$2,770	$0	$1,633	$7,689	$22,151	$2,583	$4,190	$27,229			$92	
4	CS1UNDS3	$178,879	$68,396	$14,057	$22,855	$4,982	$0	$0	$1,391	$14,413	$10,652	$0	$0	$42,133			$149	
5	CS1UNDS3	$130,715	$26,189	$20,253	$51,658	$4,435	$0	$0	$2,577	$6,863	$11,346	$2,867	$4,529	$0			$109	
1	CS1UNDS4	$292,800													$285,189	$96,500	$244	$238
2	CS1UNDS4	$235,936															$197	
3	CS1UNDS4	$220,651															$184	
4	CS1UNDS4	$450,000															$375	
5	CS1UNDS4	$226,558															$189	

Figure 9.2 Adjustor estimate data for Case Study Building 1.

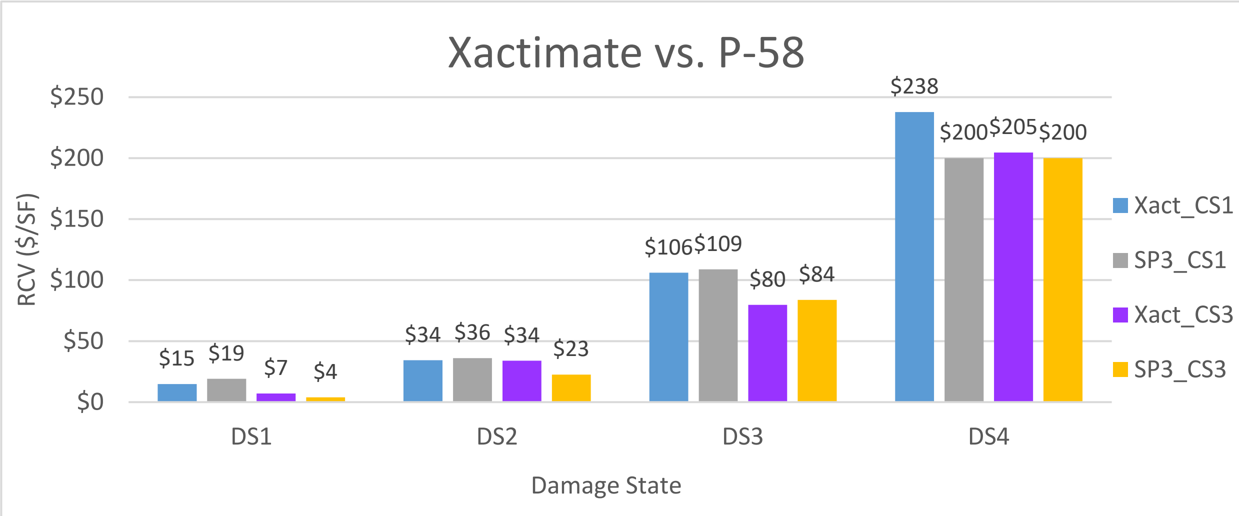


Figure 9.3 **Claims adjustor and *FEMA P-58* comparison for unretrofitted Case Study Building 1 and Case Study Building 3.**

9.2 AUDIENCE

The results from the workshop effort were used by the PEER–CEA Project Team to refine the project loss estimates. Results and conclusions will also be of value to the following stake holders: (a) the CEA; (b) catastrophe modelers for insurers and insurance companies; (c) practicing claims adjusters; (d) design team professionals involved in damage assessment and repair; (e) estimators and design professionals using the *FEMA P-58* methodology; and (f) researchers involved in earthquake loss estimation.

9.3 TYPES OF ESTIMATES

There are at least four distinct types of cost estimates relevant to the damage workshop effort and the PEER–CEA Project in general.

1. Cost estimates used by design teams.
2. Claims adjustor estimates.
3. *FEMA P-58/SP3* loss estimates.
4. Catastrophe modeler loss (damage) functions.

A significant discovery from the damage workshop effort is that linking these four types of estimates and their associated groups is not straightforward. They have different types of professional (e.g., industry vs. academia), perspectives, procedures, and products. Terminology is not consistent and assumptions vary. At times, it is as if they speak a different language.

A more detailed description of the differences highlighted in these four kinds of estimates is provided below:

1. *Design team cost estimates:* These are cost estimates for projects in design. The estimate is a prediction of what the owner will eventually pay. Drawings and

specifications are produced in various stages, beginning with the concept level, and then moving on to schematic design, design development, construction documents (or "working drawings"), and bid documents. Assumptions must be made at each stage about what is not yet shown, and factors such as escalation to the midpoint of construction need to be included, as do contingencies. Direct costs or "hard costs" paid to the general contractor are often termed "above the line." The estimates include a variety of indirect "below the line" or "soft costs" to cover other items, such as contingencies for changes or unknown conditions, utilities, design fees, plan check and permitting costs, and abatement costs. There are usually exclusions based on owner direction or standard estimating practice, but they can represent real costs that need to be accounted for in order to have a full picture of what the owner will ultimately pay, such as temporary moving fees, financing costs, and legal fees.

2. *Claims adjustor estimates:* For these estimates, the building exists, damage has already occurred, and it can be reviewed in detail. There is damage that is visually observable, but sometimes there can be additional structural damage under finishes that requires investigation or assumptions need to be made. Adjustor estimates usually include the full or "ground up" cost that would be required to repair the damage. The actual payout to the owner is typically determined separately by the underwriter based on policy conditions, including deductibles, caps and exclusions. Distinctions are made between salvage (the item can be reused as is), repair (the damaged item is repaired to its "pre-earthquake state using similar nonstructural finishes, materials, and approaches), and replacement (the damage element is replaced or rebuilt with new materials). A distinction is typically made between replacement cost value (the cost to replace the damaged items with a new version) and actual cash value (where depreciation reduces the value of the item to reflect its age and market value). Typically, adjustors do not include a factor for inflation as is done in design team cost estimates. Instead, the adjustor's estimate is updated as additional information is collected or owners incur costs, and multiple updates are common on heavily damaged buildings. Sometimes engineers and other design professionals assist in determining the scope of repairs and thus the overall cost.

   Insurance companies have internal guidance on the approaches and assumptions to be used by their adjustors, in addition to following the terms of the insurance policy. *EDA-02, General Guidelines for the Assessment and Repair of Earthquake Damage in Residential Woodframe Buildings* [CUREE 2010] has provided helpful guidance on assessing earthquake damage and determining the extent of repair that is needed. In addition to funding development of the CUREE Guidelines, the CEA has recently funded an Applied Technology Council project to update the General Guidelines and add Engineering Guidelines. The CEA recommends that the resulting reports, CEA-EDA-01 [CEA 2020(a)] and CEA-EDA-02 [CEA 2020(b)], be used by participating insurers, and their consultants for the assessment and repair of earthquake damage.

3. Adjustors typically develop detailed estimates and software has been developed to automate and streamline the process. For this PEER–CEA Project, the Xactimate platform was used. The cost categories and features of the Xactimate platform can influence the techniques and assumptions used by the adjustor and how the estimate is presented.

4. *FEMA P-58/SP3 loss estimates:* Loss estimation techniques have been developed to predict damage to buildings given different levels of earthquake shaking. Studies can be for an individual building or on a regional scale where results from all the buildings in the region are aggregated. Standardized archetypes are used to represent the building. Fragility curves relate shaking intensity to damage states for the archetypes. Fragility curves can be for damage to the whole building or for damage specific to individual components within the building. Repair and replacement costs are related to damage states. Ground-motion data and shaking intensity at the building site are then combined with the fragility curves and cost data to produce estimates of damage and repair costs.
5. Common programs and methods for loss estimation include HAZUS [FEMA 2014], an open-source software program developed by the Federal Emergency Management Agency, FEMA P-58, or SP3 (a commercial program that helps implement the FEMA P-58 methodology). *FEMA P-58* was used for this PEER–CEA Project by other working groups, with some modifications. *FEMA P-58* has component-based fragility curves and cost data, unlike HAZUS (which has general fragility curves for the building as a whole). Worth noting is that *FEMA P-58* is primarily focused on post-1950 engineered buildings and not the single-family wood-frame houses that are the subject of the PEER–CEA Project. The component and damage categories available in *FEMA P-58* are much more limited than the detail that is available in Xactimate. On the other hand, methods like *FEMA P-58* are capable of conducting many analyses runs to help understand parameter sensitivity and dispersion.
6. *Insurance catastrophe modeler DFs:* Catastrophe loss modelers use functions that relate ground-shaking measures (such as intensity or spectral acceleration) to damage. These functions are based on past claims and modeler judgment. The past is used to predict the future. Damage functions are proprietary and specific to each modeling company. The term "damage" is used by insurers to represent the overall cost of repair, whereas "loss" is a term insurers use to represent how much the insurer loses through the payout; thus, implicitly it includes the impact of deductibles and other policy limits. A distinction often made between fragility functions (which relate shaking with a physical description of building or component damage) and DFs (which relate shaking and the dollar value of damage). The ultimate goal of the PEER–CEA Project is to provide modelers with improved DFs for older existing and retrofit houses with cripple walls and stem walls. Typically, even though insurers and modelers may have a large quantity of claims that represent real payouts to owners, the details of the damage are not typically known by the modeler. Damage functions can vary according to building attributes, but they typically relate to the building as a whole. Thus, it is difficult to compare costs at the component level with other types of estimates, such as those noted above where component level information is available.

9.4 CONCLUSIONS

9.4.1 Speaking the Same Language

The damage workshop effort, including the survey question responses and the discussion at the online damage workshop, reinforced the observation that cost estimates used by design teams, claims adjustor estimates, damage estimates such as those obtained from HAZUS or *FEMA P-58*, and insurance catastrophe modeler estimates are all done by different types of professionals with different perspectives using different processes to produce different products. When these different disciplines come together, if representatives from one discipline do not fully understand the assumptions made by the other disciplines, they will not have consistent results or a clear understanding of all facets of the process. The difference in the adjustor estimates before and after the workshop is evidence of this. Revisions in estimating instructions led to meaningful changes in the results. It was also a surprise that some basic assumptions used by some disciplines, such as the use of escalation to adjust estimates to the midpoint of construction, are not part of the practice or even terms used by other disciplines. It is important to try to speak the same language to better understand one another's work. The damage workshop conversations helped to shed light on this need.

9.4.2 Detailed Estimating Assumptions Are Necessary

Working Group 6 members included practitioners with substantial experience in reviewing cost estimates for design projects and in regional loss estimation. It was well understood before the damage workshop effort that detailed estimating assumptions would be required to define what to include and what not to include in the estimates. Significant review was made of the assumptions prior to the workshop by project advisors, experts in claims adjusting and the use of Xactimate, and insurance catastrophe modelers. Despite this effort, the process still revealed a significant number of refinements and instructions that were needed to achieve improved clarity and consistency. Some examples included when to replace building paper under damaged stucco, the extent of repairs to apply for different levels of damaged and racked cripple walls, and how to handle contingencies, utility costs, and additional living expenses during repair work. Loss estimate studies should be viewed with an eye to this issue. Do they define their terms? Do they list the estimating assumptions there were made? Are the assumptions realistic?

9.4.3 Estimate Results from Adjustors Are Similar to Results using the *FEMA P-58* Methodology

Even though the methods and tools used by claims adjustors and *FEMA P-58* are different, and even though there are cost categories missing in *FEMA P-58* that are used by claims adjustors, the bottom-line results at both the building level and at key component levels like the cripple wall and foundation were similar for both methods. This was something of a surprise and required careful examination. As a result, the Project Team concluded that general adjustments to the *FEMA P-58* results were not needed beyond adjustments made following careful review of existing *FEMA P-58* functions prior to workshop assessments.

9.4.4 Some Key Assumptions Must Be Recognized for Meaningful Comparisons

Using the damage workshop results for comparisons with DFs used by insurance catastrophe modelers requires recognizing some key assumptions. These include the following.

- Demand surge caused by increased labor and material pricing after a major earthquake was deliberately not included in the adjustor estimates. This was done because it is understood that catastrophe modelers address demand surge separately from the basic loss functions;

- There are special features that are not common in individual buildings but are represented by a portion of the buildings in the community that will increase costs. Adjustors were directed to exclude them in their estimates for consistency and simplicity. These include buildings with high-end finishes; concrete foundation damage; sidewalk and driveway damage; possible building code upgrades required by local building officials; damage from liquefaction, lateral spreading, and fault rupture; additional special inspection and testing; legal fees; hazardous materials besides lead paint and asbestos, like mold, soil contamination, and radon; premiums for historic buildings; ADA upgrade costs; increased costs if access and utilities at the site are compromised; construction management costs; and financing costs. If these were included, the median would likely rise as would the upper end of the estimated range;
- Adjustors reported that abatement of lead paint and asbestos in California can add substantial cost. This had not been appreciated by members of the Project Team before the workshop. Estimating assumptions were refined as a result. Further study of these costs and the attributes that influence them is needed;
- Costs for repairing brittle finishes like tile can be a substantial portion of the repair cost because it is difficult to match original tile. Generally, this leads many adjustors to recommend full replacement of tile in the room, even if the extent of damage is small. This issue had not been fully appreciated by members of the Project Team before the workshop; and
- Insurance policy rules, including deductibles and caps and depreciation assumptions, can impact the amount paid out. Thus, comparing the cost of the total damage with insurance payouts can be difficult and inconsistent.

9.5 RECOMMENDATIONS

The findings and conclusions from the damage workshop effort led to the following recommendations.

- The project approach using the *FEMA P-58* methodology should continue with only minor refinements needed on some specific individual components but requires addressing items not well covered by *FEMA P-58*, such as lath and plaster repairs;
- Cost estimates for earthquake damage repair need to be done with very clear and very detailed descriptions of the assumptions that were made, and the results need to be viewed in the context and limitations of those assumptions;
- Because of the significant and increasing cost of lead paint and asbestos abatement in earthquake damage repair in California, more in-depth study of this issue is needed to better understand the cost and policy implications;
- The *EDA-02 General Guidelines for the Assessment and Repair of Earthquake Damage in Residential Construction* [CUREE 2010] provide guidance to claims adjustors on common types of earthquake damage that occur in wood-frame residential construction, how to assess the significance of the damage,

and what techniques should be used to repair the damage. The guidelines are a valuable tool used by claims adjustors, but updates are needed, particularly for heavily damaged buildings requiring structural repairs. The CEA funded a project managed by the Applied Technology Council to provide updated general guidelines and engineering guidelines. These documents, *CEA-EDA-01* [ATC 2020(a)] and *CEA-EDA-02* [ATC 2020(b)] have been published. These documents should be promoted within the insurance and design communities to improve understanding and consistency of repair assessment and estimating;

- Greater understanding is needed of the issues that trigger moving from (1) repairing damaged cripple walls to (2) jacking and repairing the wall to (3) jacking and replacing the wall to (4) full building replacement. There was no clear consensus between adjustors on what approach to take for heavily damaged conditions. They typically defer to structural engineering advisors. There is also no clear consensus among structural engineers. More study is needed; and

Insurance claim payouts remain a highly desirable resource for the scientific community to improve its analytical loss estimating research, but proprietary considerations limit the availability of the information. Sharing this valuable information, particularly at the detailed component level as well as detailed inventory data (while finding ways to preserve anonymity and proprietary advantage), would be extremely beneficial to the effort of improving insurance pricing for seismic retrofitting of components such as cripple walls and sill anchorage. For example, insurers could aggregate anonymous claim payout information in the cost estimate categories used in the damage workshop effort. A second step would be to include building characteristics together with the claims payout data, but perhaps stripped of identifiable locations.

10 Working Group 6b: Interaction with Catastrophe Modelers

Working Group Leader and Participants: Evan Reis and David P. Welch

10.1 INTRODUCTION

Working Group 6 focused on interaction with the catastrophe modelers and compared the DFs developed by the PEER–CEA Project with those currently contained in modeling software developed by the three largest insurance catastrophe modeling companies: RMS, CoreLogic, and AIR Worldwide; from this point referred to as the Modelers. A DF is the loss measured as repair costs, without insurance policy considerations (i.e., assuming zero deductible and no policy caps), as a function of shaking intensity at a specific period. A semi-blind study was conducted in collaboration with the Modelers to compare damage estimates for a selection of the Index Buildings developed in the PEER–CEA Project. The Working Group 6 Project Team conducted several meetings with the Modelers to answer questions and present the comparative results described herein. Note: the terms “existing” and “unretrofitted” are used interchangeably in this report.

The PEER–CEA Project Working Group 6 Team formulated a framework for comparing DFs derived from the PEER–CEA Project with those used by the Modelers, in order to develop a comparison of the Index Buildings derived as part of the scope for PEER–CEA Project Working Groups 2 and 5. This comparison was used as a way for the Modelers to evaluate the DFs produced by the Project and determine how best to incorporate the outcomes of the project into their catastrophe models. The PEER–CEA Project Team provided links to the raw data accumulated through the Project for use by the Modelers as they consider incorporating the information into their catastrophe models.

10.2 INDEX BUILDINGS COMPARISON SET

As described in the PEER–CEA Project Working Group 2 report, the total number of Index Buildings considered within the Project was based on the combinations of variables representing building construction characteristics, which numbers in the hundreds. Not all variables considered in the study are included within the primary or secondary modifier options of the Modelers’ damage models. For example, the PEER–CEA Project study considered a range of cripple wall heights, from two feet to six feet, whereas all Modelers designated only a single variable representing the presence of a cripple wall. Another example is that the PEER–CEA Project models considered interior wall finishes consisting of either gypsum wallboard or lath and plaster, the latter being substantially heavier than the former. None of the Modelers' allowed for the selection of interior finish materials.

One hundred and forty-four of the PEER–CEA Project Index Buildings were initially selected as potential candidates for use in comparing the results of the PEER–CEA models vs. the results of the Modelers' efforts. Of these, twelve were selected to be evaluated as an initial test of the models.

The Index Buildings were distinguished by age, number of stories, exterior siding material, and foundation type, and whether the buildings were seismically retrofitted. A raised foundation refers to the condition where the first floor is built on a wood stud cripple wall that sits on the concrete footing. The consensus of the Project Team was that a 2-ft tall cripple wall was the most common height of a cripple wall, with 4-ft-tall and especially 6-ft-tall cripple walls being much less common. A stem-wall foundation refers to the condition where the first-floor joists rest directly on the sill plate, which sets on the footing.

One of the variables initially considered was the condition of the building materials, which would adjust the analysis model parameters to reflect the quality of the structural materials (to account for deterioration, quality of construction, etc.). The Project Team recognized that the quality of construction and the condition of structures in California varies significantly as a function of age, location, climate, quality of construction, and other factors. The base shear capacity and damage states of buildings are dependent on these factors. It is important to note that the benefit of seismic retrofit will vary depending on the condition of the existing structures. Therefore, the condition of the building to be modeled is key in meeting the criteria laid out by the project scope of work for this Project.

The Project Team held a meeting in December 2019 with its Project Review Panel (PRP) to discuss this issue and several others affecting the models. The Team and PRP concluded that no scientific data was available that considers the impact of material condition on the characteristics associated with the performance of wood-frame single-family homes. Furthermore, the testing program implemented as part of the testing effort conducted by PEER–CEA Project Working Group 4, while incorporating construction detailing and methods appropriate for the eras of construction, it could not realistically embed "deteriorated" or otherwise poor condition quality into the tests. For example, the testing project did not artificially rust nails or crack stucco finishes in an attempt to represent the potential effects of aging.

The Project Team and PRP determined that the only way to include the effects of material conditions would have been to use expert judgment to represent an estimate of poor or good condition by increasing or decreasing the strength-stiffness and hysteretic behavior of the model components. Ultimately the group decided that this would only introduce the variable "expert judgment" into the overall DFs, which could not be objectively or numerically justified when comparing the PEER–CEA Project results to those of the Modelers.

Another point of consideration was that within the results provided by the Modelers there was very little difference in damage based on the condition variable available within their models; see Table 10.1. The replacement cost value (RCV) of the house represents costs adjusted for depreciation of the building as opposed to actual cash value. Thus, the Project Team notes that the performance of single-family homes may exhibit better or worse behavior dependent on *in situ* conditions, and that insurance pricing should reflect this uncertainty in an actuarially appropriate manner.

Table 10.1 Variation in Modelers' loss results as a function of condition modifier.

Average results of all nodelers for comparison set				
	AAL (% of RCV)			
Condition	**San Francisco**	**San Bernardino**	**Northridge**	**Bakersfield**
Good	0.19%	0.35%	0.29%	0.07%
Average	0.21%	0.37%	0.30%	0.07%
Poor	0.21%	0.39%	0.32%	0.07%
	At 250-year return period (% of RCV)			
Condition	**San Francisco**	**San Bernardino**	**Northridge**	**Bakersfield**
Good	14.6%	26.2%	19.9%	4.5%
Average	15.4%	27.8%	20.8%	4.8%
Poor	16.2%	29.0%	22.0%	5.0%

Based on these factors, the Project Team decided to remove "condition" as a variable in the PEER–CEA Project models. A single "best estimate" of material properties based on the available science and testing was used. In January of 2020, this decision was discussed during the presentation of results to the Modelers; the Modelers' universal consensus was that this was a rational and defensible decision.

The result of this decision was that by eliminating the "Good" and "Poor" condition variables, the original set of 144 Index Buildings to be compared in the calibration study was reduced to 48. Table 10.2 below lists the 48 Index Buildings that were ultimately considered in the calibration study. The PEER–CEA Project models assumed the following for each of the Index Buildings.

- All buildings were assumed to have the same plan layout;
- Wood siding refers to a cripple wall with horizontal sheathing boards and diagonal wood bracing. The consensus of the Project Team and the PRP was that diagonal bracing was a common practice in older construction used to provide basic stability of the house prior to the placement of the exterior sheathing;
- A raised foundation refers to a 2-ft-tall cripple wall;
- A stem-wall foundation refers to the condition where the first-floor joists rest directly on the sill plate;
- The retrofitted condition refers to seismic mitigation that meets the ATC-110 plan set, given the building's location, number of stories, siding, and interior finish materials;
- Index Buildings in the <=1945 age category were assumed to contain lath and plaster interior wall finishes;
- Index Buildings in the 1956–1970 age category were assumed to contain gypsum wallboard interior wall finishes; and

- Damage estimates for Index Buildings in the 1945–1955 age category were assumed to be the arithmetic average between the <=1945 and 1956–1970 age categories, representing an assumed equal distribution between buildings with lath and plaster and gypsum wallboard finishes.

Table 10.2 Forty-eight Index Buildings used in Modeler comparison study.

Index number	Height	Age	Siding	Foundation	Condition	Retrofitted
3	1 story	<=1945	Wood	Raised	Best Estimate	Yes
4	1 story	<=1945	Wood	Raised	Best Estimate	No
9	1 story	<=1945	Wood	Stem Wall	Best Estimate	Yes
10	1 story	<=1945	Wood	Stem Wall	Best Estimate	No
15	1 story	<=1945	Stucco	Raised	Best Estimate	Yes
16	1 story	<=1945	Stucco	Raised	Best Estimate	No
21	1 story	<=1945	Stucco	Stem Wall	Best Estimate	Yes
22	1 story	<=1945	Stucco	Stem Wall	Best Estimate	No
27	1 story	1945-1955	Wood	Raised	Best Estimate	Yes
28	1 story	1945-1955	Wood	Raised	Best Estimate	No
33	1 story	1945-1955	Wood	Stem Wall	Best Estimate	Yes
34	1 story	1945-1955	Wood	Stem Wall	Best Estimate	No
39	1 story	1945-1955	Stucco	Raised	Best Estimate	Yes
40	1 story	1945-1955	Stucco	Raised	Best Estimate	No
45	1 story	1945-1955	Stucco	Stem Wall	Best Estimate	Yes
46	1 story	1945-1955	Stucco	Stem Wall	Best Estimate	No
51	1 story	1956-1970	Wood	Raised	Best Estimate	Yes
52	1 story	1956-1970	Wood	Raised	Best Estimate	No
57	1 story	1956-1970	Wood	Stem Wall	Best Estimate	Yes
58	1 story	1956-1970	Wood	Stem Wall	Best Estimate	No
63	1 story	1956-1970	Stucco	Raised	Best Estimate	Yes
64	1 story	1956-1970	Stucco	Raised	Best Estimate	No
69	1 story	1956-1970	Stucco	Stem Wall	Best Estimate	Yes
70	1 story	1956-1970	Stucco	Stem Wall	Best Estimate	No
75	2 story	<=1945	Wood	Raised	Best Estimate	Yes
76	2 story	<=1945	Wood	Raised	Best Estimate	No
81	2 story	<=1945	Wood	Stem Wall	Best Estimate	Yes
82	2 story	<=1945	Wood	Stem Wall	Best Estimate	No
87	2 story	<=1945	Stucco	Raised	Best Estimate	Yes
88	2 story	<=1945	Stucco	Raised	Best Estimate	No
93	2 story	<=1945	Stucco	Stem Wall	Best Estimate	Yes
94	2 story	<=1945	Stucco	Stem Wall	Best Estimate	No
99	2 story	1945-1955	Wood	Raised	Best Estimate	Yes
100	2 story	1945-1955	Wood	Raised	Best Estimate	No
105	2 story	1945-1955	Wood	Stem Wall	Best Estimate	Yes
106	2 story	1945-1955	Wood	Stem Wall	Best Estimate	No
111	2 story	1945-1955	Stucco	Raised	Best Estimate	Yes
112	2 story	1945-1955	Stucco	Raised	Best Estimate	No
117	2 story	1945-1955	Stucco	Stem Wall	Best Estimate	Yes
118	2 story	1945-1955	Stucco	Stem Wall	Best Estimate	No
123	2 story	1956-1970	Wood	Raised	Best Estimate	Yes
124	2 story	1956-1970	Wood	Raised	Best Estimate	No
129	2 story	1956-1970	Wood	Stem Wall	Best Estimate	Yes
130	2 story	1956-1970	Wood	Stem Wall	Best Estimate	No
135	2 story	1956-1970	Stucco	Raised	Best Estimate	Yes
136	2 story	1956-1970	Stucco	Raised	Best Estimate	No
141	2 story	1956-1970	Stucco	Stem Wall	Best Estimate	Yes
142	2 story	1956-1970	Stucco	Stem Wall	Best Estimate	No

10.3 SEISMIC HAZARD CONSIDERATIONS

To make as direct a comparison of DFs as possible within the limitations of the present study, hazard should be a control variable. The Project Team selected four sites for the study to be used by the Project Team and all the Modelers, each assumed to be of Site Class D with V_{s30} = 270 m/sec. Basin and near-field effects were not included. Despite the goal of hazard being a control variable, the difference in hazard curve ordinates between the Project Team and the Modelers were on the order of +/- 10% to +/- 30%, depending on the range of return periods compared. Figure 10.1 shows the hazard curves of the PEER–CEA Project and Modelers for the Northridge location. The differences between the Modelers and the PEER–CEA Project results are, therefore, a result both of the differences in the hazard curves and differences in the underlying DFs themselves. To eliminate the differences associated with hazard, the Modelers would need to match the PEER–CEA Project hazard curves exactly across all return periods, or the Modelers would need to provide the Project Team with their DFs for the Index Buildings directly.

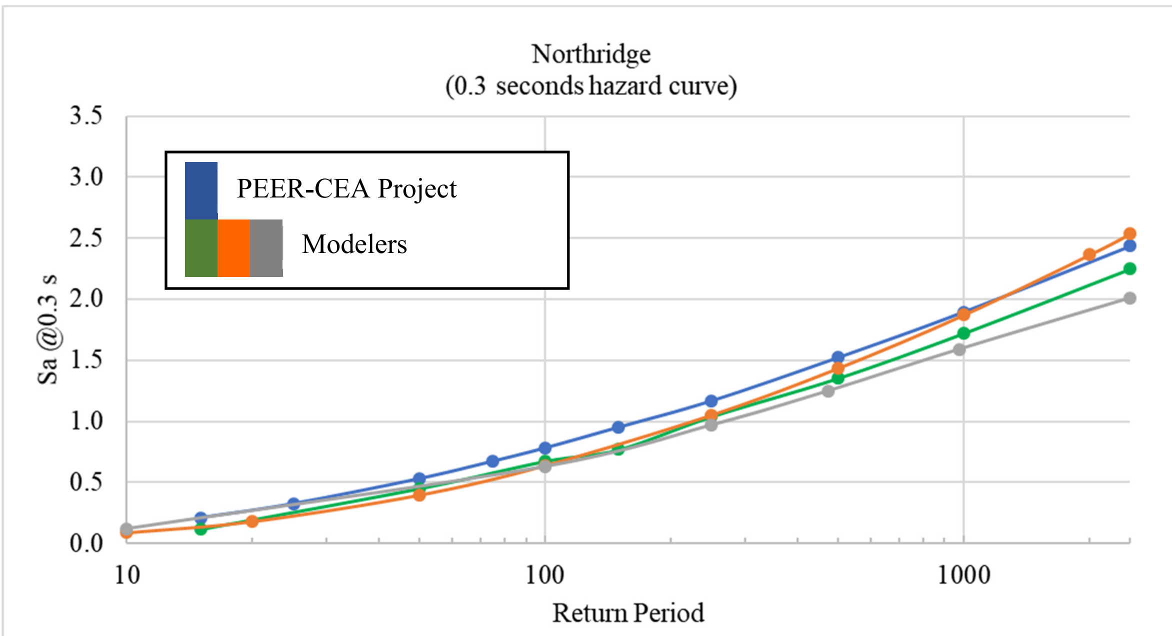


Figure 10.1 Hazard curves by location at *Sa* = 0.3 sec.

10.4 RESULTS AND FINDINGS

The Project Team compared two result values against those provided by the Modelers: damage (i.e., repair costs) for an earthquake with a 250-year return period and average annual loss (AAL). The damage at a 250-year return period represents a single point along the DF where the spectral acceleration value at a 1.0-sec period is equal to that obtained in the hazard curve at an annual recurrence frequency of 1/250 = 0.004. As described above, because the PEER–CEA Project and Modeler hazard curves have not been normalized to match, the comparison of damage at the 250-year return period is not a direct comparison of damage at the same value of spectral acceleration. The AAL is a value obtained by integrating the DF over the annual frequency of each value of spectral acceleration. Examples of the comparative results for San Francisco are shown in Figure

10.2 through Figure 10.5. Figure 10.6 and Figure 10.7 compare the unretrofitted condition for the 2-ft-tall cripple wall averaged across all four site locations. Figure 10.8 and Figure 10.9 compare the unretrofitted with the retrofitted condition for the 2-ft-tall cripple wall averaged across all four site locations.

Key findings from the results of the comparison study include:

- **Key finding #1:** For the unretrofitted 2-ft-tall cripple wall conditions, the Modelers consistently estimated lower damage, for both AAL and at 250-year return period across all age groups, heights and locations, with respect to the PEER–CEA Project models. The 250-year return period values of the PEER–CEA Project models were on the order of 200% to 250% larger than the values of the Modelers, and AAL values were on the order of 400% to 700% larger. The PEER–CEA Project models, backed up by the testing program, consistently demonstrated that cripple walls were a significant weak link in the performance of all houses;
- **Key finding #2:** Both the Modelers and PEER–CEA Project predicted greater damage for two-story, raised cripple wall houses versus the one-story home, but the difference is more significant in the PEER–CEA Project models. The PEER–CEA Project AAL values for two-story homes are on the order of 200% of the one-story values, whereas the ratio of the Modelers is closer to 150%. The added weight of the second story adds significant seismic inertial force at the cripple wall. Because the historical design of the unretrofitted cripple wall is typically independent of the number of stories, more damage would be predicted;
- **Key finding #3:** For unretrofitted stem-wall conditions, the Modelers consistently estimated higher damage at the 250-year return period across all age groups, heights, and locations with respect to the PEER–CEA Project models, on the order of 33% to 50%., On the other hand, the AAL values compare with the PEER–CEA Project values quite well, on the order of within 10% to 25%. All three of the Modelers indicated to the Project Team that the quality of their claims inventory data is poor insofar as it distinguishes raised versus stem-wall conditions, and that the differences in performance between the two results primarily emanate from expert opinion within the modeling companies;
- **Key finding #4:** For retrofitted conditions, the results of the PEER–CEA Project and Modelers compare significantly better than in the unretrofitted condition, particularly for single-story construction, with the values for both AAL and 250-year return period for both raised and stem-wall conditions generally within 10% to 25% of each other. For two-story conditions, the Modelers consistently estimated lower damage in the two-story condition compared with the PEER–CEA Project results, by 10% to 40% for the 250-year return period and 30% to 100% for the AAL;
- **Key finding #5:** The PEER–CEA Project results show consistent performance improvement with age, regardless of location, number of stories, and exterior siding material. This is unsurprising as the only difference within the PEER–

CEA Project models for each age category was the weight of the interior finish material. Lath and plaster, representative of older construction is heavier than gypsum wallboard, and thus adds considerable mass and seismic demand, while contributing relatively little additional strength. The Modelers do not consider interior finish material as an explicit variable. The Modelers' results consistently show improvement from the 1945–1955 age band over the pre-1945 age band, but poorer performance from the 1955–1970 age band over the 1945–1955 age band. It is our opinion that the Modelers' results reflect what is known in the industry and is reflected empirically in insurance claims: that the quality of single-family housing decreased in the 1960s and 1970s due to a range of factors, including larger interior open spaces in homes and less union labor being used in California. The Project Team could find no explicit way to model these conditions;

- **Key finding #6:** The Modelers' results show virtually no difference in performance between stucco and wood siding for any of the conditions considered. The PEER–CEA Project models show distinctly better performance for stucco over wood siding in the unretrofitted condition with a raised cripple wall, in both the one- and two-story conditions. The particular weakness of the horizontal siding cripple wall when compared with stucco was evident from the PEER–CEA Project models. This difference mostly disappeared in the stem-wall condition where the stucco and wood-sided houses performed similarly; and

- **Key finding #7:** The PEER–CEA Project results show that retrofitting a two-story, stem-wall house using the ATC-110 plan set, results in slightly poorer performance. This is counterintuitive but is explained as follows. The failure mode for the stem-wall condition, as described by Working Group 5, is the separation of the first-floor joists from the sill plate, which remains attached to the concrete foundation. Once this separation occurs, the superstructure is somewhat isolated from the foundation and the earthquake ground motions. The models show that the amount of slippage of the first-floor joists relative to the sill plate is low—less than an inch—for return periods up to more than 250 years. The repair of this condition would typically be to push the house back to its original position, reattach the joists to the sill plate, and repair damaged stucco or siding up to about two feet above the sill plate. Overall, this is not a particularly expensive repair job. When the house is retrofitted, by solidly attaching the first-floor joists to the sill plate in conformance to the ATC-110 plan set, the isolation effect is lost, and the ground motion is transmitted into the superstructure. As a two-story home is heavier compared to a one-story home, the damage tends to concentrate in the first story, thus exceeding the repair cost associated with the first floor sliding on the sill plate in the unretrofitted condition. The Modelers' results show no such increase in damage in the retrofitted stem-wall condition.

  The significance of this finding should not be overstated. There are many conditions that could lead to poorer performance of unretrofitted two-story stem-wall houses that were not fully evaluated in this limited study. These may include homes where: (1) the existing sill plate connection is weaker than assumed in this study, due to deterioration, inadequate nailing or lack of bolting; (2) the first-story walls are stronger than assumed in this study; (3) the existing sill plate is narrower than assumed; (4) the floor plan or foundations have irregular configurations; and/or (5) the flexibility of the first-floor diaphragm may lead to localized areas of increased deformation and increased risk of separation of the floor from the stem-wall. There could also be considerable variability in the repair cost of a stem-wall house that does slide partially off its foundation sill plate. Given these and other uncertainties in a study of this scope, retrofitting stem-wall houses according to the ATC-110 plan set remains the preferred engineering recommendation

Legend for Figure 10.1 through Figure 10.5.

PEER–CEA Project Results

Modeler Results

Legend for Figures 10.6 and 10.7.

PEER–CEA Project Results

Modeler Averages

Legend for Figure 10.8 through

Legend for Figures 10.8 and 10.9.

PEER–CEA Project Results unretrofitted

PEER–CEA Project Results retrofitted

Modeler Averages unretrofitted

Modeler Averages retrofitted

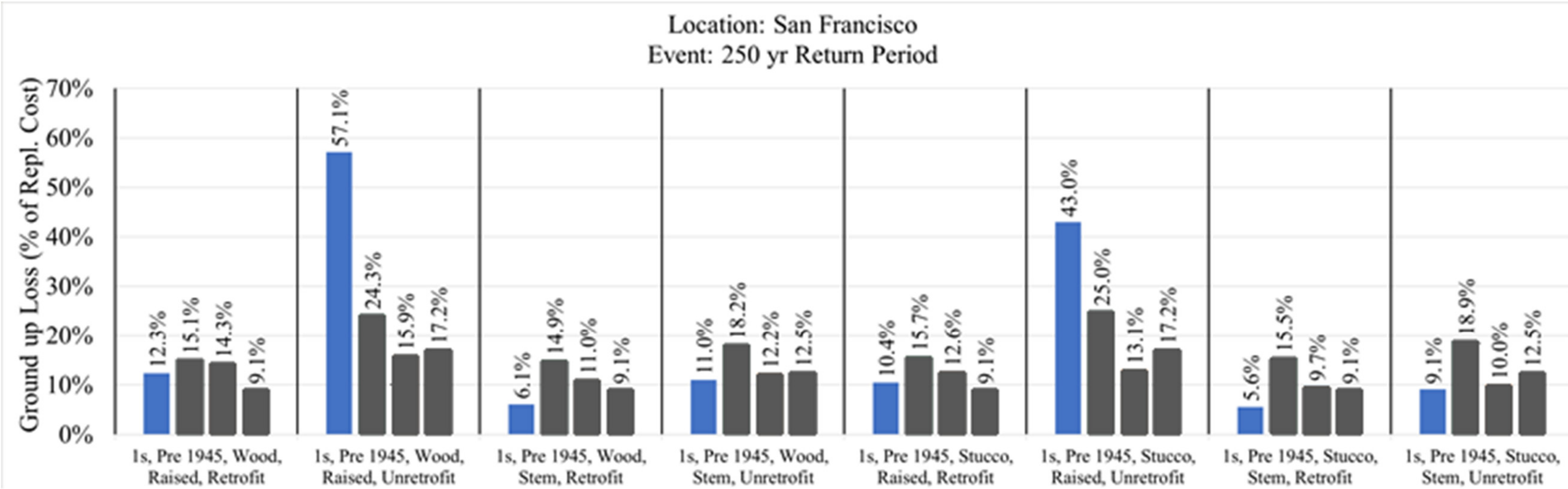


Figure 10.2 San Francisco: loss-comparison results among the four models for a one-story home built pre-1945 with a 250-year return period.

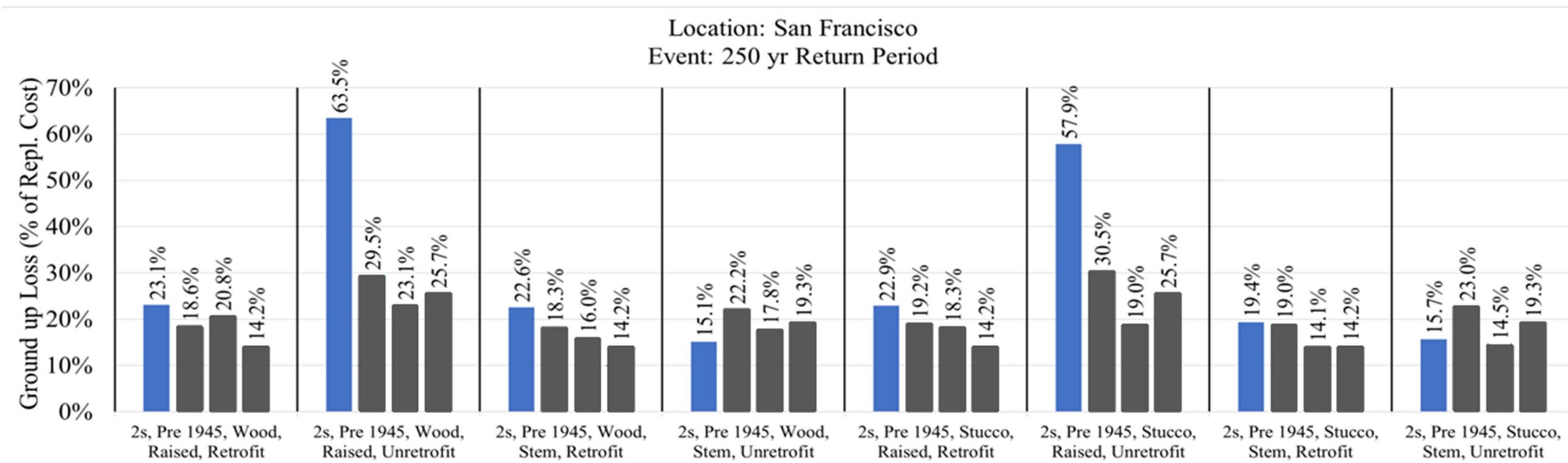


Figure 10.3 San Francisco: loss-comparison results among the four models for a two-story home built pre-1945 with a 250-year return period.

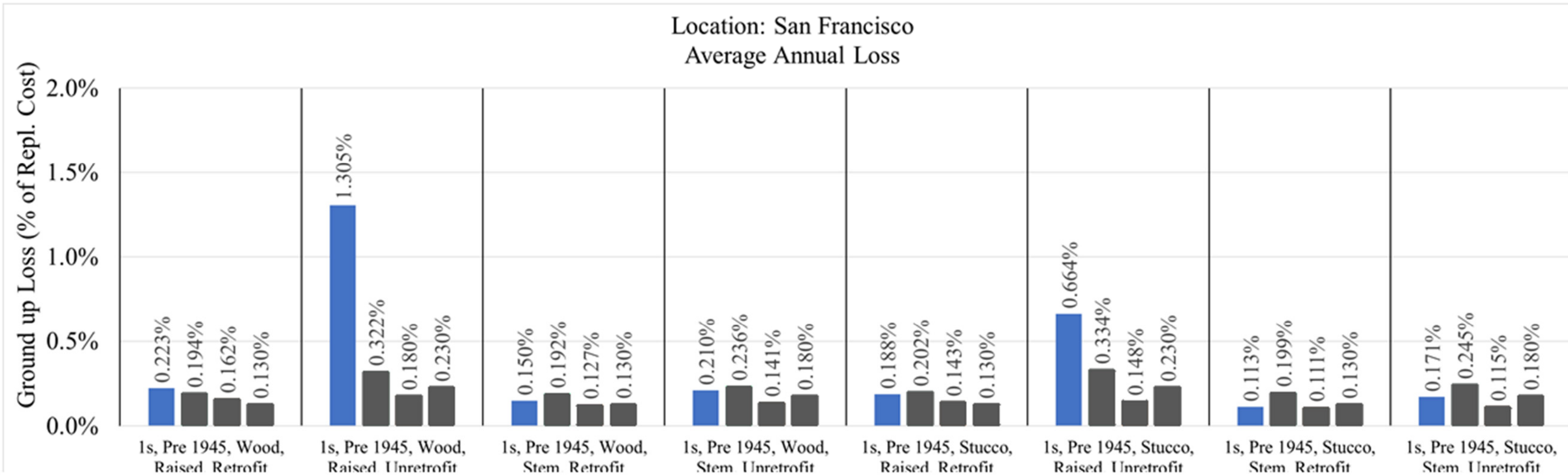


Figure 10.4 San Francisco: comparison of average annual loss among the four models for a one-story home built pre-1945.

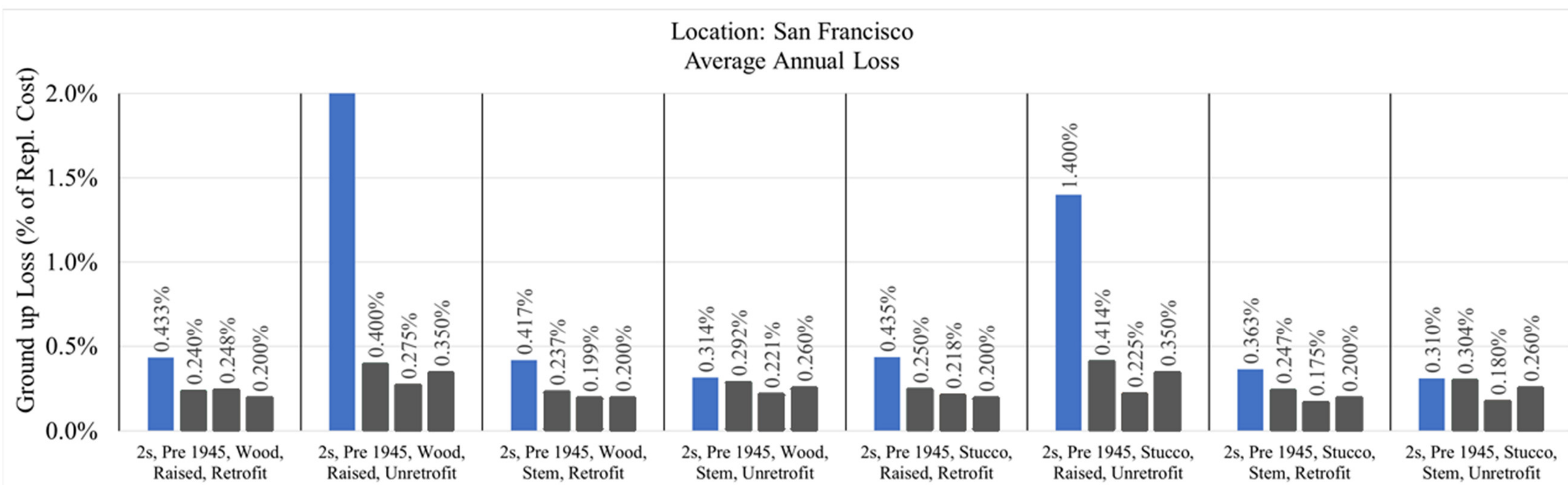


Figure 10.5 San Francisco: comparison of average annual loss among the four models for two-story home built pre-1945.

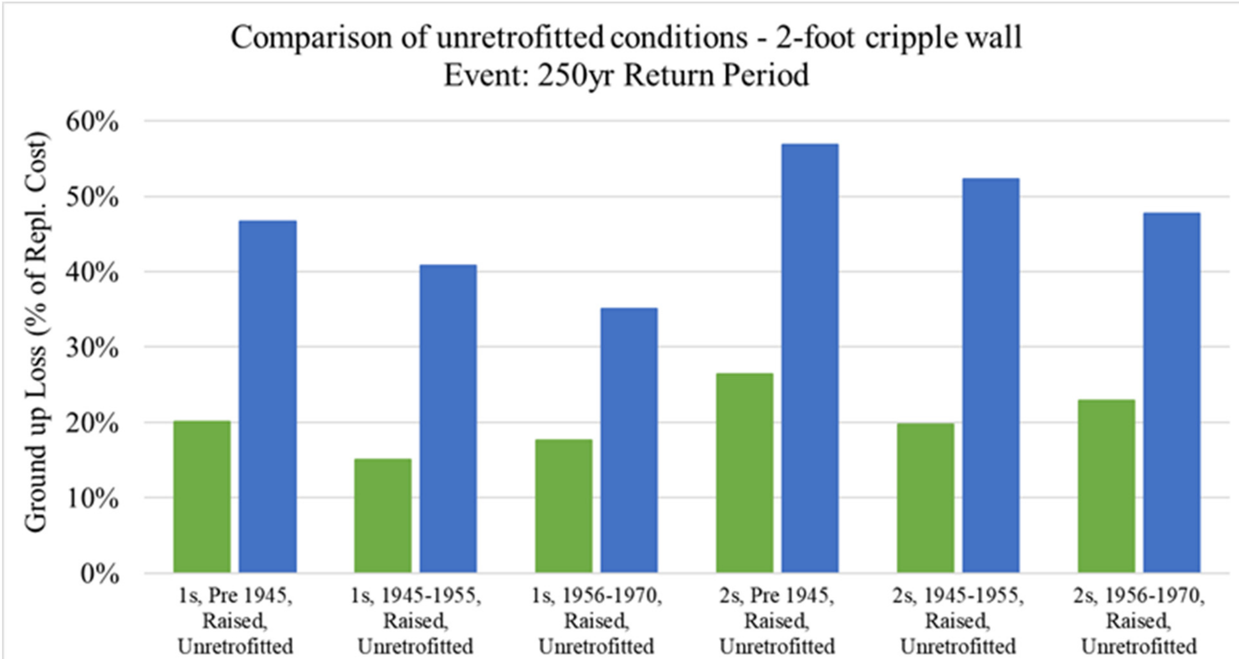


Figure 10.6 **Comparison of unretrofitted conditions, wood siding, 250-year return period.**

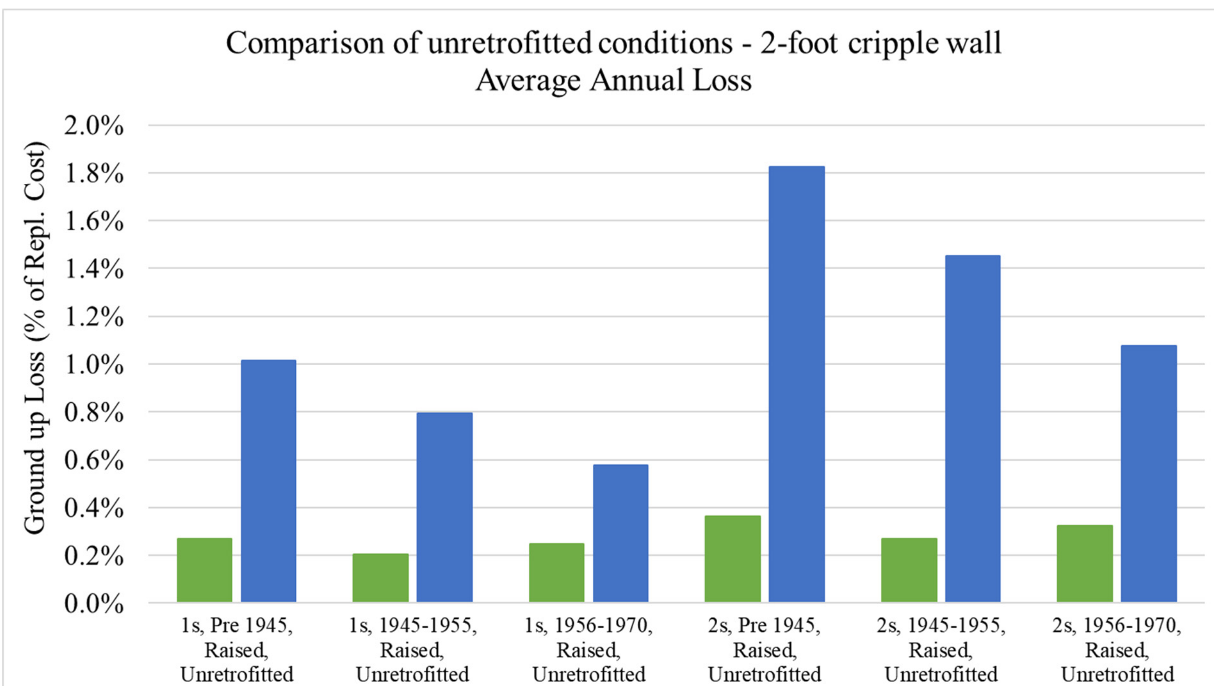


Figure 10.7 **Comparison of unretrofitted conditions, wood siding, average annual loss.**

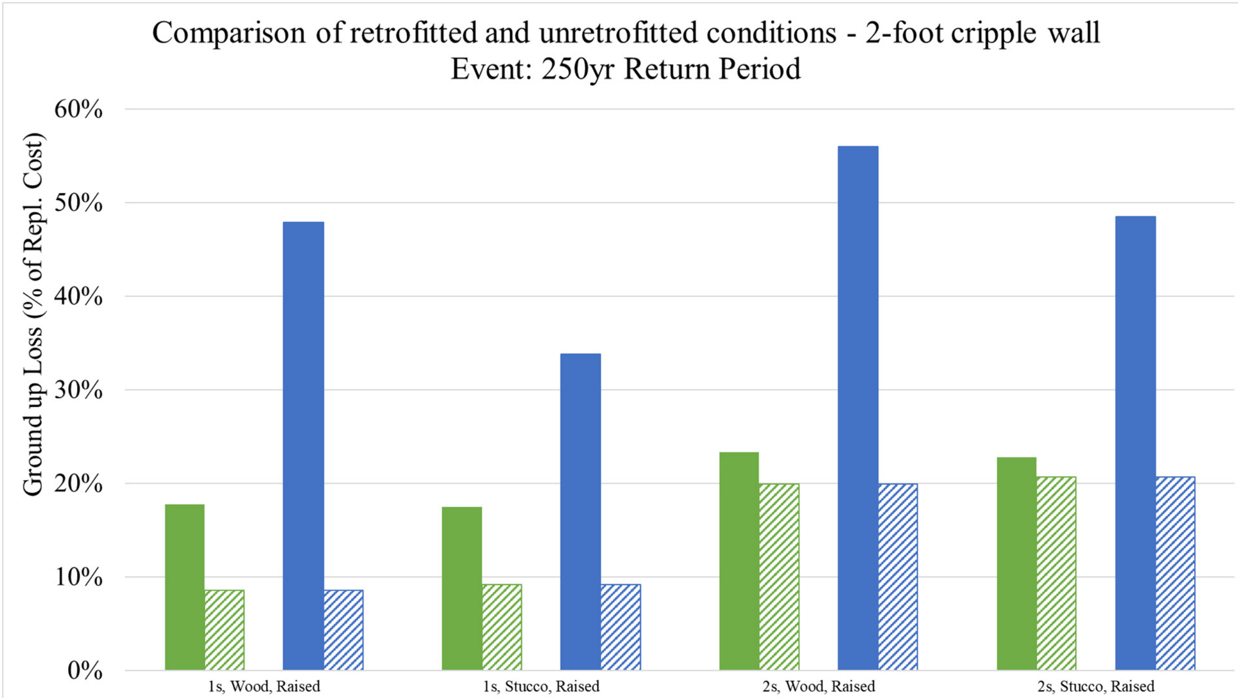


Figure 10.8 **Comparison of unretrofitted and retrofitted conditions, 250-year return period.**

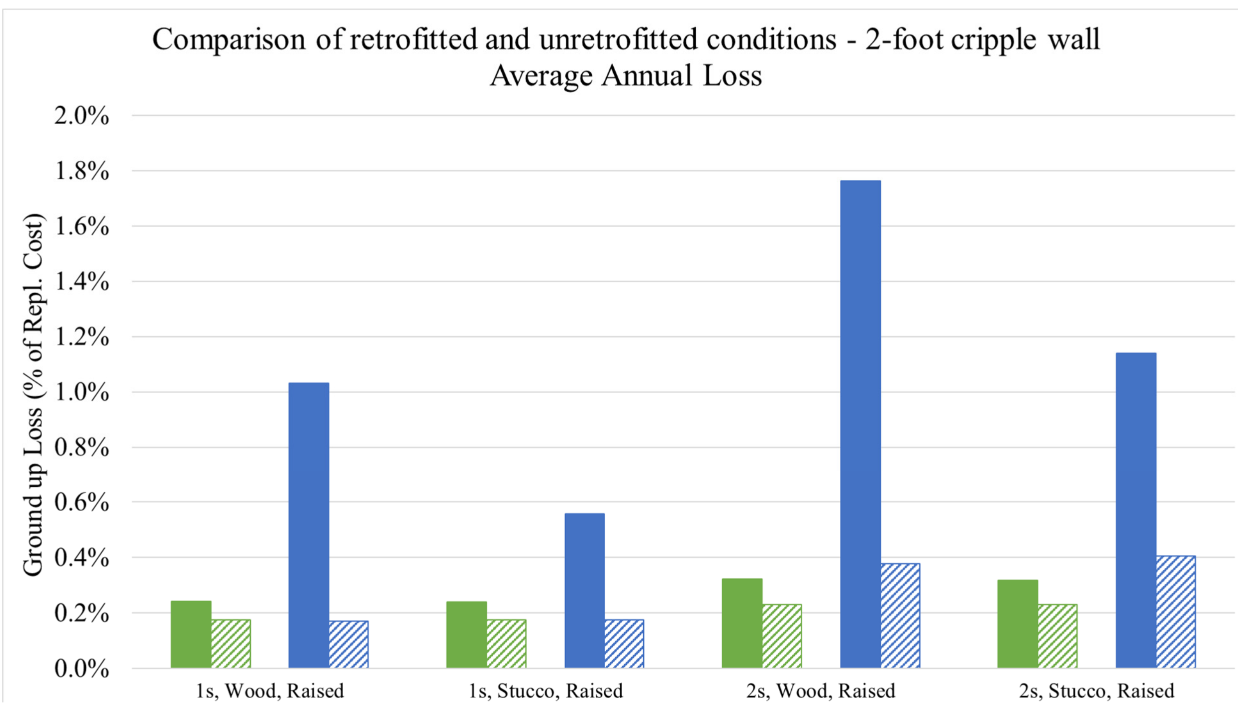


Figure 10.9 **Comparison of unretrofitted and retrofitted conditions, average annual loss.**

An important consideration when comparing the results of the PEER–CEA Project and the Modelers is the deaggregation of building characteristics within the Modelers' DFs. The comparison study was crafted explicitly to consider primary and secondary modifiers (i.e., age, stories, siding, cripple walls, retrofitted condition, etc.) that are available inputs in the Modelers' models. However, all Modelers noted to the Project Team that the differentiation in their damage models is not entirely based on empirical claims data. Much of the claims data incorporated into their models does not contain complete descriptions of the buildings and does not identify the primary and secondary modifiers. Thus, the Modelers must also incorporate expert judgment in assigning DF adjustment factors to account for the individual building characteristics.

An example is even the presence of a raised cripple wall itself. The Project Team identified a single report from the U.S. Department of Housing and Urban Development that attempted to quantify damage to single-family houses in the 1994 Northridge earthquake [HUD 1994]. A sample of 341 structures were surveyed. Of those 341 structures, only 3% were raised cripple wall houses, with the remainder being slabs-on-grade or stem-wall foundations. In a study conducted in 2004, Wesson et al. [2004] developed a DF for single-family homes using ZIP Code-based insurance claims data from the 1994 Northridge earthquake. If the deaggregation across building characteristics of the Wesson data is similar to the HUD study inventory, the DFs would therefore almost completely reflect homes <u>without</u> raised cripple walls. Further assuming that the claims data used by the Modelers in the development of their own DFs would also be heavily influenced by the Northridge insurance data—which comprises a large share of the available empirical data over the past fifty years—it would also be credible to conclude that the Modeler functions are heavily weighted toward slab or stem-wall conditions. Thus, the justification for the significant difference in the AAL and losses at the 250-year return period between the Modelers and the PEER–CEA Project results for raised cripple wall homes may be explained by the implicit weighting of the former toward non-cripple wall structures.

Similarly, the HUD inventory identified that 79% of the surveyed homes were single story. If this was reflective of the insurance data ultimately used by the Modelers, as discussed above, then the DFs could also be heavily weighted toward single-story homes. The large difference in predicted performance between the PEER–CEA Project and the Modelers for two-story homes could also be a factor in the rationale. Modelers may have to adjust DFs based on number of stories.

Similar conclusions could be made for retrofitted versus existing conditions. It would be uncommon for underwriters or claims adjustors to crawl under homes, especially damaged ones, to make a determination as to whether the house had been seismically retrofitted, and certainly not to the extent relative to the ATC-110 plan set, which was only completed in 2019. Therefore, considerable judgment would have been used by the Modelers to adjust DFs to account for retrofit, whereas the PEER–CEA Project considered retrofit explicitly in the modeling and testing.

These key findings suggest that loss modeling would benefit greatly if empirical claims data gathered in future earthquakes contained more detailed information on building characteristics. Damage estimates will be improved by including the following additional required information in the underwriting data collection process and the catastrophe Modelers' software:

- Distinguish between a raised cripple wall and stem wall;
- Distinguish between interior finishes of lath and plaster, and those of gypsum wall board; and
- Distinguish between unretrofitted and retrofitted conditions.

Furthermore, if engineers and the scientific community are to continue to improve methods of credibly estimating building performance in earthquakes and other hazards, it is essential that they collaborate with insurers and have access to the underwriting and claims inventory at a granular level. Sharing this valuable information while finding ways to preserve anonymity and proprietary advantage, would be extremely beneficial toward improving insurance pricing for earthquakes and other natural hazards.

10.5 HAZUS COMPARISON

A comparison of the DFs developed by the Project Team with those developed by FEMA's HAZUS [2014] program and an empirical study of 1994 Northridge insurance claims (Figure 10.10) yielded the following observations:

- The PEER–CEA Project consistently predicted significantly more damage to unretrofitted, raised cripple wall homes, both one and two story, and wood and stucco siding, than the aggregate HAZUS and the Northridge results, which were not broken down by individual building characteristics. This may be explained by the expectation that it is likely that less than 10% of the Northridge dataset contained raised cripple wall homes;
- The PEER–CEA Project predicted less damage than HAZUS for one-story stem wall or retrofitted homes with raised crawl spaces; and
- The PEER–CEA Project predicted generally similar damage as HAZUS for two-story stem wall or retrofitted homes with raised crawl spaces.

As stated above, the Project Team was able to identify a single report from the Department of Housing and Urban Development [HUD 1994] that attempted to quantify damage to single-family houses in the 1994 Northridge earthquake. With this study, some conclusions can be made that are instructive.

- Only 3% of the surveyed homes had raised cripple walls. This would indicate that while the comparative results between Wesson et al. [2004] (which also represents actual Northridge data) and the PEER–CEA Project show that the Project estimated much higher losses for homes with cripple walls. If they did represent only a small fraction of the inventory of homes gathered by HUD and Wesson, then this difference may be easily explained in the implicit weighting of the Wesson results;
- Nearly 80% of the HUD surveyed homes were single story. The Wesson DF compares better with the Project DFs for two-story homes vs. one-story homes. If the construction characteristics of the Wesson data are similar to the HUD inventory, then this would seem to be counterintuitive;
- Ninety-five percent of the HUD houses had stucco siding. The Project results indicate that stucco-clad houses performed better than wood-sided houses, which might mitigate some of the large difference noted with the Wesson et al. results [2004] when compared against the Project wood-siding conditions for cripple wall configurations; and

- Twelve percent of the homes in the HUD study were constructed after 1970. The Project only considered homes built before 1970. If homes in Southern California built after 1970 more commonly employed plywood sheathing or T1-11 siding, then the Wesson results, if they are reflective of the HUD inventory, would predict a lower aggregate DF than the PEER–CEA Project, which only considered homes without plywood.

Until a deaggregation of the Wesson, HUD, and Kircher (HAZUS) DFs can be made by number of stories, cripple wall configuration, retrofitted condition, and siding characteristics, the Project Team does not believe there is justification to modify the DFs developed by this study. In particular, the largest differences being with raised cripple wall configurations can be rationalized because only 3% of the HUD inventory contained cripple wall homes, and it is assumed that the Wesson inventory is similar. In the figure below, cripple wall structures have been removed from the graph, and, in general, the Project, HAZUS, and Wesson DFs are reasonably comparable.

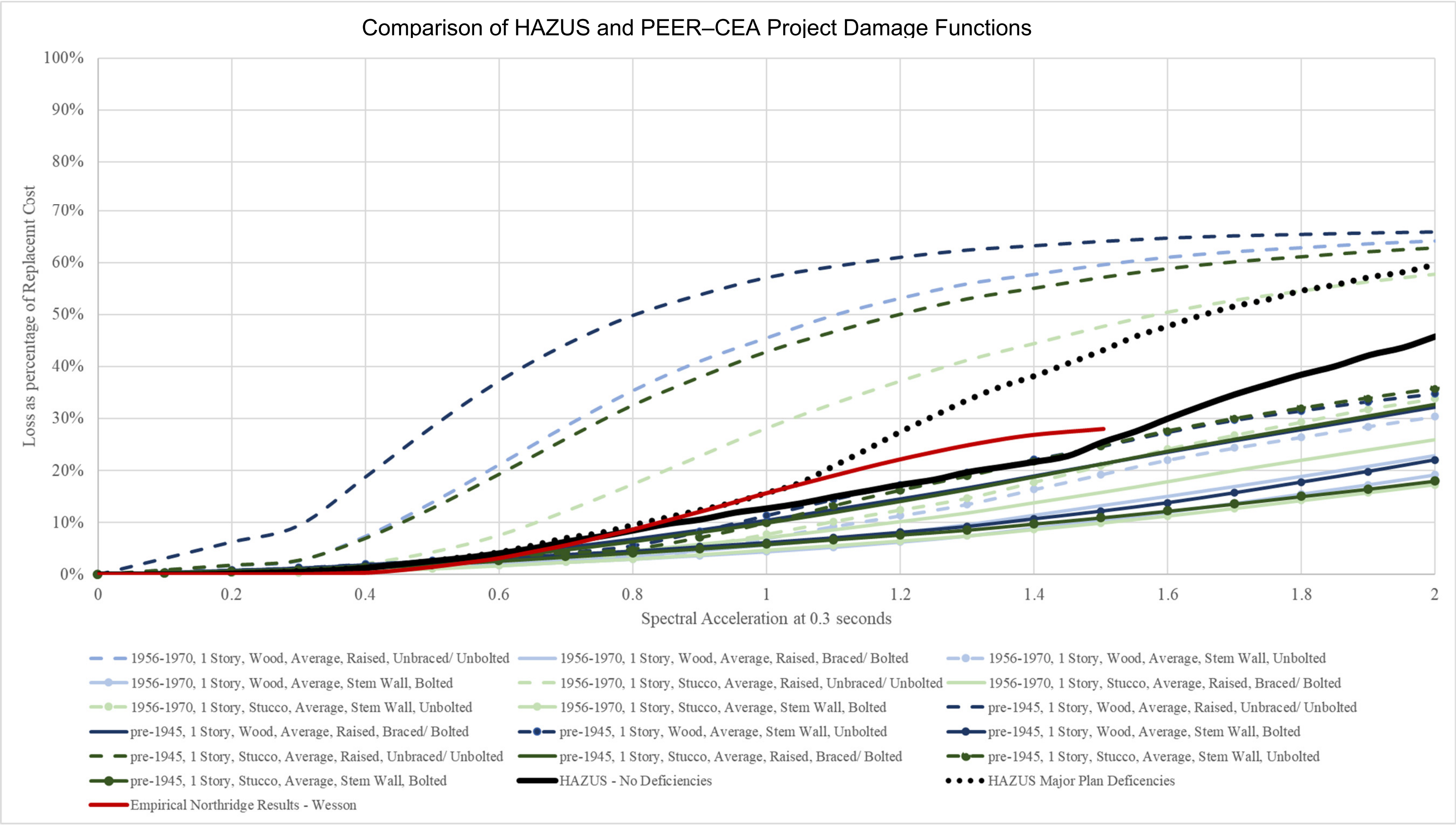


Figure 10.10 Damage function comparisons: HAZUS and Wesson vs. PEER–CEA Project for a one-story building, the average of San Francisco, San Bernardino, and Northridge sites [Wesson et al. 2004, Kircher 2018].

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Appendix A Glossary

TERM	ACRONYM (if any)	DEFINITION AND RELEVANCE	SYNONYMS OR RELATED CONCEPTS
250-year return period	RC250	The level of earthquake shaking that has a 0.4% chance of being exceeded in any given year, or approximately an 18% chance of being exceeded over a 50-year period at a specific site.	
475-year return period		The level of earthquake shaking that is estimated to have a 10% chance of being exceeded over a 50-year period at a specific site.	
actual cash value	ACV	The market value of an item, incorporating depreciation and age.	
actuator		A device used to apply forces and displacements to simulate earthquake loading on part of a building or component.	
annual probability (of exceedence)		The probability that a given level of seismic hazard (typically some measure of ground motion, e.g., intensity or ground acceleration) or seismic risk (typically economic loss or casualties) will be equaled or surpassed within an exposure time of one year.	exceedance probability
as-incurred cost		The actual costs the owner or insurer incurs as repair or related work is undertaken. It is distinguished from estimates for costs not yet incurred.	
average annual loss	AAL	AAL is the expected (average or mean) loss per year, *averaged over many years due to the risk of earthquake damage*. It is computed by averaging the losses in all modeled events, multiplied by their chances of happening in any given year.	expected annual loss, EAL
catastrophe modeling		The practice of using computer-aided calculations to estimate potential losses that could be sustained in a catastrophic event such as a hurricane or earthquake, also called *cat modeling*.	catastrophe modelers
claims adjustor		In property damage claims, a claims adjuster is the person that carries out a detailed investigation into the claim by: Inspecting the damage, reviewing documentation, speaking to property owners, and researching material and labor costs to arrive at an estimate of the insured replacement cost.	
collapse fragility		A mathematical functon that defines the probability of a building collapsing at a given level of earthquake ground shaking intensity.	

TERM	ACRONYM (if any)	DEFINITION AND RELEVANCE	SYNONYMS OR RELATED CONCEPTS
component		A replica of a part or subsection of a house's structure, such as a portion of a wall built to mimic how it would connect to an actual house's foundation, floors, and adjacent walls.	
cost estimate		An estimate of the dollar amount it would take to repair or reconstruct a house following damage, according to the terms of the insurance policy. A cost estimate is typically broken down by contributors to the repair cost, for instance, floors, ceilings, windows or finishes; and the estimate is reported either in dollar amounts or as a fraction (percentage) of building replacement value.	replacement cost value (RCV)
Coverage A		Insurance term for coverage to damage to the main residence.	
Coverage B		Insurance term for coverage for damage to secondary structures besides the main residence.	
Coverage C		Insurance term for coverage for personal property damage.	
Coverage D		Insurance term for coverage for additional living expenses for temporary housing when the residence cannot be occupied due to damage or repair work.	
cripple wall		A framed wall extending from the top of the foundation to the underside of the floor framing of the first floor. Cripple walls usually occur in unoccupied crawl spaces.	crawlspace wall
damage		The term damage, as used by insurers, represents the overall dollar cost of repairing a house to a roughly equivalent former state, not including policy deductibles, exclusions and limits. Insurers also sometimes call this "ground up loss." When used by engineers, damage more commonly refers to a physical description of the impacts of an earthquake on a building. Loss as commonly defined by engineers is assumed to be equivalent to the term damage as used by insurers.	
damage function	DF	An equation or set of data points describing the expected repair cost of damage across a range of of earthquake shaking intensities.	
damage state		A qualitative description of particular kinds and levels of damage, often expressed in categories that represent multiple increasing tiers or levels from none to less to more severe.	
deformation		Any changes in the shape or size of an object due to an applied force.	

TERM	ACRONYM (if any)	DEFINITION AND RELEVANCE	SYNONYMS OR RELATED CONCEPTS
demand surge		A regional increase in price for construction services that can happen following a large, local disaster.	
design ground motion		The level of ground motion used in structural design of a house to resist earthquakes for a particular location, often as required by local building codes.	design earthquake
direct costs		Costs paid by the owner to the general contractor. They are often termed “above-the-line costs” or “hard costs.”	indirect costs
displacement		How far and in what direction a building component has moved from its original position.	
drift		Drift is a measure of the horizontal displacement of a building or component’s under earthquake shaking that can cause damage both to the structural frame and to non-structural elements. *Global drift* is the largest observed deformation in the overall structure, and *story drift* is the horizontal displacement of one level relative to the level below.	
engineering demand parameters	EDPs	Structural response measures of forces, deformations or accelerations imposed on a house by an earthuqake (or other) loadings that are used to estimate the potential for structural and nonstructural damage (e.g., story drifts and peak floor accelerations).	
escalation		The increase in costs between the date assumed for pricing (typically when the estimate is made) and the future date when the work will actually occur. A common approach in the design profession is to escalate between the time of the estimate to the mid-point of construction. Escalation can occur for instance because of inflation, season, or demand surge.	
existing		A description meaning a building is as-built or in its original state, without any seismic retrofitting to the cripple wall or sill anchorage.	unretrofitted, unretrofit
FEMA P-58		Guidelines for estimating damage and losses to buildings, developed by the Federal Emergency Management Agency, which is based on detailed structural analyses and damage assessment to building components and systems.	
FEMA P-1100 prestandard and plan sets		A prestandard and plan sets for the vulnerability-based assessment and retrofit of one- and two-family wood light frame residential buildings, described in the following report: https://www.atcouncil.org/atc-110-cea-2 (Accessed May 5, 2020)	

TERM	ACRONYM (if any)	DEFINITION AND RELEVANCE	SYNONYMS OR RELATED CONCEPTS
fragility function	FF	An equation that describes the relationship between the ground motion shaking intensity (or the demands imposed by the ground motion) and a physical description of the expected damage to a structure or component.	fragility curve
ground-motion parameter		A parameter characterizing a ground motion, such as peak acceleration, peak velocity, and peak displacement (peak parameters) or ordinates of response spectra and Fourier spectra (spectral parameters).	ground-motion intensity measure (IM)
ground up loss		An insurance notation that describes earthquake damage and repair costs irrespective of insurance policy terms such as deductibles and limits, usually specified as a monetary value or as a fraction or percentage of the total value of a property or portfolio of properties.	
hazard curve		A plot of probabilistic seismic hazard (usually specified in terms of annual probability of exceedance) or average return period versus a specified ground-motion parameter or intensity measure (IM) for a given site.	seismic hazard curve
HAZUS	HAZUS	HAZUS is a nationally applicable standardized methodology that estimates potential losses from earthquakes, hurricane winds, floods, and tsunamis developed by the Federal Emergency Management Agency (FEMA). HAZUS uses state-of-the-art Geographic Information Systems (GIS) software to map and display hazard data and the results of damage and economic loss estimates for buildings and infrastructure. It also allows users to estimate the impacts of natural disasters on populations.	
hysteretic response curve		A plot relating the load (or pressure) and the deformation of a system (in this case, a building or building component) to changes, which are dependent upon past reactions to change.	
index building		Description of a set of building characteristics that define a category of woodframe houses with similar expected response to earthquake shaking.	representative inventory, prototypes, representative categories
indirect costs		Costs paid or assumed in estimates in addition to direct costs, such as contingencies for changes or unknown conditions, utilities, design fees, plan check and permitting cotss, and abatement costs, temporary moving fees, financing costs, and legal fees. They are often termed “below-the-line costs” or “soft costs.”	direct costs
lateral load		The horizontal forces acting on a structure or component.	

TERM	ACRONYM (if any)	DEFINITION AND RELEVANCE	SYNONYMS OR RELATED CONCEPTS
loading protocol		The selected procedure for applying forces and/or displacements to test how building components will respond to earthquakes.	
load path		The route by which gravity and lateral forces are transmitted from the top of the structure down through a structure to the foundation.	
loss		An insurance or engineering term used to describe the damage in terms of expected repair costs. In insurance, loss has a formal definition specific to the policy, where coverage limits and exclusions such as deductibles or specific causes are not included. It is the basis of a claim for damages under the terms of a policy. In engineering, loss is more commonly defined as the total, overall amount of damage without consideration of policy deductibles, exclusions and limits.	damage
loss model		Mathematical models of the amount of damage expected under various earthquake scenarios. In insurance, loss refers to damage with consideration of deductibles, exclusions and limits. In engineering, loss more commonly refers to damage without consideration of policy deductibles, exclusions and limits.	loss estimation, loss assessment
modifier		House characteristics known to be associated with different degrees of earthquake vulnerability and expected responses to earthquake shaking. For this project, modifiers are what define one type of index building from another. Modifiers are primary or secondary.	index building, variants
monotonic envelope		The plot diagram of applied force compared to resulting change in shape (deformation) as load is steadily and increasingly applied to a building or structural component.	
near-field effects		Variations in structural responses, or ground motion characteristics, at sites located in close proximity of an earthquqake fault rupture.	
nonlinear response history analysis		A simulation method for modelling how an existing or proposed strutural design may respond to imposed earthquakeground motions. Nonlinear response considers how building performance changes over the duration of an earthquake event as systems and components are increasingly stressed or damaged.	nonlinear dynamic analysis

TERM	ACRONYM (if any)	DEFINITION AND RELEVANCE	SYNONYMS OR RELATED CONCEPTS
nonstructural components		The parts of a building system that are not specifically designed to support applied loads (e.g., architectural partition walls, exterior claddings, windows, doors, lighting fixtures, etc.). While nonstructural components may not be designed to resist applied loads, they may in fact contribute to the overall resistance to earthquakes – especially in wood-framed residential houses.	
pre-earthquake state		The condition of the building before the earthquake.	
probabalistic seismic hazard analysis		Available information on earthquake sources in a given region is combined with theoretical and empirical relations among earthquake magnitude, distance from the source and local site conditions to evaluate the exceedance probability of a certain ground motion parameter, or intensity measure (IM), at a given site.	
probability of exceedance		The probability that, in a given area or site, an earthquake ground motion will occur that is greater than a given value, over a specified time period.	exceedance probability
probable maximum loss	PML	The probability that a given level of seismic hazard (typically some measure of ground motion, e.g., intensity or ground acceleration) or seismic risk (typically economic loss or casualties) will be equaled or surpassed within a specific time period.	
quasi-static cyclic experiment		In quasi-static laboratory tests of structural components, loads and/or displacements are applied at a slow rate to study structural performance through the gradual, increasing size and rate of propagation of damage.	
replacement cost		The cost to replace damaged part or the complete building with a roughly equivalent new version. This does not include the value of the land.	cost estimate
ground-motion response spectra		A relationship between the natural period of vibration of a single degree of freedom system and the maximum response that it experiences under an earthquake ground motion.	response spectra
repair cost		The estimated cost of returning the home to its pre-earthquake state using similar nonstructural finishes, materials, and approach. This may include patching or replacing in kind of either nonstructural or structural elements, but structural elements are not strengthened beyond their original pre-earthquake state.	

TERM	ACRONYM (if any)	DEFINITION AND RELEVANCE	SYNONYMS OR RELATED CONCEPTS
replacement cost of the entire structure		The cost to rebuild the home as defined under "replacement of the entire structure." This includes the demolition and removal cost of the existing damaged building.	replacement of the entire structure
replacement of the entire structure		"Replacement" in this context means to rebuild the home such that it is as similar to the building before the earthquake as possible, together with any required building-code upgrades. This term does not include individual elements and nonstructural finishes, which may be replaced separately as part or repairs or upgrades.	replacement cost of the entire structure
replacement cost value	RCV	The estimated cost to replace a damaged house with a new version roughly equivalent to how it was in its pre-earthquake state. In insurance, a repair cost estimate can be higher or lower than the replacement cost.	replacement cost
residual displacement		For the cripple wall, this is the displacement that remains at the end of the earthquake at the top of the first floor relative to the top of the foundation. It is visible as a lean in the cripple wall. For the superstructure in a one-story building, this is the displacement between the top of the walls at the eave level and the top of the first floor. In a two-story building, it is the displacement between the first and second floor, or between the second floor and the eaves. Residual displacement is a useful metric for correlations with damage. The residual displacement lean occurs both in the direction parallel to the wall (termed "in-plane") and the direction perpendicular to the face of the wall (termed "out-of-plane").	displacement
retrofit		Changes made to a completed building to meet needs that were not considered at the time it was built, in this Project, to make it better able to withstand an earthquake. Examples include adding sill plate connections, wood sheathing or bracing, anchors, bracing, bolts and/or tie-downs. The terms "retrofit" and "retrofitted" are used interchangeably used to describe cripple walls to which sill anchorage and bracing have been added. Further, the term retrofit only refers to cripple wall-related seismic improvements, not other potential seismic upgrades such as chimney bracing or removal; full foundation replacement; ground stabilization; changing roof material or attachment; porch, garage, or appendages; or nonstructural measures.	seismic retrofit, seismic strengthening

TERM	ACRONYM (if any)	DEFINITION AND RELEVANCE	SYNONYMS OR RELATED CONCEPTS
retrofitted		The building has been improved from its original state by seismic retrofitting to the cripple wall and/or sill anchorage. In this project, the term retrofit only refers to cripple wall-related seismic improvements, not to chimney bracing or removal; full foundation replacement; ground stabilization; roof material or attachment; porch, garage, or appendages; or nonstructural measures.	retrofit, seismically strengthened
return period		The average time between exceedance of a specified level of ground motion at a specific location. It is equal to the inverse of the annual probability frequency of exceedance.	recurrence interval
shear wall		Structural walls that are constructed with wood sheathing (e.g., plywood or oriented strand board, OSB) or wall coverings added to a structure to resist horizontal (shear) forces. These are usually solid elements, and are not necessarily designed to carry the weight of the structure.	
sill anchorage		The use of bolts, brackets or other hardware to connect a superstructure, crawlspace wall, or stemwall to the foundation sill.	sill bolting
single-family dwelling	SFD	A structure intended for use by one household, as opposed to a building with multiple apartments or units. For purposes of earthquake insurance, two unit buildings (duplexes) are also eligible.	single-family wood-frame house or home, one- and two-family dwellings
square-root-of-sum-of-squares	SRSS	A mathematical calculation method used in seismic analysis that estimates the combined effects of independent variables.	
spectral accerelation		A unit measured in *g* (the acceleration due to Earth's gravity, equivalent to *g*-force) that describes the maximum acceleration in an earthquake on an object. When combined with the mass of the object, spectral accelartion results in a force that acts upon the object. For this project, spectral acceleration assumptions were used in the modelling of how a house would respond to different intensities of earthquake shaking.	
stem wall		A type of foundation where the first floor joists rest directly on the foundation sill plate, which sets on a concrete or masonry foundation stem wall. In other words, a framing configuration without any unbraced crawl space (cripple) wall.	Zero-height cripple wall
structural system		A system of building elements that resist the weight of a building's downward force of gravity and horizontal forces that can be caused by such hazards as wind, blast, or earthquakes.	

TERM	ACRONYM (if any)	DEFINITION AND RELEVANCE	SYNONYMS OR RELATED CONCEPTS
subassembly		For this project, a unit assembled separately for testing but designed as if it were to be incorporated with other units into a complete model of a house.	
superstructure		The part of a building or construction entirely above its foundation or basement. For this project, the structural parts of a house that are above the crawlspace, from the first floor and up.	
underwriter		A person whose job is to calculate the risk that is involved in providing insurance for a particular customer, and to decide how much should be paid for insurance.	
unretrofitted		The building is in its original, as-built state, without any seismic retrofitting to the cripple wall or sill anchorage. Also referred to in other Project reports as “existing” or “unretrofit,” all terms being synonymous	existing, unretrofit
upgrade		Going beyond repair to improve the building so that its structural performance is expected to be better than it would be in the pre-earthquake state. Adding plywood and associated connections to the framing to a wall, roof, or floor that did not have plywood would be an example of an upgrade. This definition does not include upgrades to nonstructural finishes to a higher level of quality than existed prior to the earthquake.	
variant		A specific combination of building characteristics that define a category of wood-frame houses with similar expected response and susceptibility to damage from earthquake shaking.	index building
wood siding		For this project, wood siding refers to lumber siding boards horizontally-applied to the outside of a building or crawlspace wall.	shiplap siding
Xactimate		For this project, the Verisk Xactware Xactimate X1 software platform was used by claims adjustors to categorize and repair cost values for damage to houses.	

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