PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

Cripple Wall Small-Component Test Program: Dry Specimens

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 2020/17

Pacific Earthquake Engineering Research Center

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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 measures and documents seismic performance of wood-frame houses with cripple wall and sill anchorage deficiencies as well as retrofitted conditions that address those deficiencies. Three primary tasks support the earthquake loss-modeling effort. They are: (1) the development of ground motions and loading protocols that accurately represent the diversity of seismic hazard in California; (2) the execution of a suite of quasi-static cyclic experiments to measure and document the performance of cripple wall and sill anchorage deficiencies to develop and populate loss models; and (3) nonlinear response history analysis on cripple wall-supported buildings and their components.

This report is a product of *Working Group 4: Testing,* whose central focus was to experimentally investigate the seismic performance of retrofitted and existing cripple walls. This present report focuses on non-stucco or "dry" exterior finishes. Paralleled by a large-component test program conducted at the University of California, Berkeley (UC Berkeley) [Cobeen et al. 2020], the present report involves two of multiple phases of small-component tests conducted at University of California San Diego (UC San Diego). Details representative of era-specific construction–specifically the most vulnerable pre-1960s construction–are of predominant focus in the present effort. Parameters examined are cripple wall height, finish style, gravity load, boundary conditions, anchorage, and deterioration. This report addresses all eight specimens in the second phase of testing and three of the six specimens in the fourth phase of testing. Although conducted in different testing 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. Each specimen was 12 ft in length and 2-ft or 6-ft in height. All specimens in this report were constructed with the same boundary conditions on the top, bottom, and corners of the walls. Parameters addressed in this report include: dry exterior finish type (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 (including instrumentation), and measured as well as physical observations are summarized. Results from these experiments are intended to support advancement of numerical modeling tools, which ultimately will inform seismic loss models capable of quantifying the reduction of loss achieved by applying state-of-practice retrofit methods as identified in *FEMA P-1100 Vulnerability-Base Seismic Assessment and Retrofit of One- and Two-Family Dwellings.*

ACKNOWLEDGMENT AND DISCLAIMER

ACKNOWLEDGMENT

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DISCLAIMER

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1 Introduction

1.1 PREAMBLE

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.

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 & 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.

Working Group (WG) 4 testing focused on the first phase of an experimental investigation to study the seismic performance of retrofitted and existing cripple walls with sill anchorage. All tests discussed in this report were finished with non-stucco or "dry" materials. Paralleled by a large-component test program conducted at University of California Berkeley (UC Berkeley) [Cobeen et al. 2020], the present study involves the second and a portion of the fourth phase of the four phases of small-component tests conducted at UC San Diego.

In *Cripple Wall Small-Component Test Program: Wet Specimens I*, the strategy for characterizing the primary variables and their ranges for the small-component cripple wall test program at UC San Diego is described. In addition, the background and motivation for the study, field observations from past cripple wall failures, previous research on the topic, and details of how the loading protocol were selected [Schiller et al. 2020(a)]. Thus, in the present report, only the salient features of the program as well as specific differences with respect to the scope of the specimens reported herein, are summarized.

1.2 UC SAN DIEGO TEST PROGRAM

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 considered a similar theme allowing for meaningful comparisons amongst specimens within a particular phase, and yet were complimentary to other phases for cross comparison upon completion of subsequent phases. The scope and purpose of each testing phase is as follows:

- *Phase 1.* The first phase of testing contained six cripple wall specimens. Each of the cripple walls were 2 ft tall and finished on their exterior face with stucco installed over horizontal lumber sheathing. In addition, a uniform vertical load of 450 lbs/ft was applied to each specimen. Parameters amongst specimens in this phase included: the specimens boundary conditions, anchorage conditions, and existing or retrofit detailing. By controlling the exterior finish, height, and applied vertical load, the results of the Phase 1 tests work offered insight into the importance of the boundary conditions (ends, top, and bottom) of the wall on the performance of the specimens. In addition, one of the cripple walls was constructed with a wet set sill, a previously untested type of anchorage. Lastly, two of the cripple walls were identical, with one being an existing condition and the other being a retrofitted condition [Schiller et al. 2020(a)].
- *Phase 2.* The second phase of testing contained eight cripple wall specimens. Six of the cripple walls were 2 ft tall, and two of the cripple walls were 6 ft tall. Similar to Phase 1, all wall specimens are subjected to 450 lbs/ft of vertical load. The boundary conditions remain the same for all specimens. The walls differed from each other in exterior finishes, height, and retrofit condition. The eight walls were grouped in four identical pairs of existing and retrofitted walls. All specimens had sill plates attached to the foundation with anchor bolts. The main focus of Phase 2 was to document the performance of dry–or non-stucco–exterior finish materials. One pair of walls was finished with T1-11 wood

structural paneling, one pair was finished with shiplap horizontal lumber siding over diagonal lumber sheathing, and the final two pairs were finished with shiplap horizontal lumber siding. The two pairs with horizontal siding differed in height, one pair being 2 ft tall and the other being 6 ft tall. These tests provided insight regarding the performance of dry-finished specimens, with emphasis on understanding the failure mechanisms associated with short and tall cripple walls. In addition, the results of four retrofitted walls built upon knowledge gained in Phase 1 regarding the effectiveness of the *FEMA P-1100* prescriptive retrofit [Schiller et al. 2020(b)].

- *Phase 3.* The third phase of testing also consisted of eight specimens. These specimens were each 2 ft tall and had the same boundary conditions imposed on the top and ends of the cripple walls. There were three pairs of identical walls that only differed in their retrofit condition. A uniform vertical load of 450 lbs/ft was consistently applied for all specimens. Key parameters differing among the specimens in this phase included the exterior finish details and the bottom of specimen boundary conditions. Pairs of cripple walls with stucco over horizontal lumber sheathing, stucco over diagonal lumber sheathing, and stucco over framing were tested. One cripple wall was constructed with a wet set sill plate. Results of these three pairs of tests examined the performance of differing wet or stucco exterior finishes, as well as provide additional results regarding the performance of the *FEMA P-1100* prescriptive retrofit [Schiller et al. 2020(c)].
- *Phase 4.* The final phase of testing consisted of six specimens. All wall specimens were detailed with the same boundary conditions. Two pairs of identical 6-ft-tall cripple walls were tested, both existing and retrofitted. Two walls were detailed with stucco over framing exterior finishes, while the other two utilized T1-11 wood structural panel exterior finishes. Two of the six specimens were 2 ft tall. One of these had stucco over horizontal lumber sheathing and was loaded with a monotonic push. The other cripple wall had shiplap horizontal sheathing over diagonal lumber sheathing and was tested with a light uniform vertical load of 150 lbs/ft. Results from this phase investigated the effect of height on the performance of the cripple wall and the *FEMA P-1100* prescriptive retrofit. In addition, the effect of a light vertical load and a monotonic push loading protocol was evaluated [Schiller et al. 2020(b); 2020(c)].

While there are four phases of testing, the reporting of each phase is not strictly organized based on the testing phase; however, four reports are available to summarize the UC San Diego small-component test program. Their organization is designed as follows: the first report and the third report focus on wet specimens, i.e., specimens with stucco exterior finishes (i.e. Phase 1, Phase 3, and a portion of Phase 4). The present (second) report focuses solely on dry specimens, i.e. specimens finished with wood absent stucco (i.e., Phase 2 and a portion of Phase 4). The final (fourth) report presents a cross comparison of specimens, both wet and dry finishes. These reports are as follows:

- Cripple Wall Small-Component Test Program: Wet Specimens I [Schiller et al. 2020(a)]
- Cripple Wall Small-Component Test Program: Dry Specimens [Schiller et al. 2020(b)]
- Cripple Wall Small-Component Test Program: Wet Specimens II [Schiller et al. 2020(c)]
- Cripple Wall Small-Component Test Program: Comparisons [Schiller et al. 2020(d)]

1.3 SCOPE OF THIS REPORT

As with the tests discussed, a consistent wall length, framing plan, and foundation setup were utilized for each test. With a focus on dry finished specimens, the present report is organized as follow:

- Chapter 2 presents the test matrix and details of specimens reported specifically in the present report, namely all dry finished specimens. Subsequently, the testing setup and loading protocol utilized are described. In addition, a visual documentation of the construction of the cripple walls is provided. Finally, the layout of instrumentation used to acquire data for each test is presented;
- Chapter 3 presents the results from each tested specimen. Specifically, the load-deflection response, anchor bolt load histories, relative displacement measurements, distortion within panel segments of the wall specimens, and vertical displacement of the wall are presented;
- Extensive documentation of the physical damage to each cripple wall specimen is provided in Chapter 4. Visually documented damage is correlated with key attributes of the measured load-deflection curves provided in Chapter 3;
- Finally, Chapter 5 will provide concluding remarks regarding observations from the dry or non-stucco test program; and
- Appendices A, B, and C document material properties, instrumentation plans, and an expansion of all measured response results for individual specimens.

Specimen Details, Test Setup, and Instrumentation

2.1 GENERAL

The focus of this chapter is on the details of the cripple wall specimens, test setup, and testing instrumentation for the dry (non-stucco) exterior finished specimens of the test program. As described in the previous chapter, there are a multitude of variables to be examined in this entire test program. Key parameters in this report are the height, dry exterior finishes, retrofit condition, and vertical load of the cripple walls. In Phase 1 of testing, the effect of various boundary conditions on the top, bottom, and sides of the cripple wall were investigated. This allowed for baseline boundary conditions to be used in subsequent testing. The construction details for all other boundary conditions can be found in the previous report [Schiller et al. 2020(a)]. Herein, the baseline boundary conditions adopted for all specimens are top boundary condition B and bottom boundary condition c.

Three different dry exterior finishes were selected for testing, namely: (1) horizontal shiplap lumber siding; (2) horizontal shiplap lumber siding over diagonal lumber sheathing; and (3) T1-11 wood structural panels. Horizontal siding and horizontal siding over diagonal sheathing are finish styles common in 1945–1955 era of housing construction. T1-11 wood structural panels are a common finish style of 1956–1970 era of housing construction. With the exception of one specimen, all specimens tested in pairs of retrofitted and existing cripple walls to elicit the benefits of retrofitting existing specimens. The specimen without a retrofit counterpart was subjected to a light vertical load (150 plf), emulating the weight of a single-story house with light building materials. All other specimens were subjected to a heavy vertical load (450 plf), which is representative of a two-story house constructed with heavy building materials. Four of the tests were 6-ft-tall cripple walls, two existing and retrofit pairs. One of the 6-ft-tall cripple wall pairs was constructed with horizontal shiplap siding, and the other pair was constructed with T1-11 wood structural panels. The other seven cripple walls were 2 ft tall and constructed with horizontal siding (2), horizontal siding over diagonal sheathing (3), and T1-11 plywood structural panel (2) exterior finishes. In totality, 11 specimens of the 28 conducted within the overall test program, were finished with dry materials and thus are included in the present report. Table 2.1 summarizes the variables for specimens described herein.

Table 2.1 **Dry finished specimens summarized. All specimens are subjected to a cyclic loading protocol. All boundary conditions are the same, namely top boundary condition B and bottom boundary condition c.**

Phase	Specimen	Test no. (date)	Existing or retrofit	Era	Vertical load	Anchorage	Exterior finish	Test date
2	A-7	7	E	1945–1955	H	S(64 in.)	HS	**5/11/2018**
	A-8	8	R	1945–1955	H	**S(32 in.)**	HS	**5/22/2018**
	A-9	11	E	1945–1955	H	S(64 in.)	HS+DSh	**7/19/2018**
	A-10	12	R	1945–1955	H	**S(32 in.)**	HS+DSh	**7/26/2018**
	A-11	9	E	1956–1970	H	S(64 in.)	T	**6/15/2018**
	A-12	10	R	1956–1970	H	**S(32 in.)**	T	**6/28/2018**
	A-13	13	E	1945–1955	H	S(64 in.)	HS	**8/26/2018**
	A-14	14	R	1945–1955	H	**S(32 in.)**	HS	**8/30/2018**
4	A-23	23	E	1956–1970	H	S(64 in.)	T	**9/16/2019**
	A-24	24	R	1956–1970	H	**S(32 in.)**	T	**10/3/2019**
	A-28	25	E	1945–1955	L	S(64 in.)	HS+DSh	**10/10/2019**
Denotes 1945–1955 Era			Retrofit		Low vertical load	Wet set sill or retrofit		
Denotes 1956–1970 Era								

Notes: E = existing, R = retrofit, S = anchor bolt spacing, H = heavy vertical load (450 plf), L = light vertical load (150 plf), HS = horizontal siding, DSh = diagonal sheathing, and T = T1-11 wood structural panels

2.2 SPECIMEN DETAILS

2.2.1 Framing Details

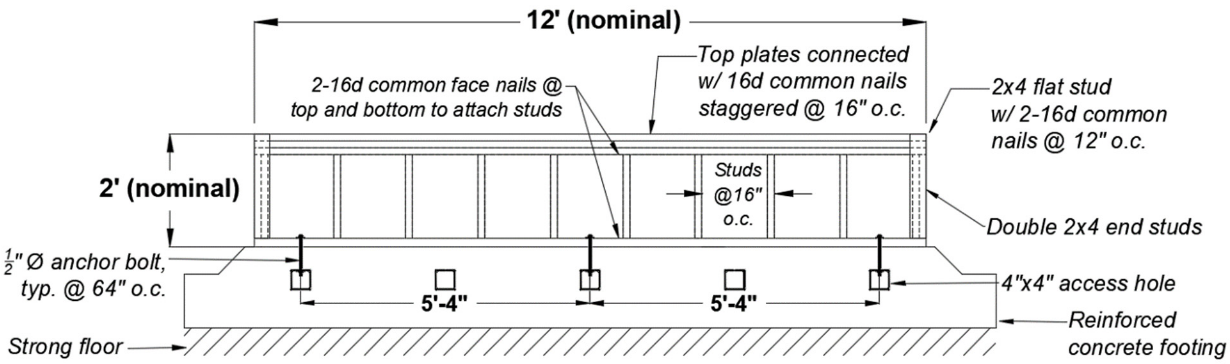
Seven of the eleven cripple wall specimens in this report are nominally 2 ft in height and 12 ft in length, as shown in Figure 2.1. The remaining four specimens are nominally 6 ft in height and 12 ft in length; see Figure 2.2. Note that the anchor bolt spacing shown in these figures applies on to existing cases. Minor differences in length can be attributed to the nuances of the exterior finish detailing. The height of the cripple wall is measured from the base of the sill plate to the top of the uppermost top plate. Framing members were constructed with #2 Douglas Fir, with wall studs and top plates nominal 2 × 4 members and sill plates nominal 2 × 6 members. All studs were placed at

16 in. on center. Studs were connected to the sill plate and top plate with 0.165-in.-diameter 2–16d common nails per stud. Additional top plates are connected with 16d common nails staggered at 16 in. on center. All of the lumber used was tested for moisture content. Upon procurement of the lumber, the moisture content was between 10–25% for the Douglas Fir (studs, sill plates, top plates, and sheathing boards). The moisture contents were read before testing as well and were in the range of 5–15% for all lumber. The loss of moisture can be attributed to the walls drying while sitting out in the laboratory. All of the moisture content readings are given in Appendix A.1.

Since the boundary conditions for all eleven cripple wall specimens were the same, namely, adopting top boundary condition B and bottom boundary condition c as characterized in Phase 1, the following sections will only highlight the details pertaining to those boundary conditions. An in-depth evaluation of all boundary conditions considered in this testing program can be found in the first report [Schiller et al. 2020(a)].

Anchor bolts used were all 1/2-in. all-thread F1554 Grade 36 straight rods, with nuts and washers at both ends. The anchor bolts were not cast in the foundation but rather installed in prepared through holes to allow for ease in removal of the specimen. To accommodate this, the concrete footings were cast with 4 in. × 4 in. access holes to allow for the anchor bolts to be tightened and replaced if damaged during a test. The access holes were spaced 32 in. apart to allow for the prescribed 32 in. on center and 64 in. on center spacing of anchor bolts. The footings were cast with poured-in-place concrete, with a 28-day compressive strength target of 8 ksi. The rebar arrangement and details of the footing can be seen in Figure 2.3. Anchor bolt holes in the sill plates were oversized 1/4 in., which is a common building practice in California as it facilitates ease of construction. Square washers of size 2 in. × 2 in. × 3/16 in. overlaid with spherical washers were used at the anchor bolt connection at the sill plate, allowing for placement of 10-kip donut load cells, which were intended to measure the tensile force in the anchor bolts during testing. Conventional nuts and washers were used at the bottom anchor bolt connection within the 4 in. x 4 in. access hole. The load cell configuration can be seen in Figure 2.4.

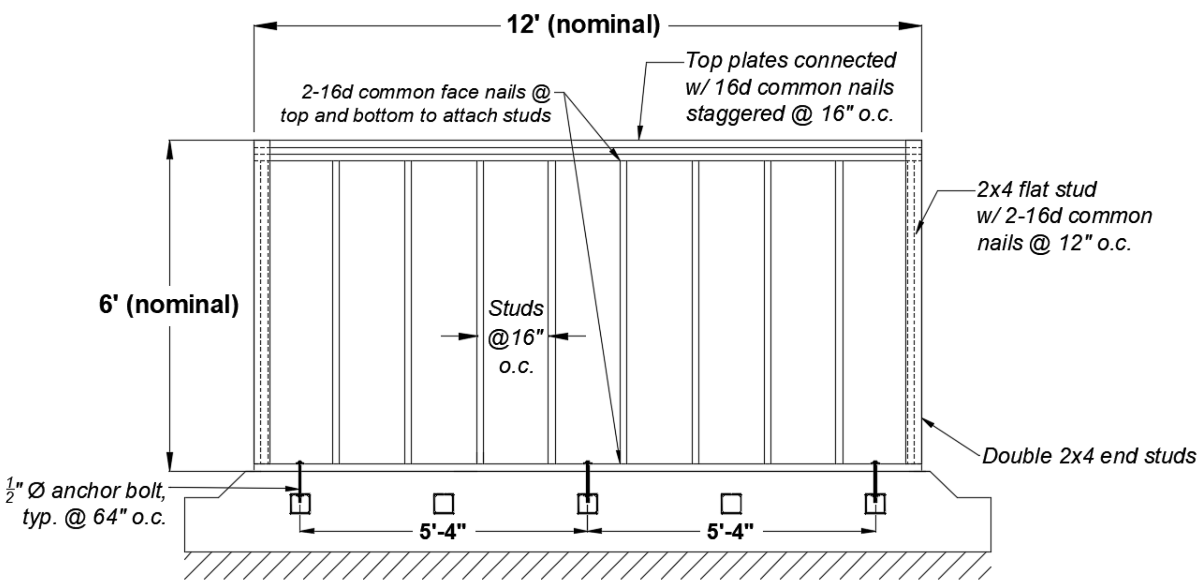
The primary resistance to sliding during imposition of lateral load to the specimen comes from the frictional resistance at the interface of the sill plate and the foundation and the bearing of the anchor bolt on the sill plate. Note that the top of the foundation had a smooth trowel finish. By oversizing the anchor bolt holes, the cripple walls have less resistance to sliding. As such, it is noted that sliding of the walls was observed for certain specimens prior to development of bearing between the anchor bolt within its hole; therefore, both the global lateral displacement response and the relative lateral displacement response are presented in Chapter 4 as they vary. The global lateral response includes the displacement of the cripple wall and the sliding of the sill plate, while the relative lateral response only considers the displacement of the actual cripple wall structure.



Notes:
1. Sill plates are 2x6 and top-plates are 2x4.
2. Anchor bolts are $\frac{1}{2}$" Ø with 2 $\frac{1}{2}$" (min.) x $\frac{3}{16}$" square washer (to receive load cell on top).

Elevation
Framing Details (Interior Face)
2' Tall Cripple Wall

Figure 2.1 2-ft-tall cripple wall framing.



Notes:
1. Sill plates are 2x6 and top-plates are 2x4.
2. Anchor bolts are $\frac{1}{2}$" Ø with 2 $\frac{1}{2}$" (min.) x $\frac{3}{16}$" square washer (to receive load cell on top).

Elevation
Framing Details (Interior Face)
6' Tall Cripple Wall

Figure 2.2 6-ft-tall cripple wall framing.

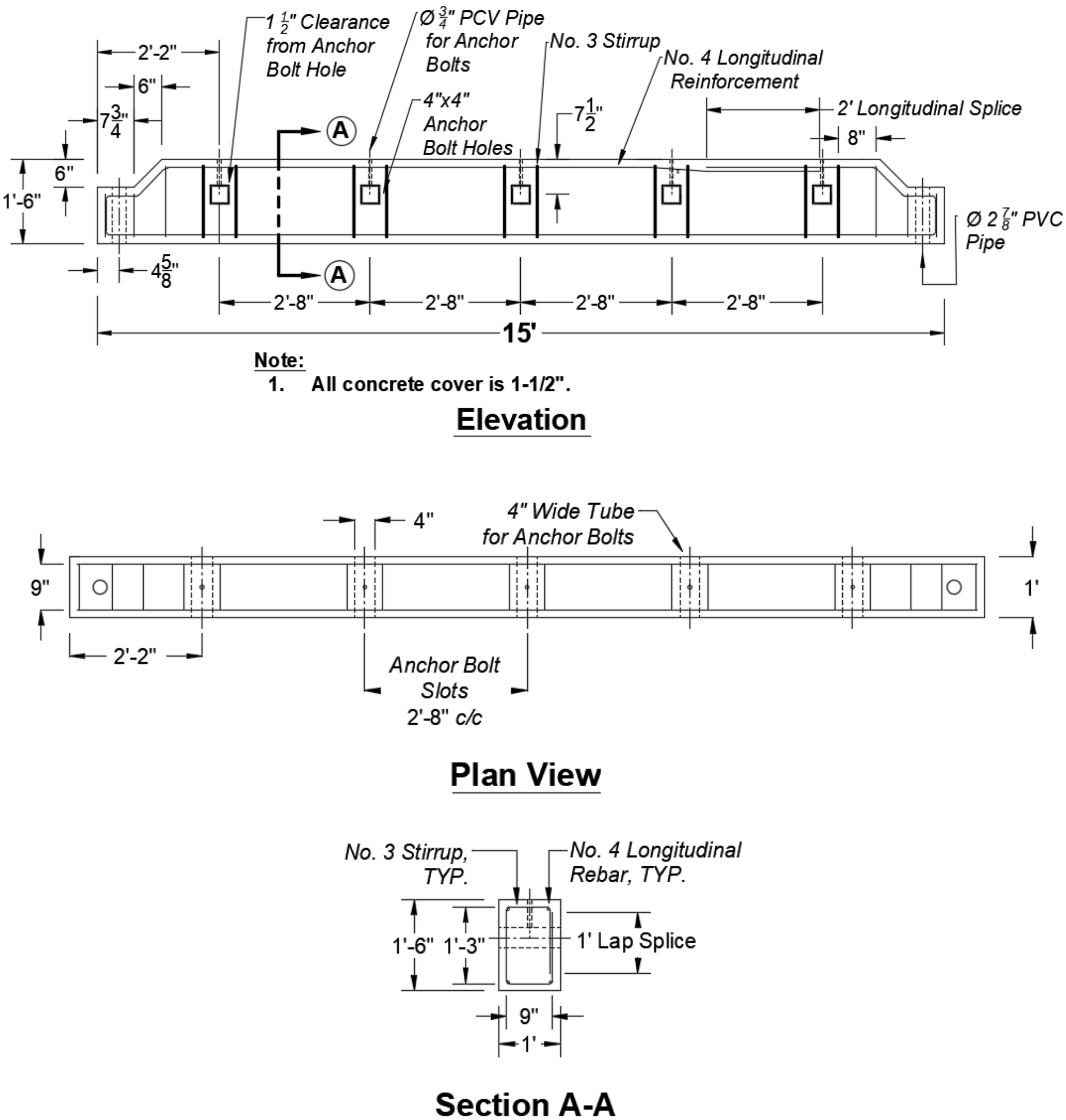


Figure 2.3 Concrete footing details for cripple wall tests.

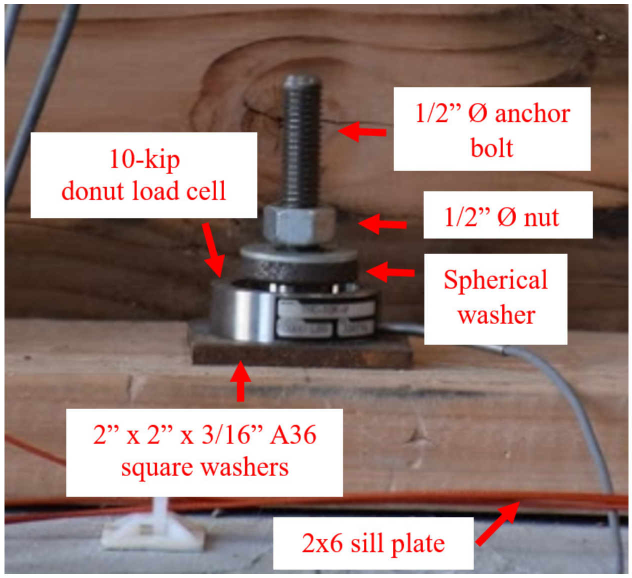


Figure 2.4 Load cell and square plate washer for anchor bolts.

The retrofit design used in this testing phase was consistent with the current recommendations from the *FEMA P-1100* prescriptive retrofit guidelines. These guidelines had not been finalized in the previous phase of testing and through the beginning on Phase 2 testing. Therefore, the retrofit designs used in Phase 1 as well as the first retrofitted specimen in Phase 2 (namely, Specimens A-5, A-8, and A-12) are slightly different from the retrofit designs in subsequent phases of testing. The primary difference between the retrofit design in the initial retrofit design and the retrofit design used in subsequent testing phases is denser edge nailing pattern [4 in. on center (o.c.) edge nailing compared to 3 in. o.c. edge nailing] and an increase in anchor bolts (5 anchor bolts compared to 7 anchor bolts). The methodology for selecting the retrofit design will be discussed in detail in Section 2.5.

2.2.2 Boundary Conditions

Framing details of the specimens were observed to be dependent upon the boundary conditions [Schiller et al. 2020(a)]. Therefore, boundary conditions were split into two categories, namely, top and bottom boundary conditions. However, based on findings of the Phase 1 tests, all specimens herein were constructed with the same boundary conditions, namely, top boundary condition B and bottom boundary condition c. It is noted that this pair of boundary conditions was adopted as the baseline condition for all specimens following Phase 1.

Top Boundary Condition B

Top boundary condition B contains built-up 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. The additional top plate was originally provided to allow for a denser furring nail arrangement at the top of the cripple wall. This detail was continued for the non-stucco specimens to maintain a uniform height for all specimens with top boundary condition B. Similar to the stucco specimens, all exterior finishes were terminated at the top of the upper top plate. The framing details for top boundary condition B can be seen in Figure 2.5. The top of the wall and corner details specific to the horizontal siding specimens can be seen in Figure 2.6. Photographs of this boundary condition for the horizontal siding finished specimens are provided in Figure 2.7. Figures 2.8 and 2.9 provide the same details for the horizontal siding over diagonal sheathing finished cripple walls, and Figures 2.10 and 2.11 show these details for the T1-11 wood structural panel finished walls. Nailing details for attachment of the exterior finishes are provided in Section 2.3.

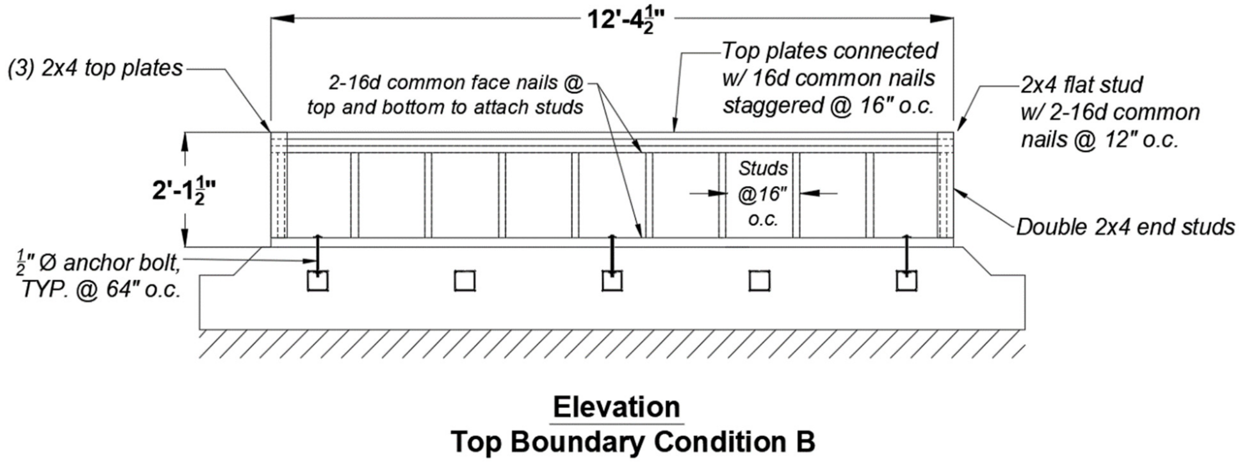
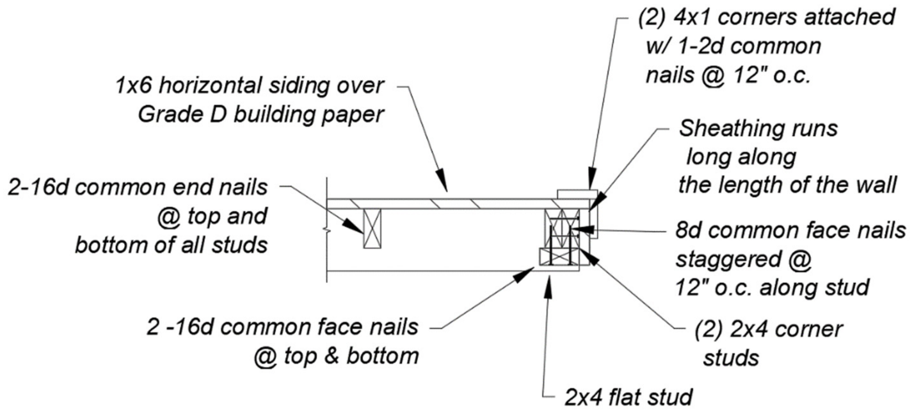
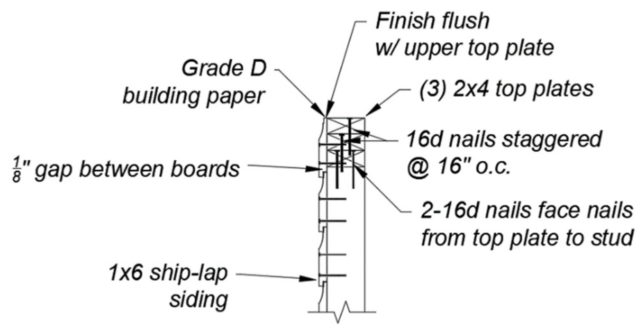


Figure 2.5 **Framing detail elevation for top boundary condition B.**



Plan View Detail
Horizontal Siding (HS)
Top Boundary Condition B



Top of Wall Detail
Horizontal Siding (HS)
Top Boundary Condition B

Figure 2.6 Corner and top of wall detail for *horizontal siding exterior finish* with top boundary condition B.

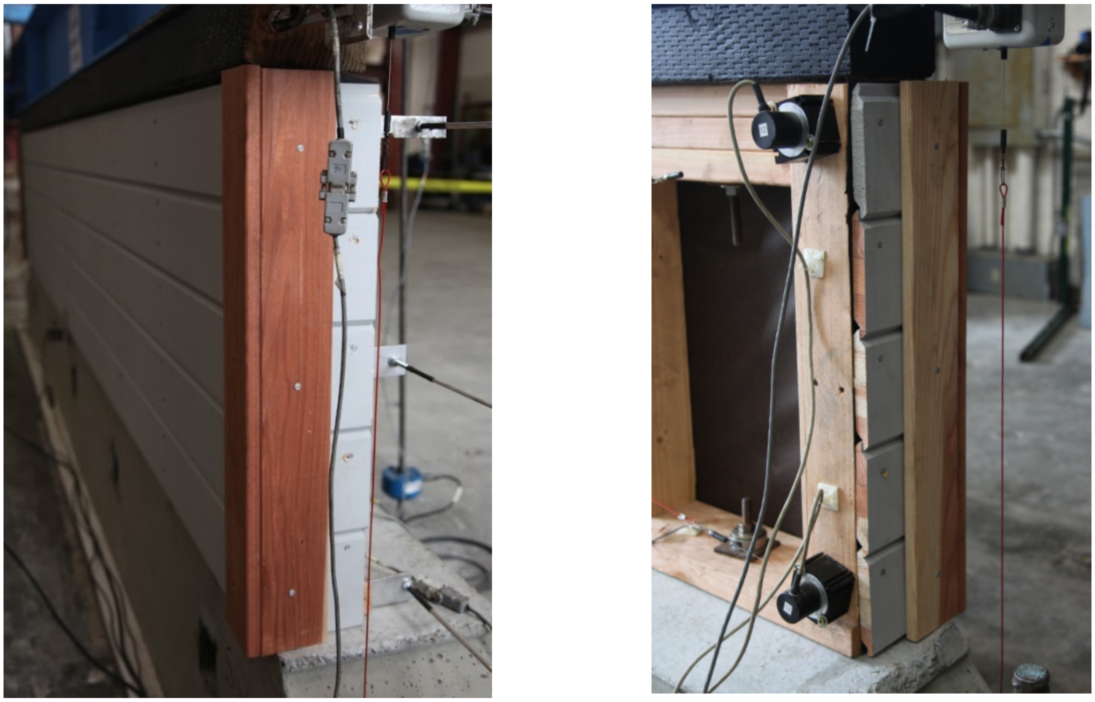
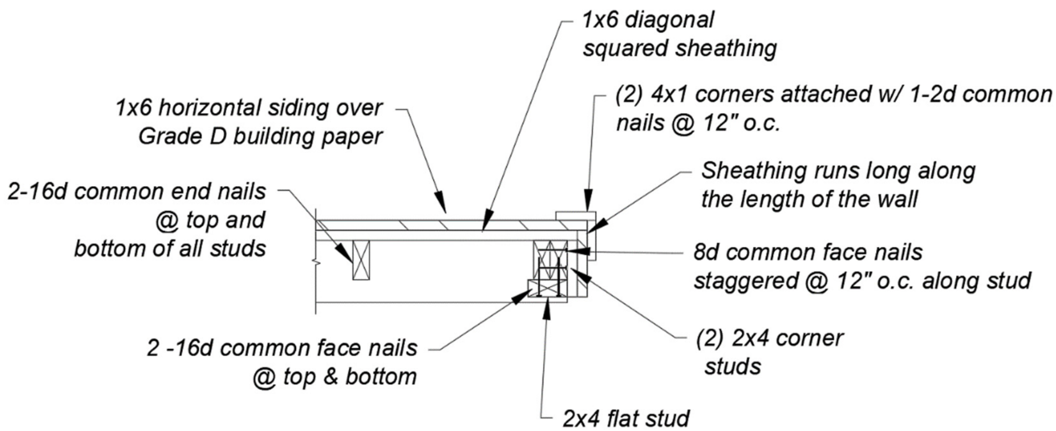
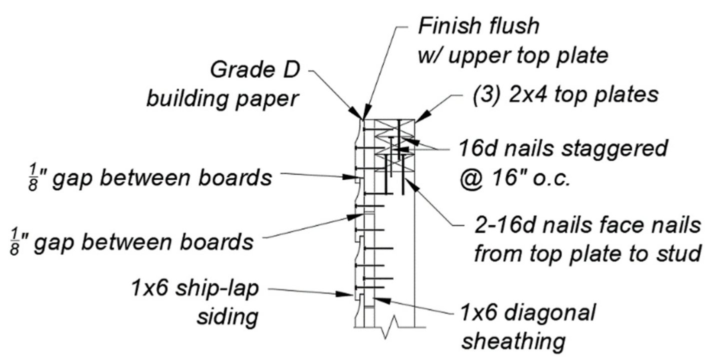


Figure 2.7 **Isometric corner views showing top boundary condition B details of horizontal siding finished cripple walls (exterior, left; interior, right).**



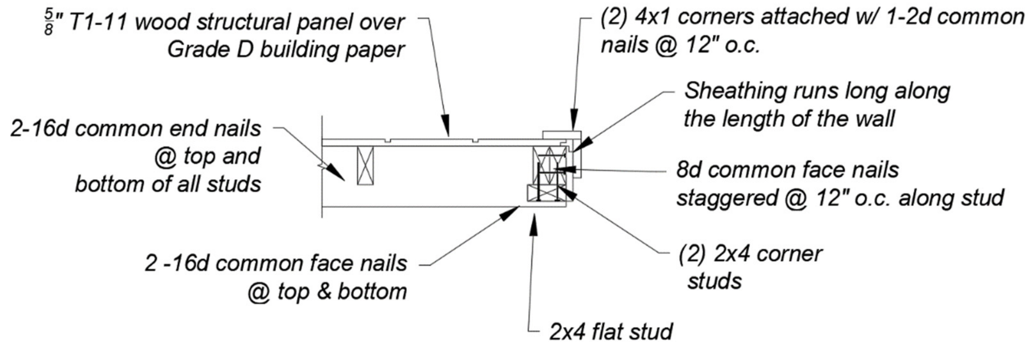
Plan View Detail
Horizontal Siding Over
Diagonal Sheathing (HS+DSh)
Top Boundary Condition B



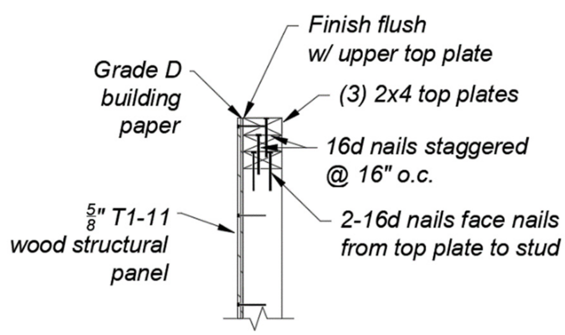
Top of Wall Detail
Horizontal Siding Over
Diagonal Sheathing (HS+DSh)
Top Boundary Condition B

Figure 2.8 **Corner and top of wall detail for horizontal siding over diagonal sheathing exterior finish with top boundary condition B.**

Figure 2.9 **Isometric corner views showing top boundary condition B details of horizontal siding over diagonal sheathing finished cripple walls (exterior, left; interior, right).**



Plan View Detail
T1-11 Wood Structural Panel (T)
Top Boundary Condition B



Top of Wall Detail
Top Boundary Condition B
T1-11 Wood Structural Panel (T)

Figure 2.10 Corner and top of wall detail for T1-11 wood structural panel exterior finish with top boundary condition B.

Figure 2.11 Isometric corner views showing top boundary condition B details of T1-11 wood structural panel finished walls (exterior, left; interior, right).

BOTTOM BOUNDARY CONDITION c

Bottom boundary condition c orientates the cripple wall so that all exterior finishes are outboard of the foundation. This is the same whether there is a combined finish material or whether there is only the presence of a siding finish. Regardless of a single or combined finish material, the innermost material is flush with the face of the footing. This is the most common condition found in California homes. This boundary condition allows all finish materials to rotate freely as the cripple wall deforms. All finishes terminate at the base of the sill plate besides horizontal siding. Due to the fixed height of the cripple walls and flush with the top of the wall alignment of the horizontal siding boards, the bottommost siding board overhangs at the base of the wall. The overhang is 1/4 in. for the 2-ft-tall specimens and 3-3/8 in. for the 6-ft-tall specimens. Figure 2.12 provides the detail of bottom boundary condition c for the horizontal siding exterior finished cripple walls, and Figure 2.13 shows a photograph of the boundary condition. The same details of the horizontal siding over diagonal sheathing finished specimens are shown in Figures 2.14 and 2.15, and in Figures 2.16 and 2.17 for the T1-11 wood structural panel finished walls. Nailing details for attachment of the exterior finishes are provided in Section 2.3.

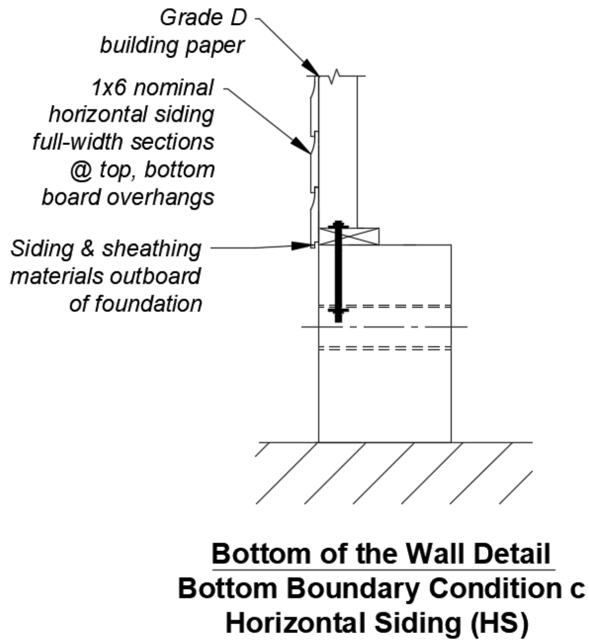


Figure 2.12 **Bottom of the wall detail for horizontal siding exterior finish bottom boundary condition c.**

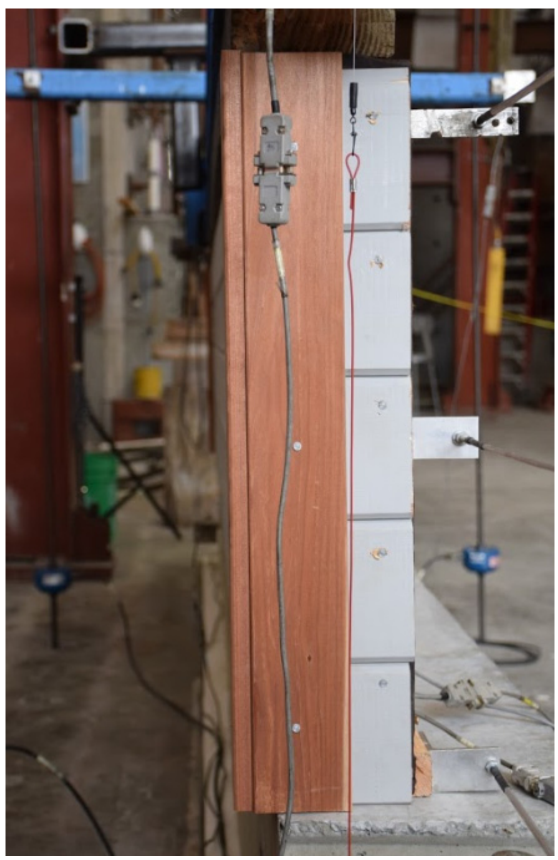


Figure 2.13 **Corner view showing horizontal siding exterior finished cripple wall with bottom boundary condition c.**

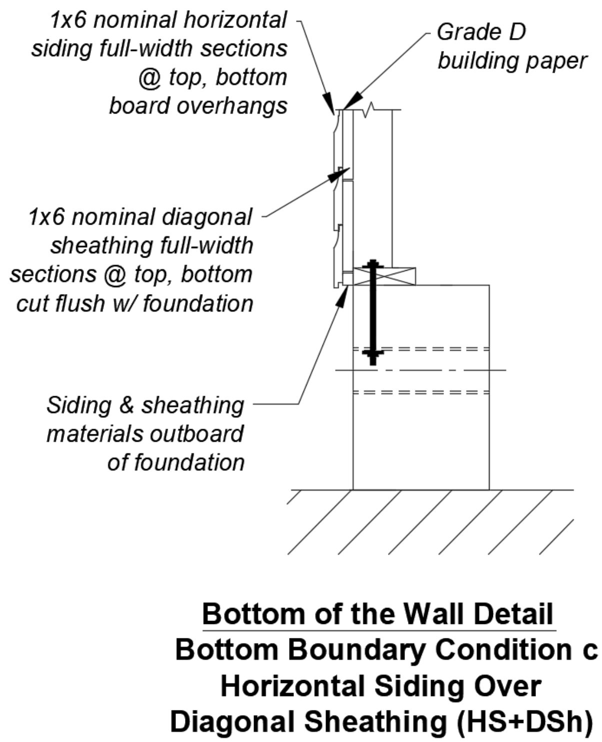


Figure 2.14 **Bottom of the wall detail for horizontal siding over diagonal sheathing exterior finish with bottom boundary condition c.**

Figure 2.15 **Corner view showing horizontal siding over diagonal sheathing exterior finished cripple wall with bottom boundary condition c.**

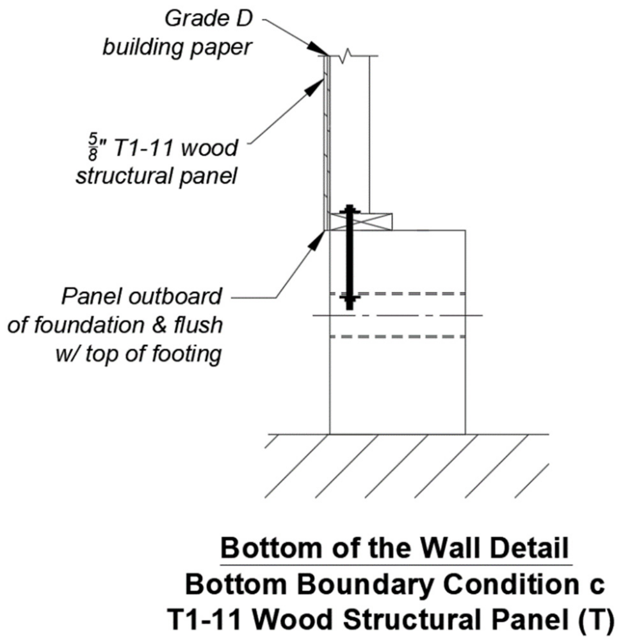


Figure 2.16 **Bottom of the wall detail for T1-11 wood structural panel exterior finish with bottom boundary condition c.**

Figure 2.17 **Corner view showing T1-11 wood structural panel finished cripple wall with bottom boundary condition c.**

2.3 INSTALLATION OF FINISHES

The eleven cripple walls discussed in this report were constructed with three different dry exterior finishes, namely: horizontal shiplap lumber siding over framing (4), horizontal shiplap lumber siding over diagonal lumber sheathing (3), and T1-11 wood structural panels (4). Of the four cripple walls constructed with horizontal siding, two of the specimens are 2 ft tall and two of the specimens are 6 ft tall. The same holds true for cripple walls constructed with T1-11 wood structural panels. All three of the horizontal siding over diagonal sheathing finished specimens were 2 ft tall.

Horizontal siding boards were shiplap 1 × 6 nominal (3/4 in. × 5½ in.), construction grade redwood. The details of the dimensions can be seen in Figure 2.18. Full siding boards were installed flush with the uppermost top plate. An 1/8–in. gap was placed between siding boards, leaving a 3/8 in. overlap between each siding board. No siding boards were trimmed at the base of the cripple wall, thus leaving an overhang for all cripple walls with horizontal siding. This culminated to 3-3/8–in. overhang for the 6-foot-tall cripple walls and a 1/8–in. overhang for the 2-ft-tall cripple walls. Grade D building paper was stapled onto the framing before installation of the siding. The horizontal siding was fastened with 2–8d nails per stud. Only the outermost end stud was nailed to the ends. The top siding board was fastened entirely to the top plates. For the 2-ft-tall specimens the bottom siding board had one nail fastened to the stud and the other fastened to the sill plate, and for the 6-ft-tall specimens only one nail was used to fasten the siding board (attached to the sill plate) due to the large overhang of the siding board. All nails used were 8d common hot-dip galvanized. Nails were spaced 3 in. apart on siding boards, centered about the middle of the board. Two pieces of 4 × 1 redwood boards were used as corner trim, which is a common aesthetic addition to cover up the corner joints of finishes. These were fastened with 2d common nails at 12 in. on center. Figure 2.19 provides details of layout of the 2-ft-tall cripple wall with horizontal siding. A photograph of the elevation and the nailing pattern is shown in Figure 2.20. The same details were provided for the 6-ft-tall cripple walls with horizontal siding and is shown in Figures 2.21 and 2.22. Since the 6-ft-tall cripple wall had a significant overhang of the bottom siding board, a photograph is provided to show this overhang in Figure 2.22. Nailing details for the construction of the horizontal siding finished specimens are provided in Figure 2.23.

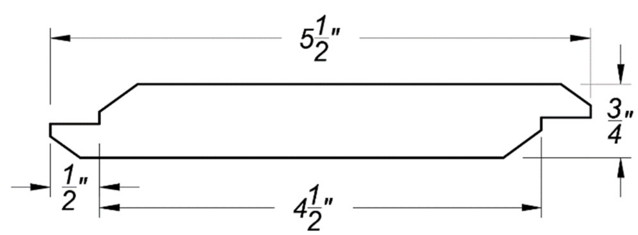
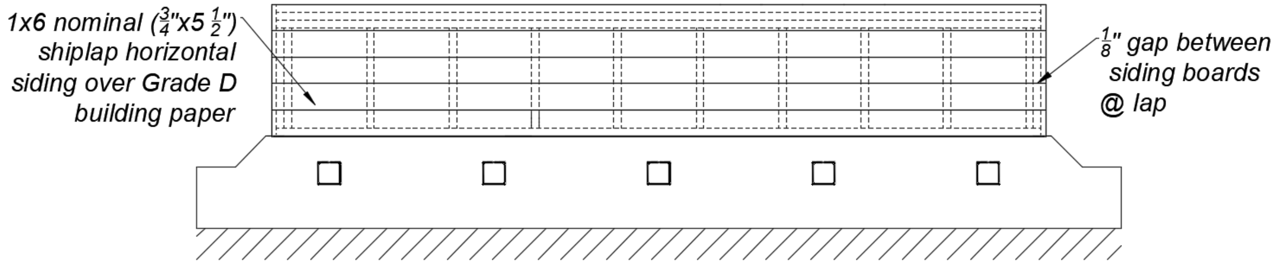


Figure 2.18 Horizontal siding board dimensions.



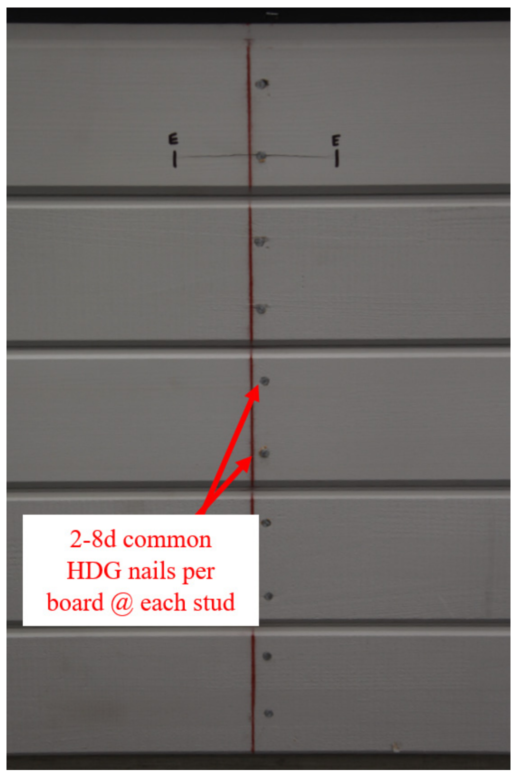
Notes:

1. Full siding boards maintained through entire height.
2. Siding is flush to uppermost top plate and outboard of footing.
3. Boards are installed from bottom to top, bottom board overhangs footing.

Elevation
Horizontal Siding (HS)
2' Tall Cripple Wall

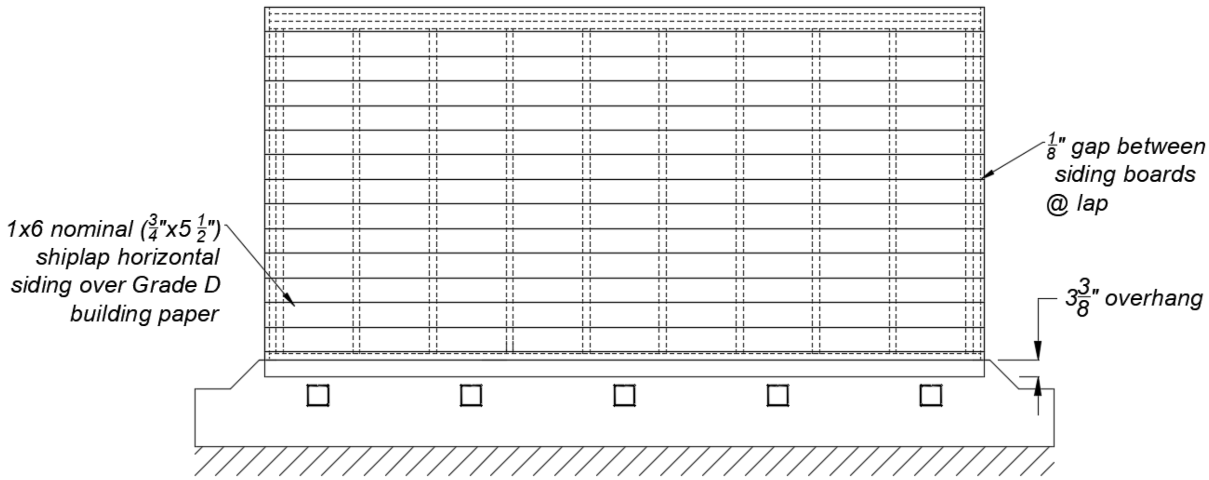
Figure 2.19 Elevation view with details for 2-ft-tall horizontal siding finished cripple wall.

(a)



(b)

Figure 2.20 Photographs of the elevation view of a 2-ft-tall cripple wall with horizontal siding finish: (a) elevation view; and (b) nailing detail.



Notes:
1. **Full siding boards maintained through entire height.**
2. **Siding is flush to uppermost top plate and outboard of footing.**
3. **Boards are installed from bottom to top, bottom board overhangs footing.**

Elevation
Horizontal Siding (HS)
6' Cripple Wall

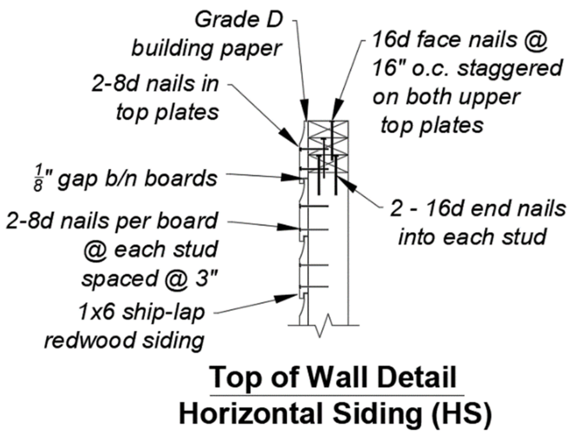
Figure 2.21 Elevation view with details for 6-ft-tall cripple wall with horizontal siding finish.

(a)

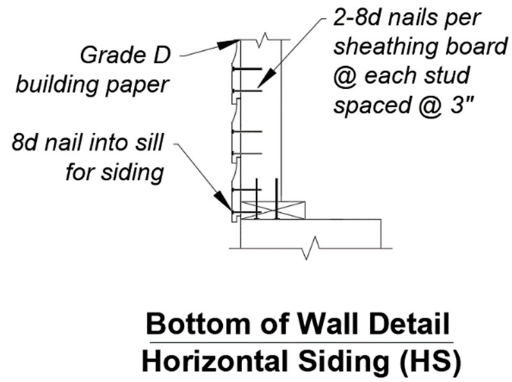
(b)

(c)

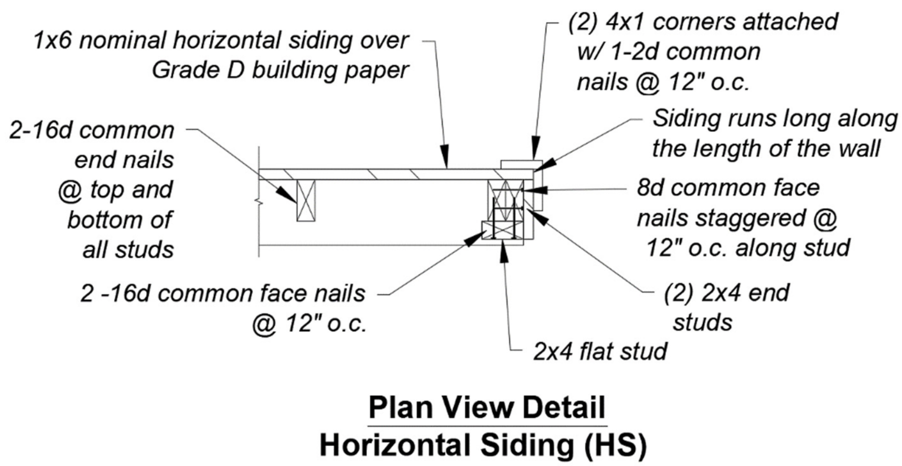
Figure 2.22 Photographs of the elevation view of a 6-ft-tall cripple wall with horizontal siding finished: (a) elevation view; (b) siding overhang; and (c) nailing detail.



(a)



(b)



(c)

Figure 2.23 Framing and finish nailing details for cripple walls with horizontal siding finish: (a) top of the cripple wall; (b) bottom of the cripple wall; and (c) plan view of corner.

Cripple walls finished with horizontal siding over diagonal sheathing were constructed with the same siding material as the horizontal siding over framing cripple walls. The diagonal sheathing boards used were square-edged, 1 × 6 nominal (7/8-in. × 5-1/2-in.) construction grade Douglas Fir. The same type of sheathing was used regardless of the sheathing orientation (horizontal or diagonal). Diagonal sheathing boards were installed at a 45° angle. A 1/8–in. gap was placed between each board using an 8d common nail to allow for expansion. Installation for the sheathing started with a full width board at one end, with each board after cut to fit onto the framing of the cripple wall. Sheathing boards were fastened to the framing with 2–8d nails per stud whereas the edge nailing was 2–8d nails per board along the sill plate, uppermost top plate, and outermost end studs. Specimens with diagonal sheathing were overlaid with Grade D building paper prior to installation of the horizontal siding boards. The horizontal siding was fastened with 2–8d nails per stud. Only the outermost end stud was nailed to the ends. The top siding board was fastened entirely to the top plates and the bottom siding board had a single nail fastened to the stud through to the sill plate. All nails used were 8d common hot-dipped galvanized. Nails were spaced around 3 in. apart on horizontal siding boards and 5 in. apart on diagonal sheathing boards. The spacing of the nails increased for diagonal sheathing boards due to the increased width of each board along the studs (5-1/2 in. for horizontal siding boards and 7-3/4 in. for diagonal sheathing boards). Nails were centered on siding and sheathing boards. Two pieces of 4 × 1 redwood boards were used as corner trim, which is a common aesthetic addition to cover up the corner joints of finishes. These were fastened with 2d common nails at 12 in. on center. Figure 2.24 shows the arrangement of both siding and sheathing materials as well as a reference to the direction of loading. The finish application sequence is shown in Figure 2.25. Nailing details horizontal siding over diagonal sheathing cripple walls can is provided in Figures 2.26 and 2.27. An additional nailing detail for the attachment of the diagonal sheathing to the framing.

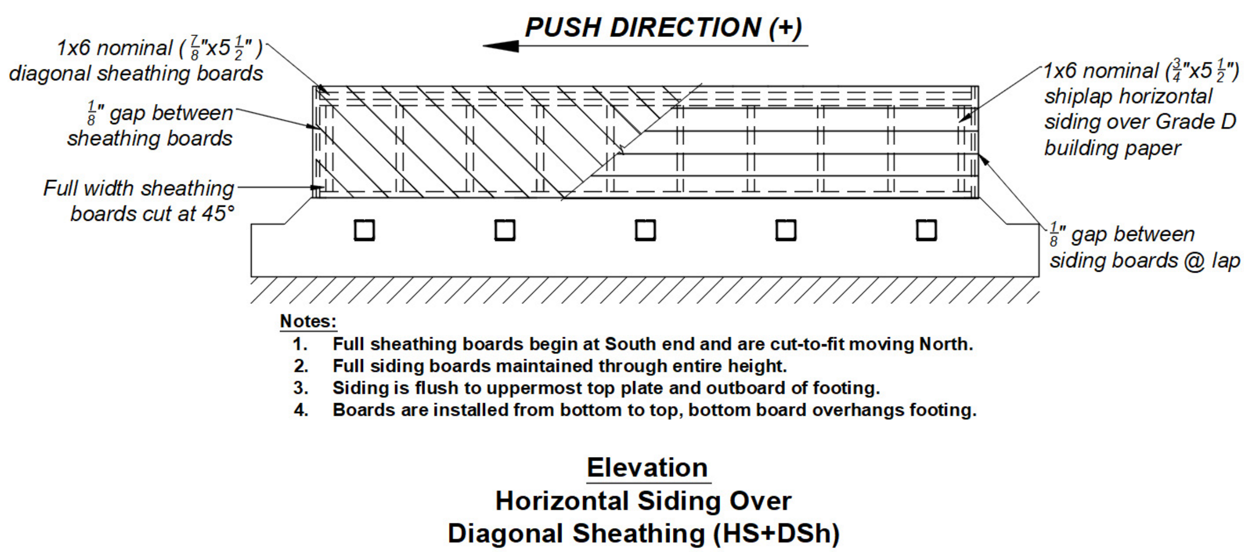
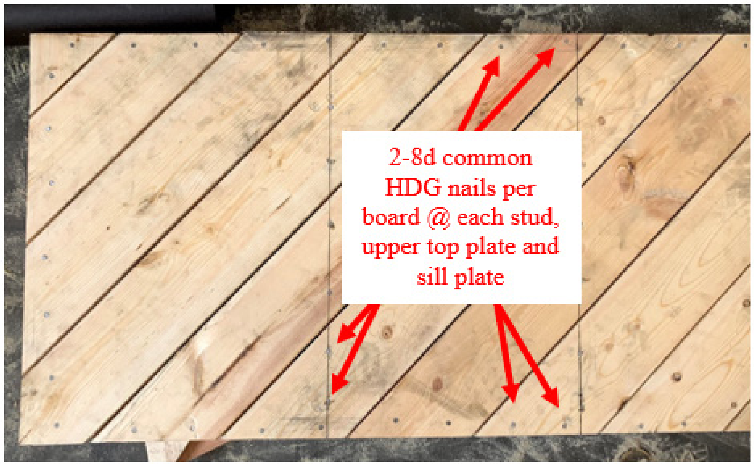


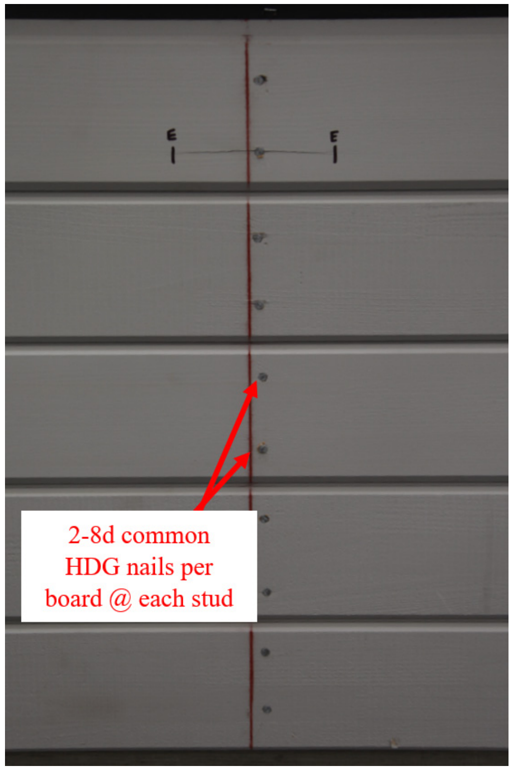
Figure 2.24 Elevation view of 2-ft-tall cripple wall with horizontal siding over diagonal sheathing finish.



(a)

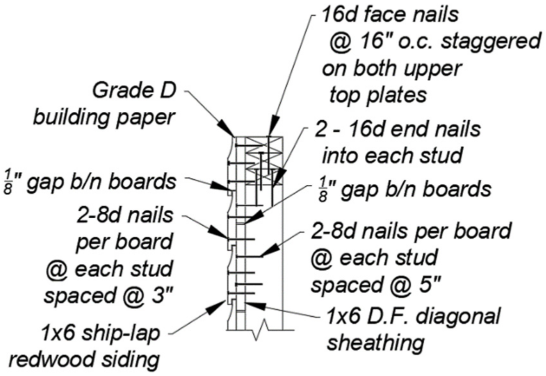
(b)

Figure 2.25 **Horizontal siding over diagonal sheathing application sequence: (a) diagonal sheathing application; (b) building paper application; and (c) horizontal siding application.**



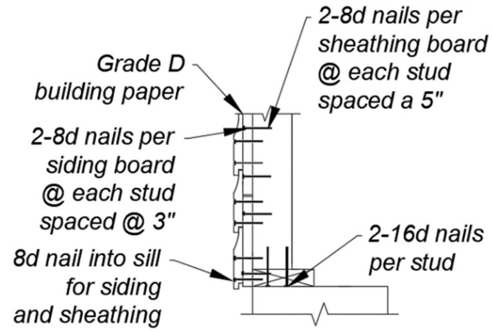
(c)

Figure 2.25 (continued).



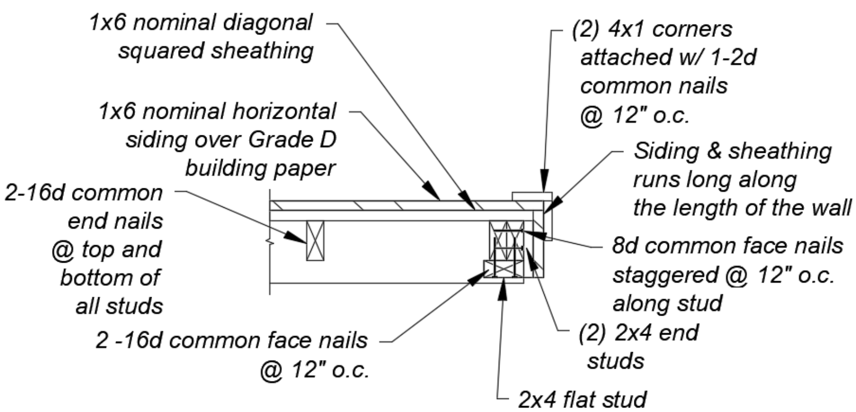
Top of Wall Detail
Horizontal Siding Over Diagonal Sheathing (HS+DSh)

(a)



Bottom of Wall Detail
Horizontal Siding Over Diagonal Sheathing (HS+DSh)

(b)



Plan View Detail
Horizontal Siding Over Diagonal Sheathing (HS+DSh)

(c)

Figure 2.26 Framing and finish nailing details for cripple walls with horizontal siding over diagonal sheathing finish: (a) top of the cripple wall; (b) bottom of the cripple wall; and (c) plan view of corner.

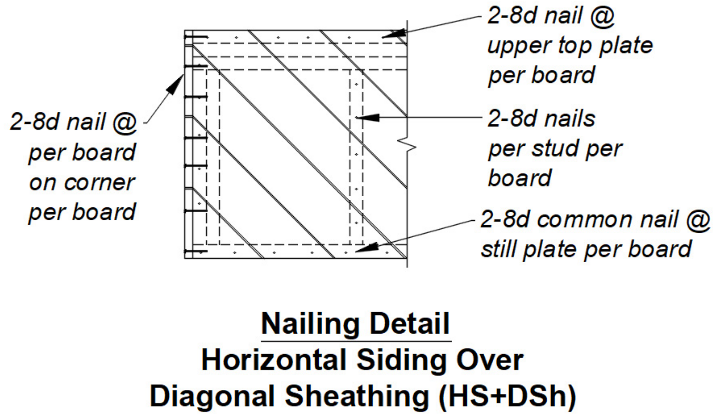
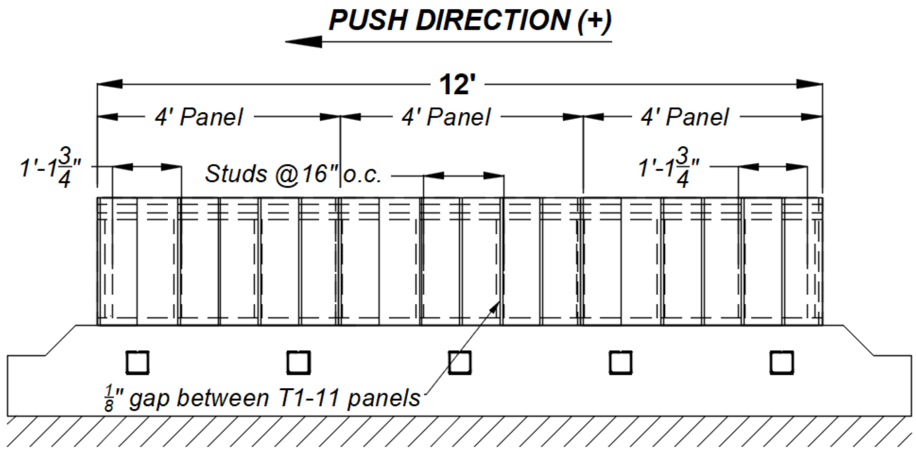


Figure 2.27 Diagonal sheathing nailing details.

The T1-11 wood structural panels used are plywood, 5/8–in.-thick × 4-ft-tall × 8-ft- wide sections, with an 8–in. on center groove pattern. Modifications were made to the length of the T1-11 cripple walls (12 ft-4-1/2 in. to 12 ft) to fit three full 4-ft sections along the cripple wall. This was done by shortening the two stud bays at the ends from 16 in. to 13-3/4 in., thus keeping the cripple wall symmetric. A 1/8–in. gap was placed between panels to allow for expansion. An elevation view of the T1-11 finished 2-ft-tall cripple walls can be seen in Figure 2.28 and a corresponding photograph is shown in Figure 2.29. The same detail and photograph for the T1-11 finished 6-ft-tall specimens can be found in Figures 2.30 and 2.31, respectively.

The T1-11 panels were fastened with 8d common, hot-dipped galvanized nails, edge nailed at 8 in. on center and field nailed at 12 in. on center for the existing case and edge-nailed at 4 in. on center and field-nailed at 12 in. on center for the retrofitted case (retrofit details will be discussed later). For the existing case, the T1-11 installation process entailed fastening one T1-11 panel with nails along three edges; see Figure 2.28. The un-nailed edge was secured by putting the next T1-11 panel in place and beginning the same nailing pattern on the next panel. As is common practice, the underlying edge of each T1-11 panel is not nailed but is secured by being pinched by the overlapping panel, as shown in Figures 2.34 and 2.35. Similar to other dry finishes, two pieces of 4 × 1 redwood boards were used as corner trim. Figure 2.32 through 2.34 provide construction details for the installation of the T1-11 wood structural panels.



Notes:

1. Full $\frac{5}{8}$"x4' T1-11 Panels used throughout (wall length made to accommodate).
2. T1-11 Panels are ship-lapped $\frac{3}{8}$" maintaining a $\frac{1}{8}$" gap for expansion.
3. Panels are installed from left to right.

Elevation
T1-11 Wood Structural Panel (T)
2' Cripple Wall

Figure 2.28 Elevation view with details for a 2-ft-tall cripple wall with T1-11 wood structural panel finish.

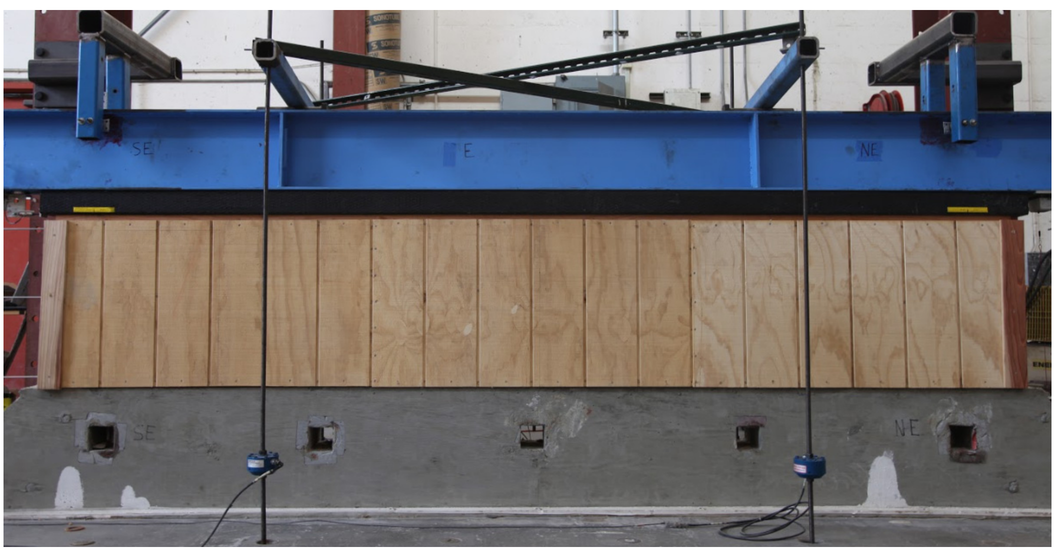


Figure 2.29 Photograph of elevation view of a 2-ft-tall cripple wall with T1-11 wood structural panel finish.

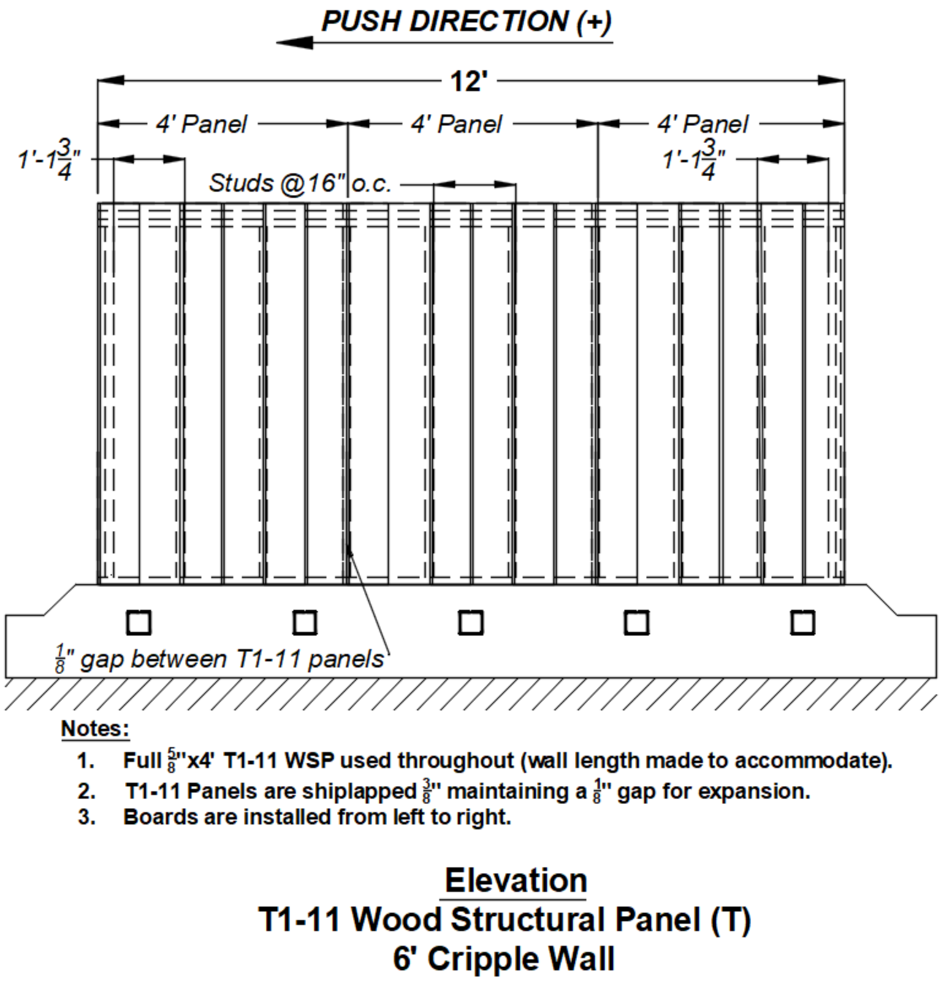
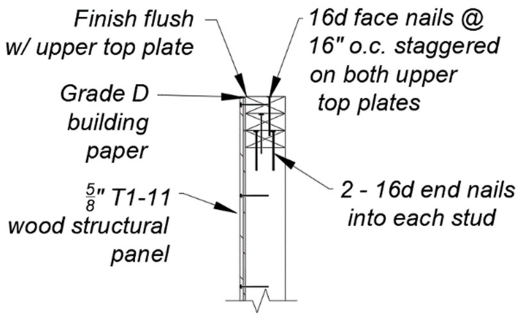


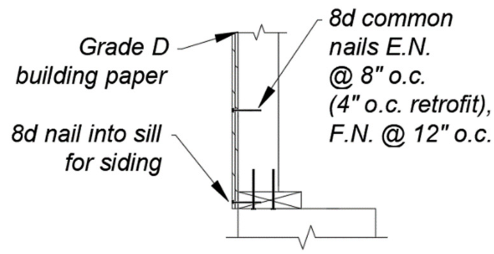
Figure 2.30 **Elevation view with details for a 6-ft-tall cripple wall with T1-11 wood structural panel finish.**

Figure 2.31 Photograph of elevation view of a 6-ft-tall cripple wall with T1-11 wood structural panel finish.



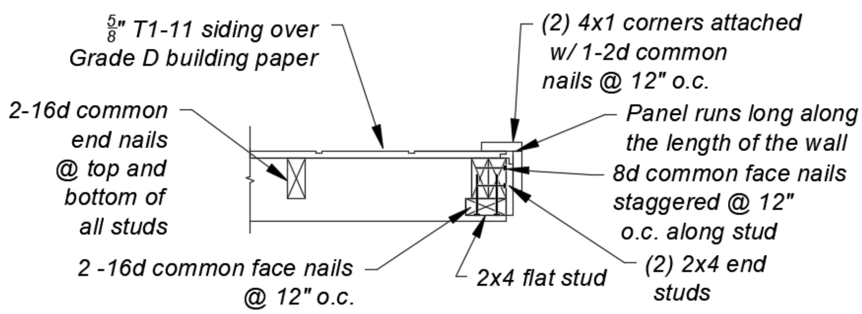
Top of Wall Detail
T1-11 Wood Structural Panel (T)

(a)



Bottom of Wall Detail
T1-11 Wood Structural Panel (T)

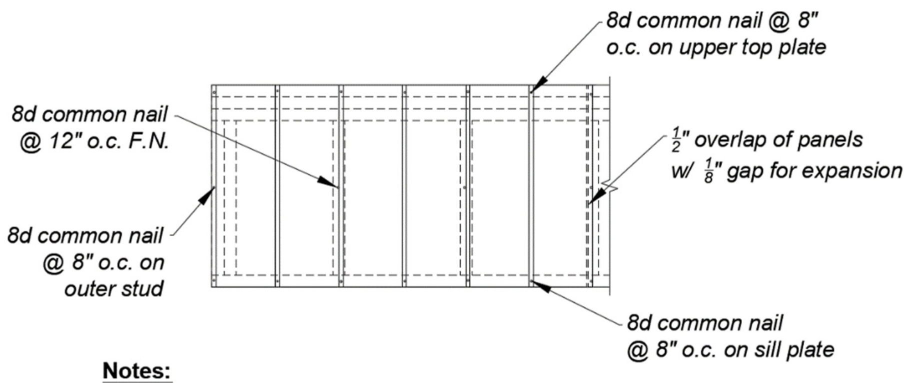
(b)



Plan View Detail
T1-11 Wood Structural Panel (T)

(c)

Figure 2.32 Framing and finish nailing details for the cripple walls with T1-11 wood structural panel finish: (a) top of the cripple wall; (b) bottom of the cripple wall; and (c) plan view of corner.

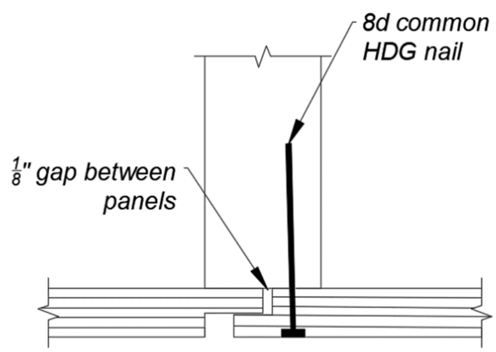


Notes:

1. $6\frac{1}{2}$" spacing used for first 2 nails on upper top plate and sill plate in order to align nails on stud @ 8" spacing.

Nailing Detail
Existing T1-11 Wood Structural Panel (T)

Figure 2.33 Nailing detail for existing T1-11 cripple wall.



Nailing Detail
Stud - Plywood Joint
Existing T1-11 Wood Structural Panel (T)

Figure 2.34 Nailing detail of T1-11 panel overlap for existing T1-11 cripple wall.

Figure 2.35 Existing T1-11 cripple wall nailing pattern.

2.4 RETROFIT DESIGN AND INSTALLATION

Five of the eleven specimens were retrofitted. Each retrofitted cripple wall has an existing specimen identical in every way besides the addition of the retrofit. The cripple wall retrofit was designed in accordance with the *FEMA P-1100* prescriptive design provisions. In practice, the *FEMA P-1100* prescriptive design provisions are chosen based on the weight classification, number of stories, height of cripple wall, and square footage of the floor plan, as well the seismicity of dwelling’s location. The weight classification is a factor of the materials in the exterior finish, interior finish, and roofing. This produces a light, medium, or heavy weight classification. The flow chart used to determine the weight classification can be seen in Figure 2.36. With the weight classification determined, the length of plywood, number of anchor bolts, plywood edge nailing spacing, and number of shear clips are then determined, based also on the number of stories, square footage, height of cripple wall, S_{DS} of the dwelling, and the presence of tie-downs. The table used for determining the retrofit design can be seen in Tables 2.2 (for a 2-ft-tall cripple wall) and 2.3 (for a 6-ft-tall cripple wall). The length of plywood, number of anchor bolts, plywood edge nailing spacing, and number of shear clips produced from the table are what is required for each perimeter wall line.

The retrofit design used for cripple wall specimens described herein was based on a model dwelling with plan dimensions of 30 ft × 40 ft. This floor plan was chosen to be in line with the index building used in the ATC-110 project [ATC 2014]. Therefore, for the retrofit design, the model building was assumed to be two stories tall and 2400 ft^2. For ten of the eleven tests, a heavy gravity load of 450 plf was used with the intention of simulating the gravity weight of two stories above the cripple wall, in addition to heavy building materials. One test had a light gravity load of 150 plf, intended to represent a one-story dwelling above the cripple wall and light building materials. This test did not have a retrofit companion specimen. The short-period design spectral response factor, S_{DS}, was assumed to be 1.0*g*. A value of 1.0*g* for S_{DS} is representative of a highly seismic area with ordinary fault conditions–not near-fault conditions. This aligns with the design of the loading protocol used in all tests discussed in this report [Zareian and Lanning 2020]. Lastly, three of the cripple walls were 2 ft tall and two of the cripple walls were 6 ft tall. Therefore, Table 2.2 shows the retrofit design provisions for the 2-ft-tall cripple walls and Table 2.3 shows the retrofit design provisions for the 6-ft-tall cripple walls. Note that in the case of the 6-ft-tall cripple walls, with the exception of the T1-11 6-ft-tall walls, tie-downs were utilized to transfer the large end wall tension forces.

From the table, the row representing two-story heavy construction for a 2400 ft^2 dwelling was used. The square footage is based on two stories with 1200 ft^2. For the 2-ft-tall cripple walls, 12 ft of wood structural panels, edge nailed at 2 in. on center, was required for a perimeter wall line. The retrofit design used consisted of fully sheathed walls with 15/32-in.-thick plywood, edge nailed at 3 in. on center, which essentially provided the same capacity as what the *FEMA P-1100* retrofit prescribed. This modification was chosen to sheath the full length of the specimens. From Tables 2.2 and 2.3, 21 all-thread, 1/2-in. anchor bolts were required along the perimeter wall, which was 40 ft in length for the model dwelling considered. For the 12-ft section of wall tested, five anchor bolts were used. In addition, *FEMA P-1100* requires an extra anchor bolt at each end of the cripple wall. Five anchor bolts were slotted into the pre-existing anchor bolt slots on the foundation, spaced at 32 in. on center, and the additional two anchor bolts were embedded 10 in. into the foundation and epoxied with Simpson Strong-Tie SET-XP, 12 in. inward from the outer two most anchor bolts, as shown in Figure 2.37.

For the 6-ft-tall cripple walls with tie-downs, 13 ft and 3 in. of wood structural panels, edge nailed at 2 in. on center, were required for a perimeter wall line. Again, this modification was intended to sheath the full wall, which changed the edge nail spacing to 3 in. on center. Therefore, the 6-ft-tall cripple wall was fully sheathed with 15/32-in.-thick plywood, edge nailed at 3 in. on center. As with the 2-ft-tall specimens, 21 anchor bolts along the perimeter wall were required as per *FEMA P-1100*. For the test specimen, five of the anchor bolts were required along with the two additional anchor bolts at each end of the wall. Due to the geometry of the cripple wall and foundation, the location of the anchor bolts attached to the tie-downs did not align with the anchor bolt slots on the foundation. Therefore, these anchor bolts, as well as the additional anchor bolt added at each end, were embedded 10 in. into the foundation and epoxied into place, as shown in Figure 2.39. The remaining three anchor bolts were slotted into the pre-existing anchor bolt slots on the foundation and spaced at 32 in. on center.

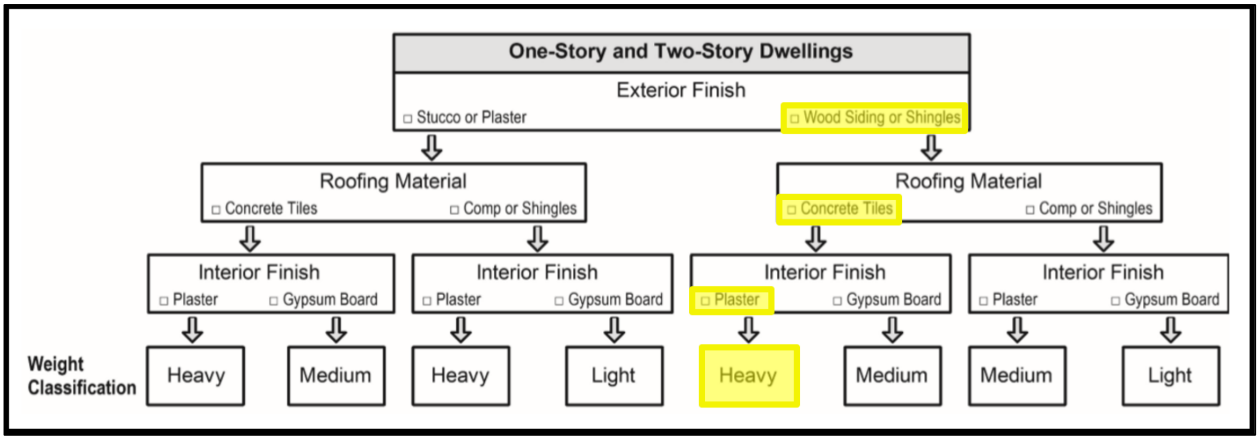


Figure 2.36 ***FEMA P-1100*** **weight classification flow chart.**

Table 2.2 ***FEMA P-1100*** **design provisions for 2-ft-tall cripple wall retrofit.**

EARTHQUAKE RETROFIT SCHEDULE (S_{DS}= 1.0 Seismic) TWO-STORY																			
Weight Category	Total Area in Square Feet	☒ Mark row that applies	Length Each of Two Braced Wall Sections Required Along Each Perimeter Wall Line									Number of Foundation Connectors or Anchors at Each Perimeter Wall Line Assume Distributed Along Length							
			Plywood Bracing Panels									Foundation Sill Anchors					Floor to Cripple Wall or Floor to Foundation Sill		
			Cripple Wall Height								Plywood Edge Nailing								
			up to 1'	1'-1" to 2'	2'-1" to 4'-0"		4'-1" to 6'-0"		6'-1" to 7'-0"										
			Without Tie-downs	Without Tie-downs	Without Tie-downs	With Tie-downs	Without Tie-downs	With Tie-downs	Without Tie-downs	With Tie-downs		Type "A"	Type "B"	Type "C"	1/2"ø Bolt	5/8"ø Bolt	Type "D"	Type "E" or "F"	Type "G"
2-Story Light Construction	up to 1600		8.0'	8.0'	10.7'	8.0'	12.0'	9.3'	13.3'	9.3'	4"	7	10	11	11	8	17	17	22
	1601 to 2000		9.3'	9.3'	12.0'	9.3'	13.3'	10.7'	14.7'	10.7'	4"	8	12	13	13	9	20	19	26
	2001 to 2400		10.7'	10.7'	13.3'	10.7'	14.7'	10.7'	16.0'	12.0'	4"	9	14	15	15	10	23	22	29
	2401 to 3000		12.0'	12.0'	14.7'	12.0'	17.3'	13.3'	18.7'	13.3'	4"	10	16	18	18	12	27	26	34
	3001 to 4000		14.7'	14.7'	17.3'	16.0'	20.0'	16.0'	21.3'	16.0'	4"	13	20	22	22	15	34	32	43
2-Story Medium Construction	up to 1600		8.0'	9.3'	10.7'	8.0'	13.3'	9.3'	13.3'	10.7'	3"	7	11	12	12	9	19	18	24
	1601 to 2000		9.3'	10.7'	12.0'	9.3'	14.7'	10.7'	14.7'	12.0'	3"	9	13	15	15	10	22	22	28
	2001 to 2400		9.3'	10.7'	13.3'	10.7'	16.0'	12.0'	16.0'	13.3'	3"	10	15	17	17	11	26	25	32
	2401 to 3000		10.7'	12.0'	14.7'	12.0'	17.3'	13.3'	18.7'	14.7'	3"	12	18	20	20	14	30	29	39
	3001 to 4000		13.3'	14.7'	17.3'	13.3'	20.0'	16.0'	21.3'	17.3'	3"	14	23	25	25	17	38	36	48
2-Story Heavy Construction	up to 1600		9.3'	9.3'	12.0'	9.3'	13.3'	10.7'	14.7'	12.0'	2"	9	14	16	16	11	24	23	30
	1601 to 2000		9.3'	10.7'	13.3'	10.7'	14.7'	12.0'	16.0'	13.3'	2"	11	17	18	18	13	28	27	35
	2001 to 2400		10.7'	12.0'	14.7'	10.7'	16.0'	13.3'	17.3'	14.7'	2"	12	19	21	21	14	32	31	41
	2401 to 3000		12.0'	13.3'	16.0'	13.3'	18.7'	14.7'	18.7'	16.0'	2"	14	23	25	25	17	38	37	48
	3001 to 4000		13.3'	16.0'	18.7'	14.7'	21.3'	17.3'	22.7'	18.7'	2"	18	28	31	31	21	48	46	60

Table 2.3 ***FEMA P-1100*** **design provisions for 6-ft-tall cripple wall retrofit.**

EARTHQUAKE RETROFIT SCHEDULE (S_{DS}= 1.0 Seismic) TWO-STORY																		
Weight Category	Total Area in Square Feet	☒ Mark row that applies	Length Each of Two Braced Wall Sections Required Along Each Perimeter Wall Line									Number of Foundation Connectors or Anchors at Each Perimeter Wall Line Assume Distributed Along Length						
			Plywood Bracing Panels									Foundation Sill Anchors					Floor to Cripple Wall or Floor to Foundation Sill		
			Cripple Wall Height								Plywood Edge Nailing								
			up to 1'	1'-1" to 2'	2'-1" to 4'-0"		4'-1" to 6'-0"		6'-1" to 7'-0"										
			Without Tie-downs	Without Tie-downs	Without Tie-downs	With Tie-downs	Without Tie-downs	With Tie-downs	Without Tie-downs	With Tie-downs		Type "A"	Type "B"	Type "C"	1/2"ø Bolt	5/8"ø Bolt	Type "D"	Type "E" or "F"	Type "G"
2-Story Light Construction	up to 1600		8.0'	8.0'	10.7'	8.0'	12.0'	9.3'	13.3'	9.3'	4"	7	10	11	11	8	17	17	22
	1601 to 2000		9.3'	9.3'	12.0'	9.3'	13.3'	10.7'	14.7'	10.7'	4"	8	12	13	13	9	20	19	26
	2001 to 2400		10.7'	10.7'	13.3'	10.7'	14.7'	10.7'	16.0'	12.0'	4"	9	14	15	15	10	23	22	29
	2401 to 3000		12.0'	12.0'	14.7'	12.0'	17.3'	13.3'	18.7'	13.3'	4"	10	16	18	18	12	27	26	34
	3001 to 4000		14.7'	14.7'	17.3'	16.0'	20.0'	16.0'	21.3'	16.0'	4"	13	20	22	22	15	34	32	43
2-Story Medium Construction	up to 1600		8.0'	9.3'	10.7'	8.0'	13.3'	9.3'	13.3'	10.7'	3"	7	11	12	12	9	19	18	24
	1601 to 2000		9.3'	10.7'	12.0'	9.3'	14.7'	10.7'	14.7'	12.0'	3"	9	13	15	15	10	22	22	28
	2001 to 2400		9.3'	10.7'	13.3'	10.7'	16.0'	12.0'	16.0'	13.3'	3"	10	15	17	17	11	26	25	32
	2401 to 3000		10.7'	12.0'	14.7'	12.0'	17.3'	13.3'	18.7'	14.7'	3"	12	18	20	20	14	30	29	39
	3001 to 4000		13.3'	14.7'	17.3'	13.3'	20.0'	16.0'	21.3'	17.3'	3"	14	23	25	25	17	38	36	48
2-Story Heavy Construction	up to 1600		9.3'	9.3'	12.0'	9.3'	13.3'	10.7'	14.7'	12.0'	2"	9	14	16	16	11	24	23	30
	1601 to 2000		9.3'	10.7'	13.3'	10.7'	14.7'	12.0'	16.0'	13.3'	2"	11	17	18	18	13	28	27	35
	2001 to 2400		10.7'	12.0'	14.7'	10.7'	16.0'	13.3'	17.3'	14.7'	2"	12	19	21	21	14	32	31	41
	2401 to 3000		12.0'	13.3'	16.0'	13.3'	18.7'	14.7'	18.7'	16.0'	2"	14	23	25	25	17	38	37	48
	3001 to 4000		13.3'	16.0'	18.7'	14.7'	21.3'	17.3'	22.7'	18.7'	2"	18	28	31	31	21	48	46	60

The two exceptions to this anchor bolt layout occurred with the 2-ft-tall cripple wall with horizontal siding exterior finish and the 2-ft-tall cripple wall with T1-11 wood structural panels. The horizontal siding specimen used five anchor bolts spaced at 32 in. on center, as shown in Figure 2.37, which is the same as Specimen A-5 (the 2-ft-tall cripple wall with stucco over horizontal sheathing exterior finish). The reason for the difference was that the *FEMA P-1100* guidelines had not been finalized at the time of testing for both of these specimens. In Appendix A.3, the calculations for determining the retrofit design for Specimen A-8 are provided. For the 2-ft-tall cripple wall with T1-11 wood structural panel, no additional retrofit was required as per *FEMA P-1100* Table 8.3-1 once the nail spacing was decreased from 8 in. on center to 4 in. on center along the edges of the panels. The minimum nailing requirements for the T1-11 wood structural panels is 8d common nails at 6 in. on center along the edges and 12 in. on center over the field. The anchor bolt arrangement did not change from its existing counterpart.

Prior to sheathing, 2 × 4 blocking was attached to the sill plate with 6-10d common nails per stud bay. For most cripple wall retrofits, split blocking was used with 6-10d common nails per stud bay. Split blocking involves using two 2 × 4 sections of blocking instead of a full 2 × 4 section to fill the entire stud bay. With this configuration, all of the anchor bolts rested on the sill plate. Split blocking is only used in stud bays containing an anchor bolt. The exceptions to the split blocking were for both 6-ft-tall cripple wall with horizontal siding and both T1-11 finished cripple walls. Full blocking was used for the 6-ft-tall cripple wall with horizontal siding. The use of 6-10d nails per block is an increase from the *FEMA P-1100* minimum requirement of 4–10d nails per block. An increase in nails used was to reduce the chance of the smaller split blocking sections from splitting during testing. For the T1-11 cripple walls, no blocking was required as the retrofit guidelines do not require plywood to be attached to the interior of the cripple wall. Since T1-11 is a wood structural panel like plywood, the retrofit instead required a reduction in nail spacing along the edges of the panels.

To accommodate the retrofit, additional 4 × 4 end studs were toe-nailed in the interior framing space, with common nails top and bottom at each end of the wall, and two interior 4 × 4 studs were toe-nailed in with 2–8d common nails top and bottom at each interior third. The addition of studs and blocking plates were used to allow the plywood panels to be nailed to the cripple wall. The interior of the framing before the application of plywood for a retrofitted specimen is shown in Figure 2.41 (a) and (b). The plywood used was 15/32-in.-thick Grade 32/16 plywood and was placed in three 4-ft sections, fully sheathing the interior face of the wall. Panels were attached with 8d common nails at 3 in. on center along the edges and 12 in. on center along the field. A 1/8-in. gap was left between panels to allow for expansion, and the nails were placed ¾ in. from the panel edge to prevent from nails tearing through the panel edges, as shown in Figure 2.41. Plywood panels terminate at the top of the middle top plate. For the 6-ft-tall cripple wall with horizontal siding, Simpson Strong-Tie HDU4-SDS2.5HDG hold-downs were used for the tie-downs at both ends. The tie-downs were hot-dip galvanized and fastened with six ¼-in. × 2-1/2 in. Strong-Drive SDS screws into the end studs; see Figure 2.42.

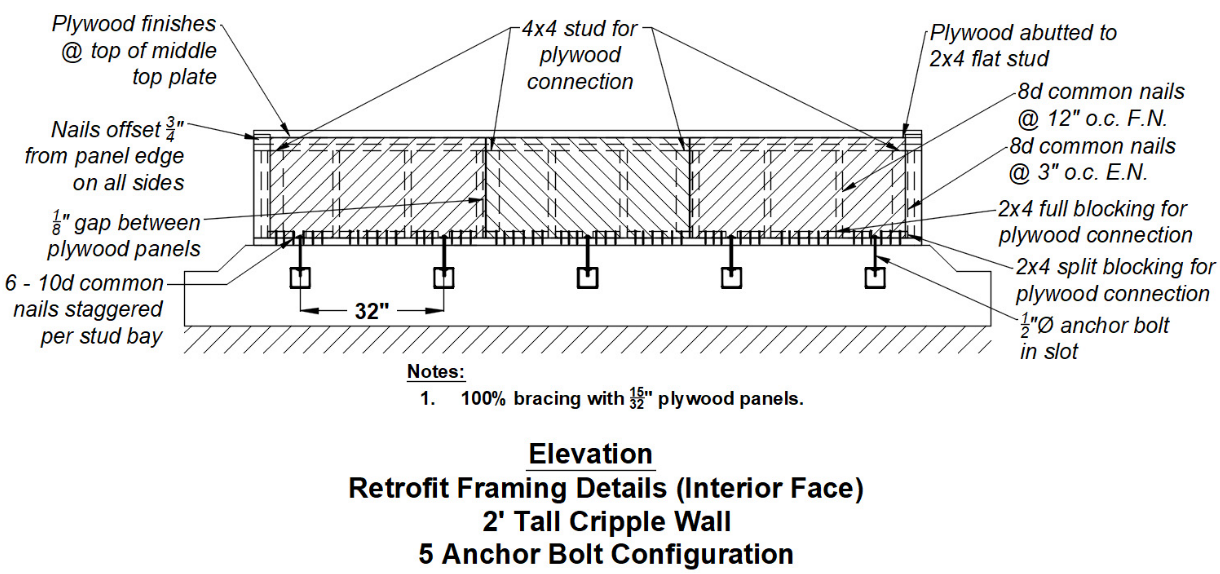
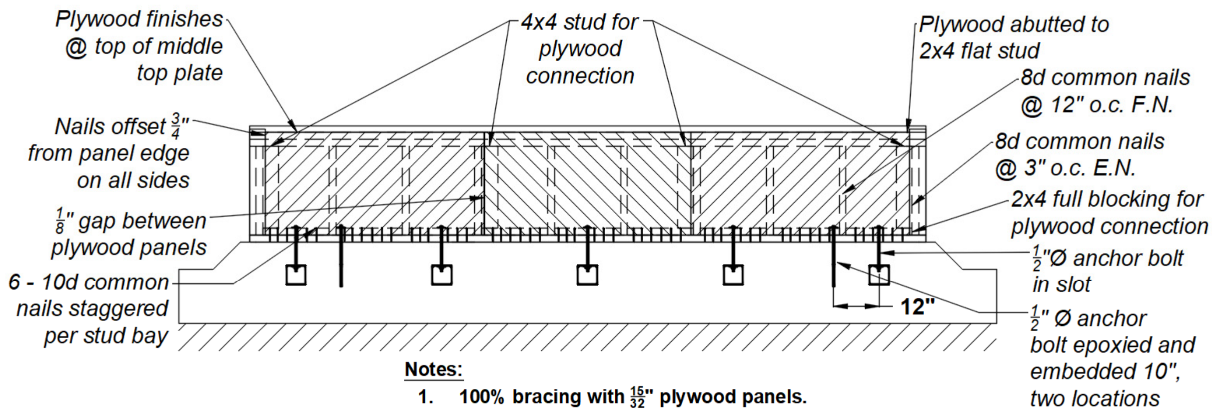
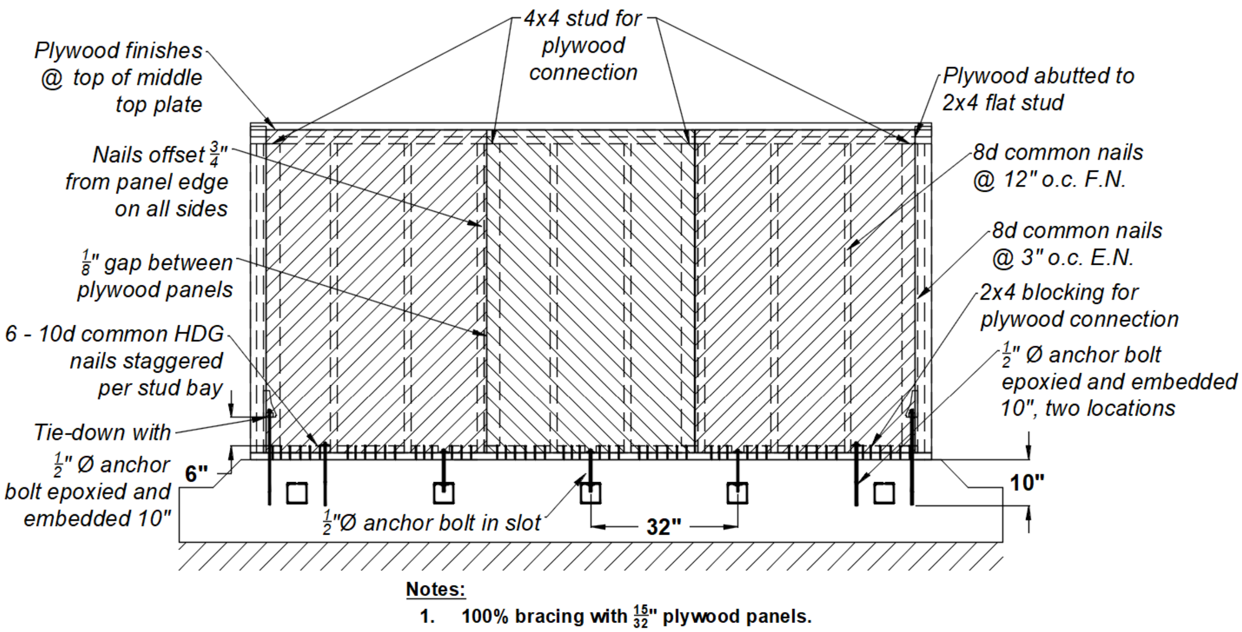


Figure 2.37 **Retrofit design for the 2-ft-tall cripple wall with horizontal siding exterior finish (Specimen A-8).**



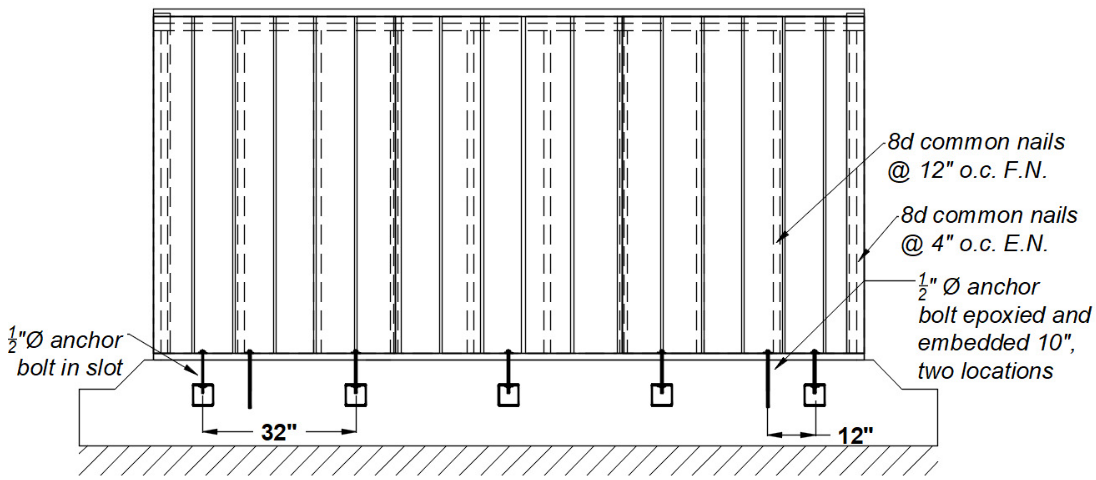
Elevation
Retrofit Framing Details (Interior Face)
2' Tall Cripple Wall
7 Anchor Bolt Configuration

Figure 2.38 Retrofit design for the 2-ft-tall cripple wall with horizontal siding over diagonal sheathing exterior finish (Specimen A-10).



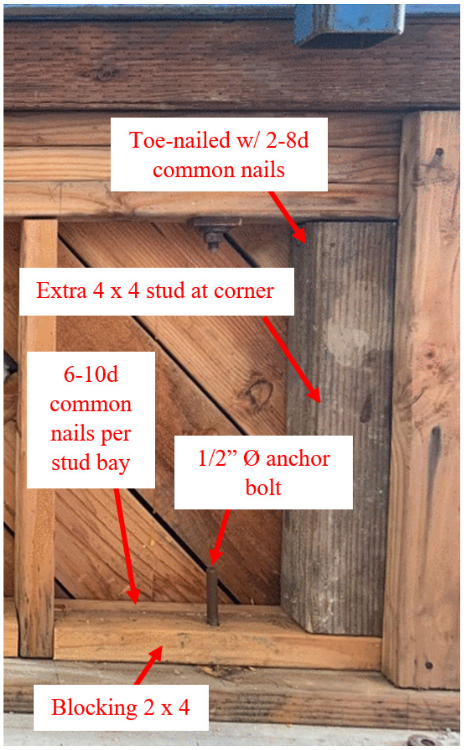
Elevation
Retrofit Framing Details (Interior Face)
6' Tall Cripple Wall
7 Anchor Bolt Configuration w/ Tie-Downs

Figure 2.39 Retrofit design for the 6-ft-tall cripple wall with horizontal siding exterior finish (Specimen A-14).

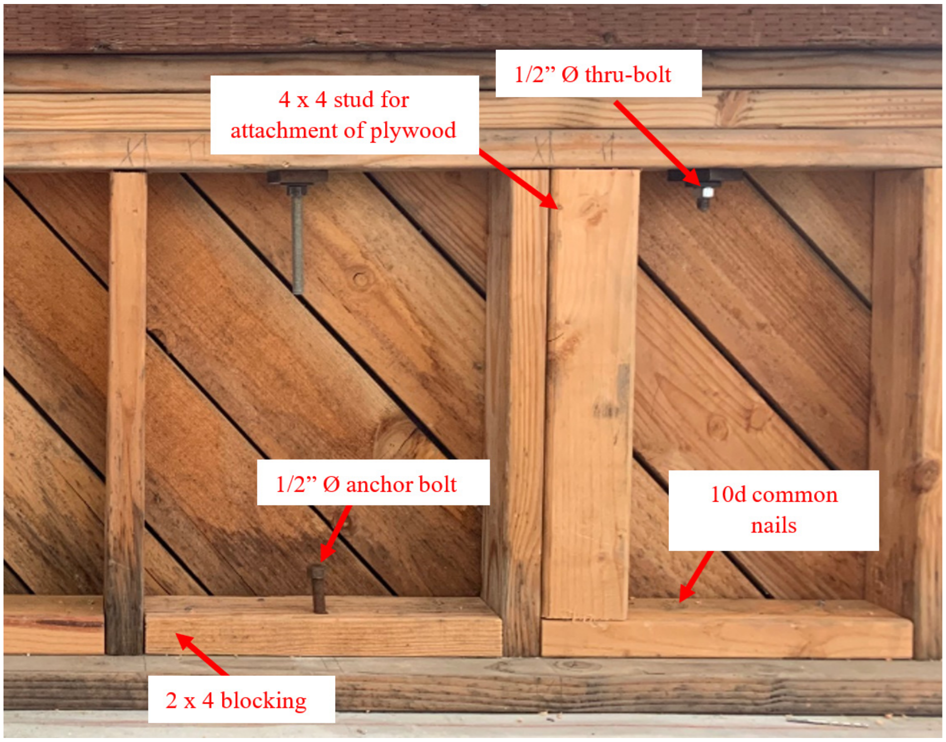


Elevation
Retrofit Framing Details (Interior Face)
6' Tall T1-11 Cripple Wall (T)
7 Anchor Bolt Configuration

Figure 2.40 Retrofit design for the 6-ft-tall cripple wall with T1-11 wood structural panel exterior finish (Specimen A-24).

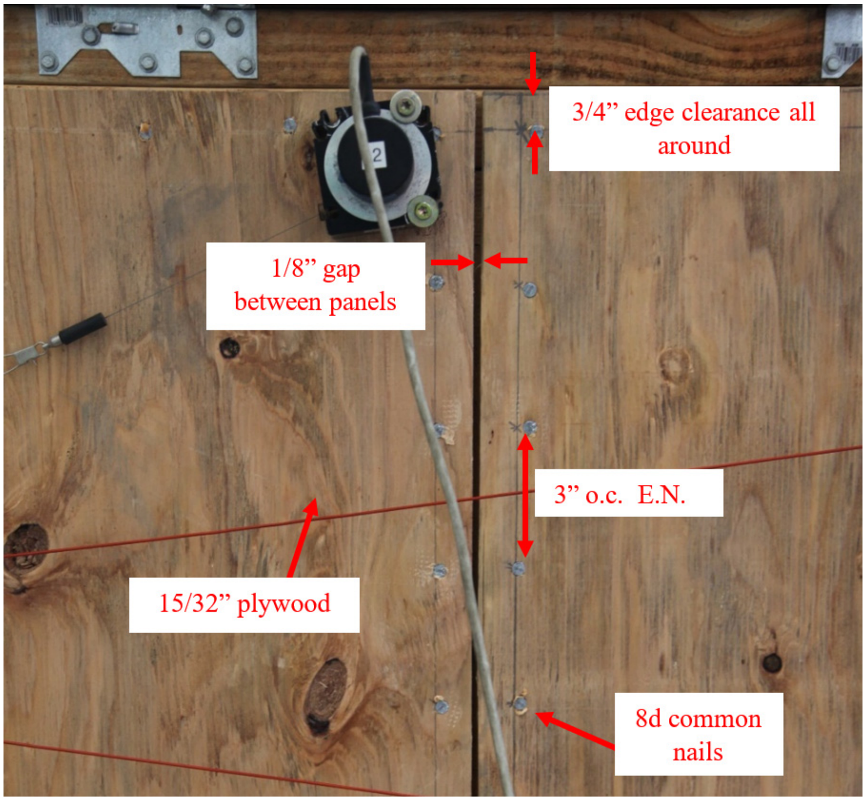


(a)

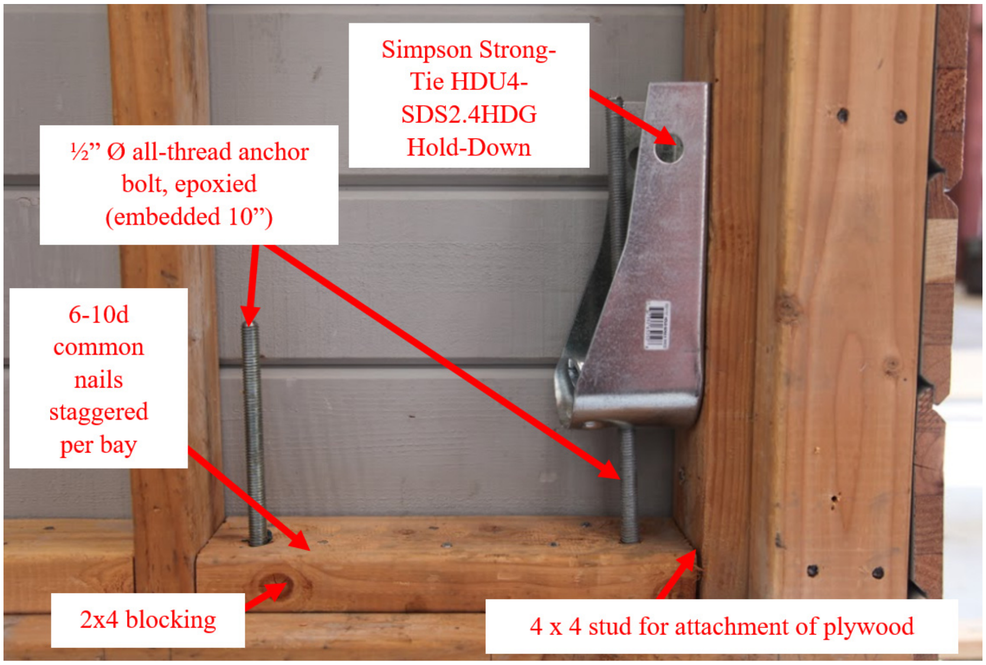


(b)

Figure 2.41 **Retrofit application details: (a) interior corner retrofit detail; (b) interior retrofit detail; and (c) plywood attachment detail.**



(a)



(b)

Figure 2.42 Specimen A-14 tie-down placement.

2.4.1 T1-11 Cripple Wall Retrofit Design and Installation

The retrofit design for cripple walls with T1-11 wood structural panels does not follow the typical *FEMA P-1100* retrofit design provisions used for the rest of the retrofitted cripple walls because the T1-11 is a wood structural panel; thus the addition of plywood on the interior was deemed excessive. In lieu of adding plywood to the interior of the framing, additional nails were added to the T1-11 panels. Originally, the nailing for the T1-11 panels was 8d common, HDG, nails edge-nailed at 8 in. on center and field-nailed at 12 in. on center. For the retrofit design, the edge-nail distance was cut in half to 4 in. on center. In addition, the original T1-11 design only had three edges of the panels nailed and relied on the next panel to sandwich the underlying panel. For the retrofit design, an additional row of nails was added to the underlying panel as shown in Figures 2.43 and 2.45. A comparison of the nailing patterns is shown in Figure 2.44. No additional anchor bolts were installed for the 2-ft-tall specimen (3 anchor bolts spaced 64 in. on center). The decision was made to use seven anchor bolts for the 6-ft-tall specimen, three original and four added in accordance with the *FEMA P-1100* prescriptive retrofit design; see Figure 2.40.

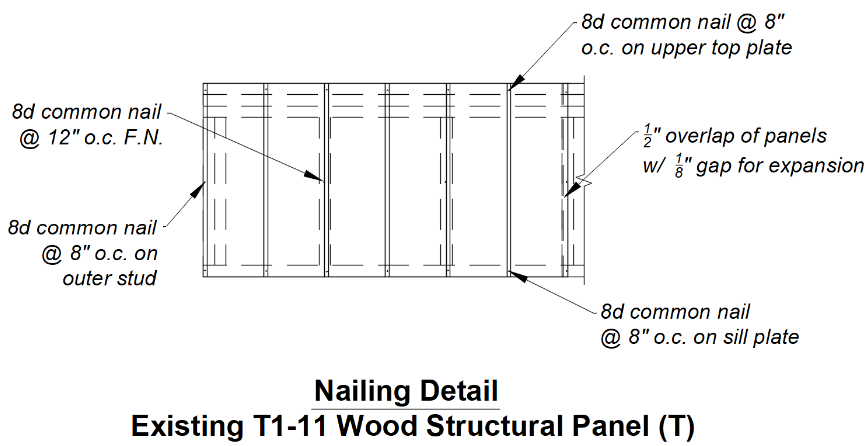


Figure 2.43 Nailing detail of T1-11 wood structural panels for retrofitted T1-11 cripple wall.

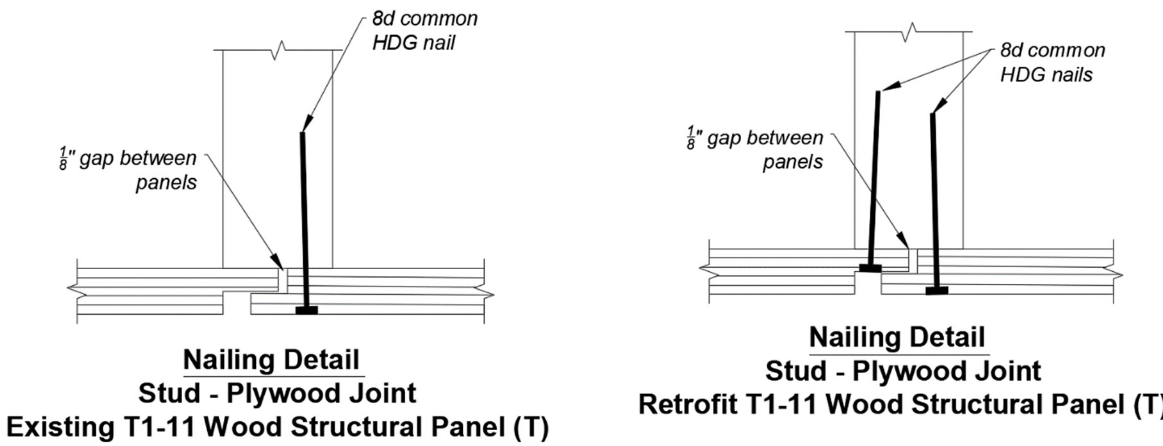


Figure 2.44 Nailing detail for existing (left) and retrofitted (right) T1-11 cripple walls.

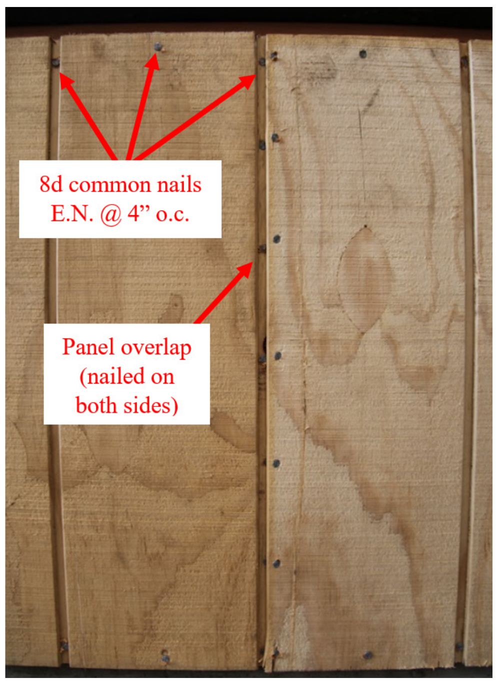
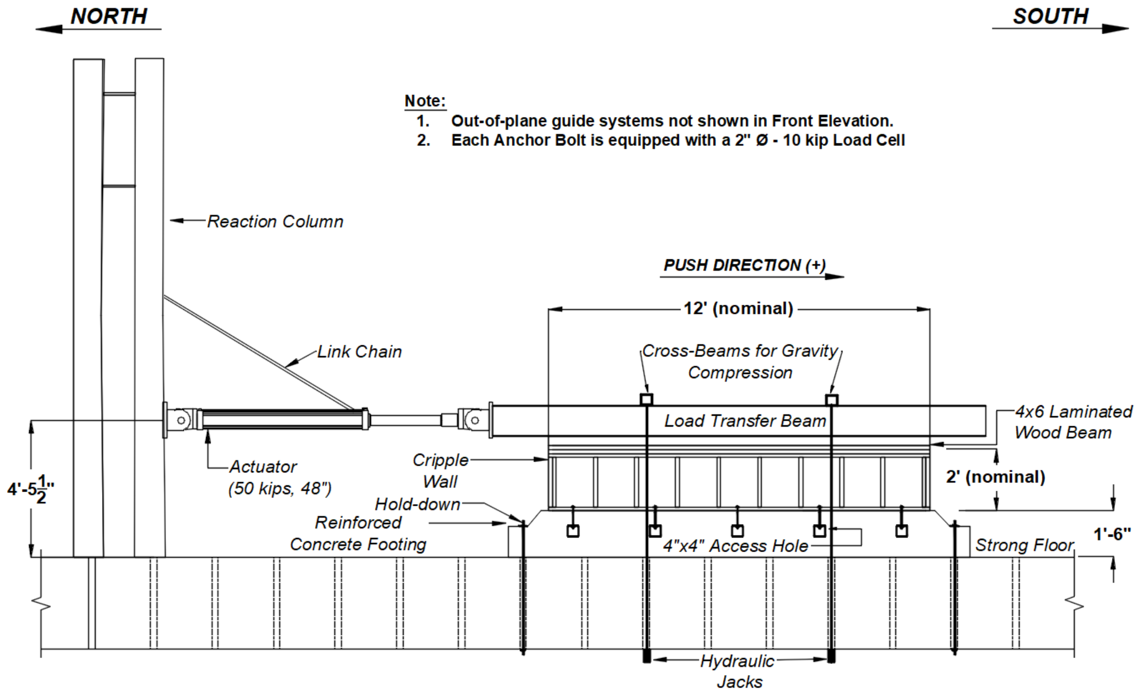


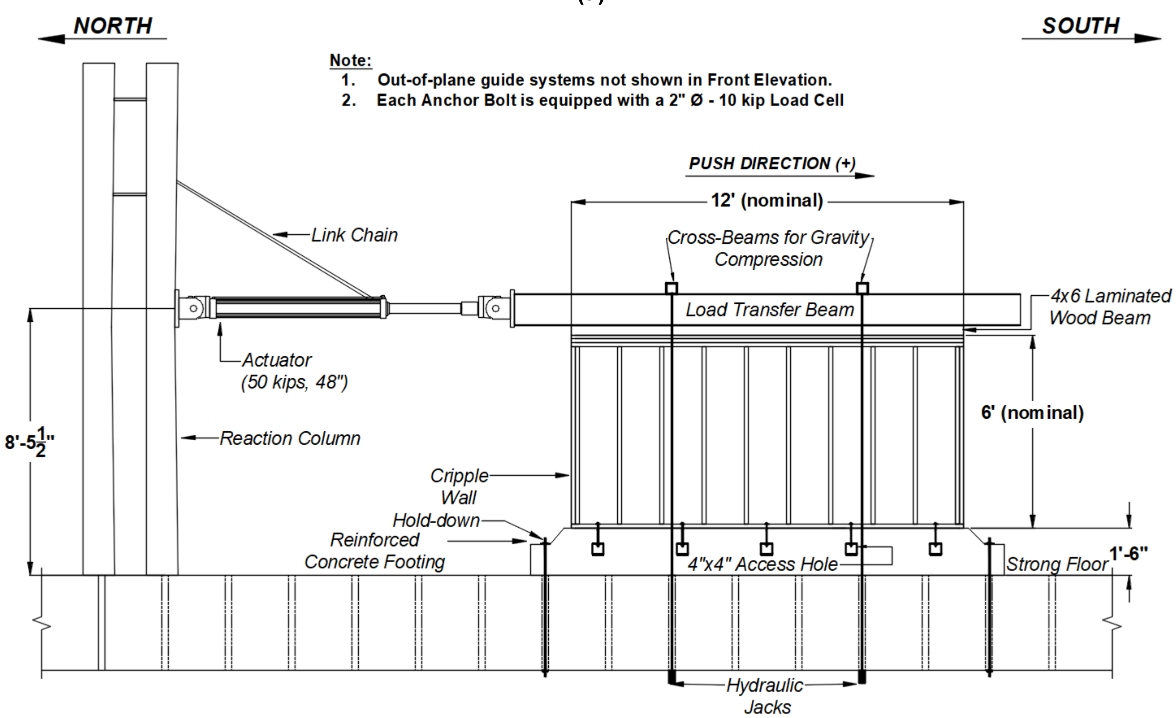
Figure 2.45 Retrofitted T1-11 cripple wall nailing pattern.

2.5 TEST SETUP

Figure 2.46 shows a plan view and elevations view of the test setup for both the 2-ft-tall and 6-ft-tall specimens. A complementary photograph of the 2-ft-tall test setup is shown in Figure 2.47. The same test setup was used for all cripple walls in this report, with the exception of the cripple wall tested with a light vertical load. The modifications to the test setup for the light vertical load are discussed in the following section. The lateral load was applied with a 48-in. (total) stroke, servo-controlled, hydraulic horizontal actuator capable of imposing 50 kips. The actuator was mounted to a strong wall using an actuator mounting plate, with its weight carried via a link chain back to the reaction wall so as to not impose a vertical load on the cripple wall. The lateral force was transferred from the actuator to the cripple wall with a stiff steel beam (W12 × 26 section). To allow for uninhibited movement of the finishes and plywood panels (when present in retrofitted walls) during testing, a 4 × 6 laminated wood beam was used as a spacer between the steel beam and the uppermost top plate of the cripple wall. This also facilitated ease of assembly of the specimens. A 1 in. × 1 in. notch was cut out of the laminated wood beam to allow for the exterior finish materials to freely rotate. Details of the connection of the steel beam, laminated wood beam, and cripple wall framing can be seen in Figure 2.48. The connection from the steel beam to the laminated wood beam was made with pairs of 3/8 in-diameter × 3-1/2-in. long lag bolts at 16 in. on center spacing, connected from the bottom flange of the steel beam top of the wood beam. The laminated wood beam was selected to be sufficiently thick as to preclude connection between the lag bolts and the cripple wall top plates. The cripple wall specimens were connected to the laminated wood beam using ½-in. diameter × 7-1/2-in. long, Grade 2 steel thru bolts spaced at 32 in. on center. These bolts were countersunk into the laminated wood beam and fastened with nuts and washers at the bottom of the lowermost top plate.

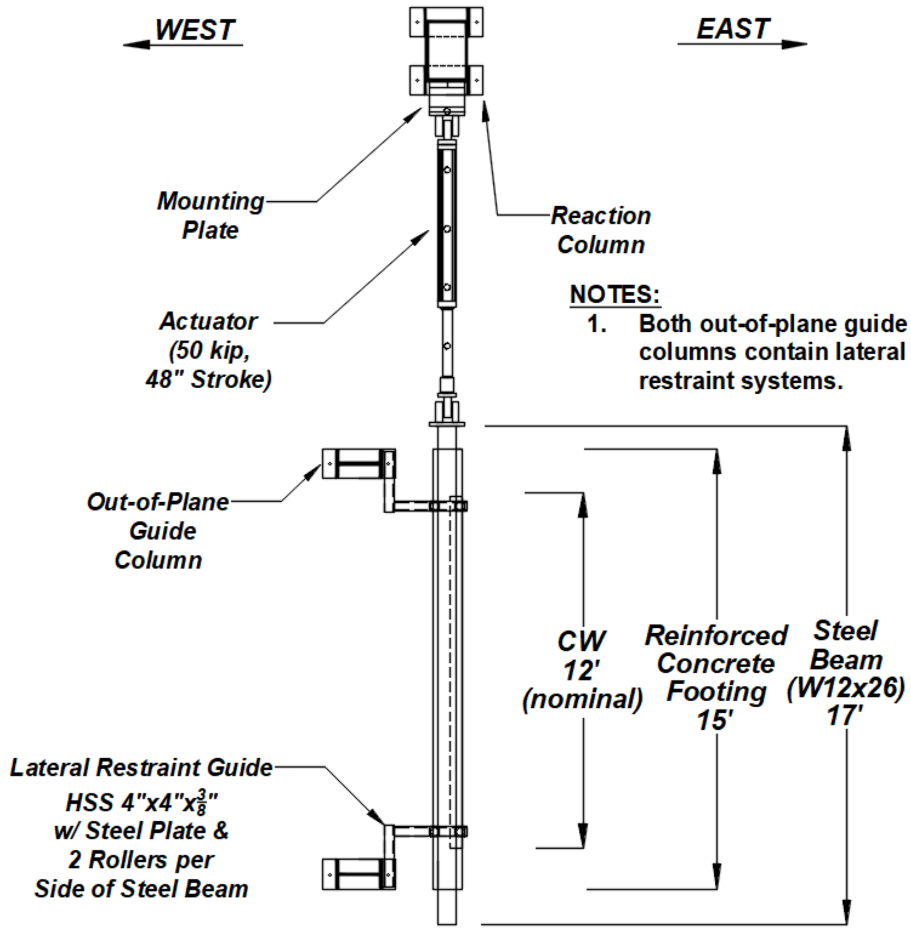


Elevation
2' Tall Cripple Wall Test Setup
(a)



Elevation
6' Tall Cripple Wall Test Setup
(b)

Figure 2.46 **Test setup: (a) elevation for 2-ft-tall cripple wall (interior face); (b) elevation for 6-ft-tall cripple wall (interior face); and (c) plan view.**



Plan View
Cripple Wall Test Setup

(c)
Figure 2.46 (continued).

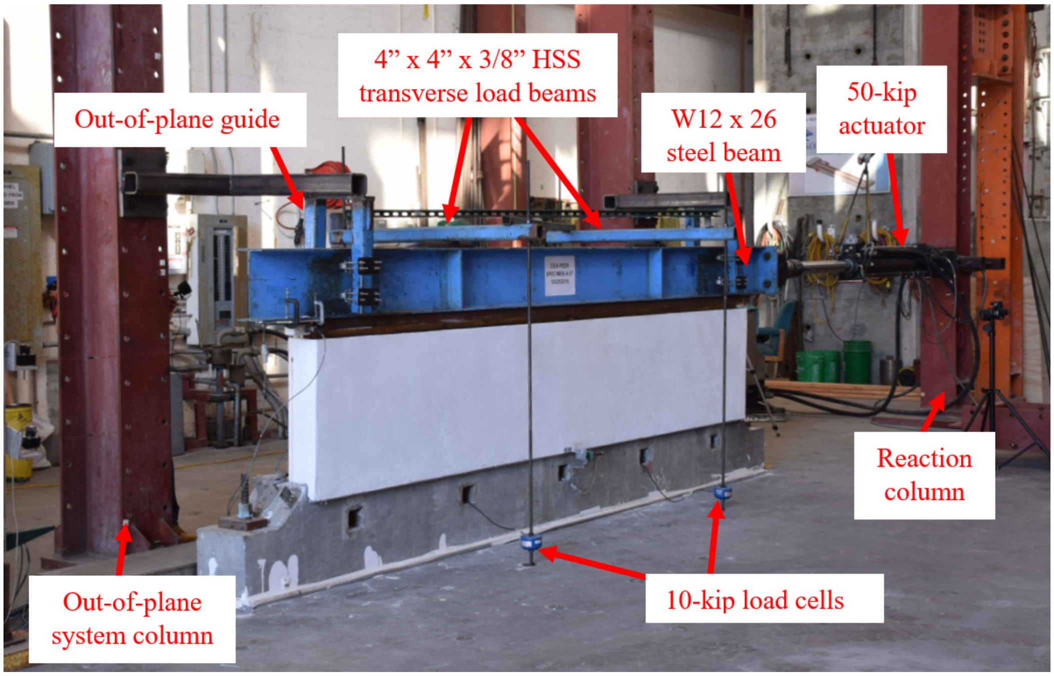


Figure 2.47 Isotropic view of the test setup for 2-ft-tall cripple walls.

As plausible, the concrete footing was reused for each test as it was fastened to the strong floor with a rod at each end, each tensioned to 50 kips. Individual dry finished specimens were constructed on the laboratory floor and erected onto the concrete footing; subsequently, the laminated wood beam and steel beam were attached. After these beams were attached, the actuator was attached with four 1-in.-diameter bolts. Subsequently, two 4 in. × 4 in. × 3/8 in. HSS sections were placed transversely at third points along the specimen, as they were utilized to apply vertical load to the steel beam. Each transverse HSS beam had a ½- in.-diameter all thread rod attached at each end. The thread rods were attached to hydraulic jacks at the base of the strong floor. The hydraulic jacks were used to apply the desired vertical load to each specimen. The location of the transverse beams can be seen in Figures 2.46 and 2.47. The choice of location for applying the loads was meant to result in an approximately uniformly distributed gravity load on the full length of the cripple wall specimen. It is noted that while additional point loads would have increased the uniformity of the load distribution, they would have also increased the complexity significantly. In addition, the stiff W12 × 26 lateral transfer beam was deemed sufficient to nominally result in a uniform load application. It is noted that 400 lbs of the target 5400 lbs (the 450 plf case) were available via the weight of the lateral steel and wood laminated transfer beams, thus the transverse HSS assembly required application of an additional 1250 lbs per point load location. Each thread rod at the HSS transverse beam load locations was equipped with a 10 kips load cell used to monitor the applied vertical load during testing.

Every cripple wall with the exception of one specimen was subjected to a constant uniform vertical load of 450 lbs/ft (5400 lbs total). The weight of the steel transfer beam, laminated wood transfer beam, and the transverse vertical loading beams coupled with the use of a pair of hydraulic jacks tied to the bottom of the strong floor was cumulatively utilized to achieve this target vertical load. It is noted that 400 lbs of the target 5400 lbs (450 plf case) were available via the weight of the lateral steel and wood laminated transfer beams, thus the transverse HSS assembly required application of an additional 1250 lbs per point load location, which was applied by each hydraulic jack. Due to eccentricity of the walls when constructed with the bottom boundary condition c, the applied loads measured were not always 1.25 kips each. Loads ranged from 1.15 kips to 1.40 kips for each hydraulic jack; nonetheless, the test setup was able to regulate the load within 10% of target at 4.8 kips to 5.0 kips for the sum of all hydraulic jacks.

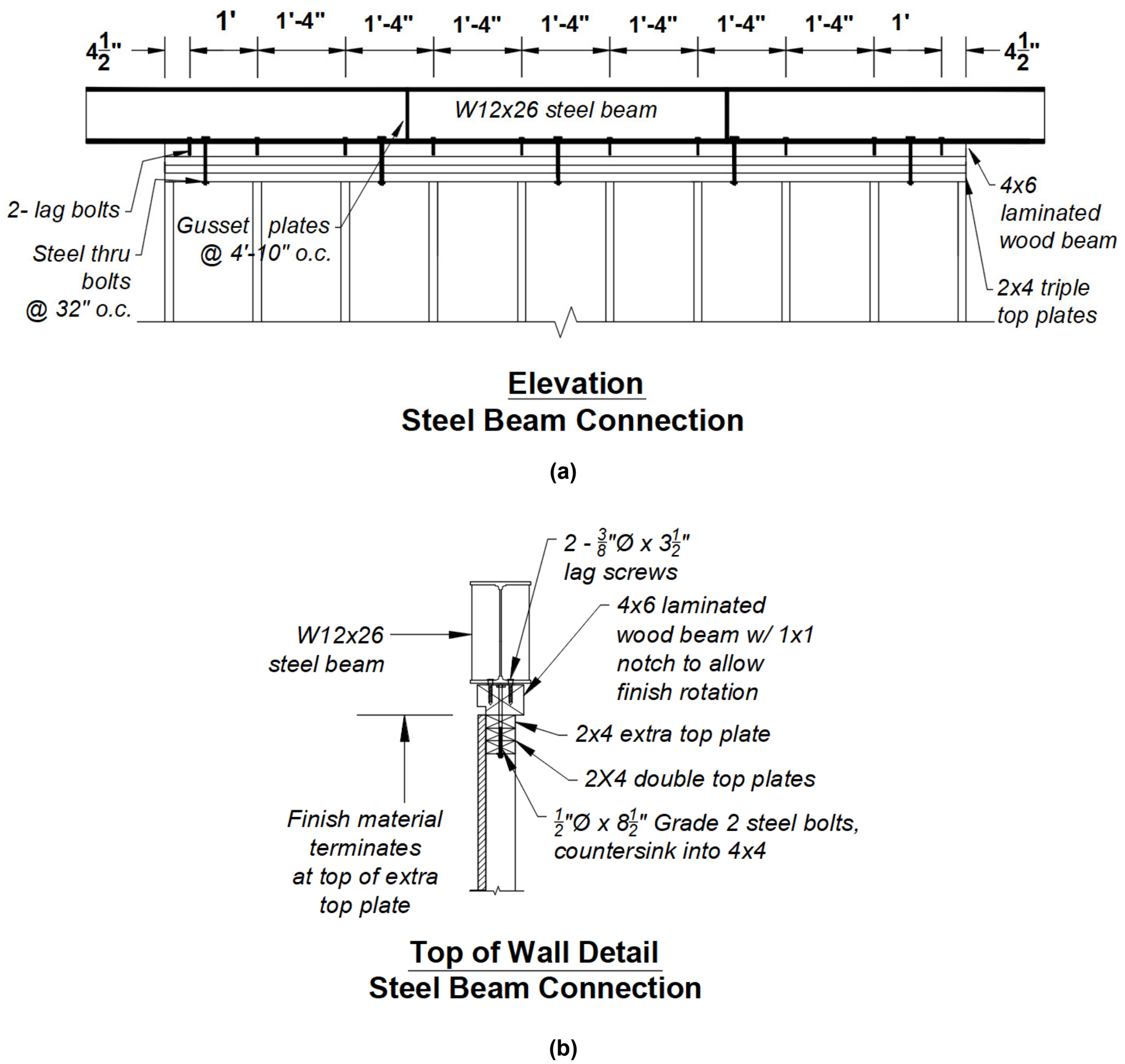


Figure 2.48 Steel beam connections: (a) elevation of steel beam connection; and (b) top of wall detail.

Before any loads were applied to the cripple wall, pairs of rollers were fastened to the sides of the out-of-plane guide. This can be seen in Figures 2.46 and 2.47. The rollers were greased, and a 1/16-in. gap was left between the steel plate and the steel transfer beam so as to not impose any artificial loads via friction force at the contact interface of the plates and beam. The purpose of implementing an out-of-plane guide system was to ensure that the imposed displacement during testing is only in-plane.

Once the vertical load was applied to the test setup, the anchor bolts were tensioned. For all tests, each anchor bolt was tensioned to 200 lbf. The change in anchor bolt tensioning was made to mimic the amount of tension commonly seen in anchor bolts of existing California homes, which would be most akin to a "hand-tightened" condition. Once the anchor bolts were tensioned, a bias of all instrumentation including the actuator load and displacement was made, and all values were recorded before and after the bias. At this point the test would begin. The lateral displacements imposed are described in the previous chapter.

2.5.1 Light Vertical Load

One of the eleven tests was subjected to 150 lbs/ft vertical load (1800 lbs total), denoted as the light vertical load case. The vertical load application setup required modification because the hydraulic jacks were not able to impose such small loads precisely and accurately. The modifications involved replacing the transverse load beams, rods, axial load cells, and hydraulic jacks with 46 steel plates and a Dywidag bar. The steel beam and laminated wood transfer beam accounted for 400 lbs of the required vertical load, and thus the steel plates and Dywidag bar were designed to account for the remaining 1400 lbs. The steel plates were 6 in. × 6 in. × 2 in., and the bar was 2 in. in diameter. The steel plates and bar were centered on top of the steel load transfer beam and then welded in place, as shown in Figures 2.49 and 2.50.

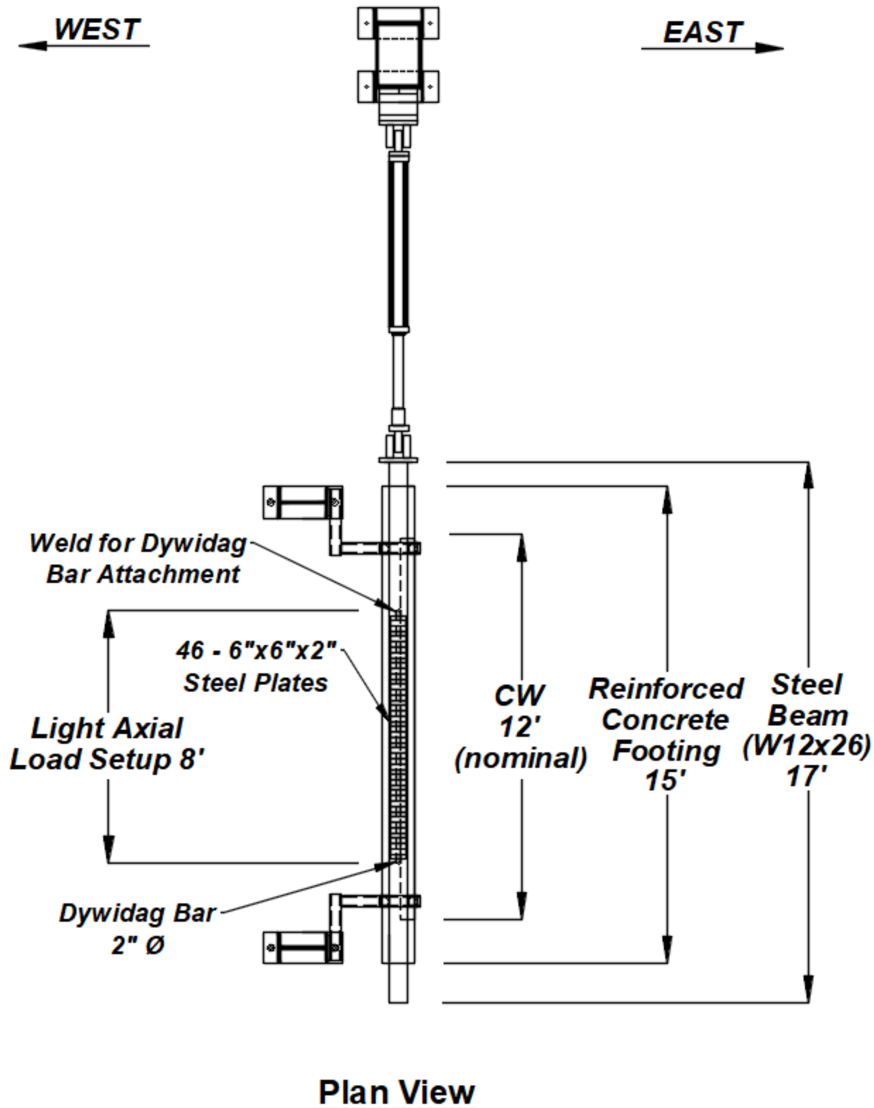


Figure 2.49 Test setup for light vertical load cripple wall.

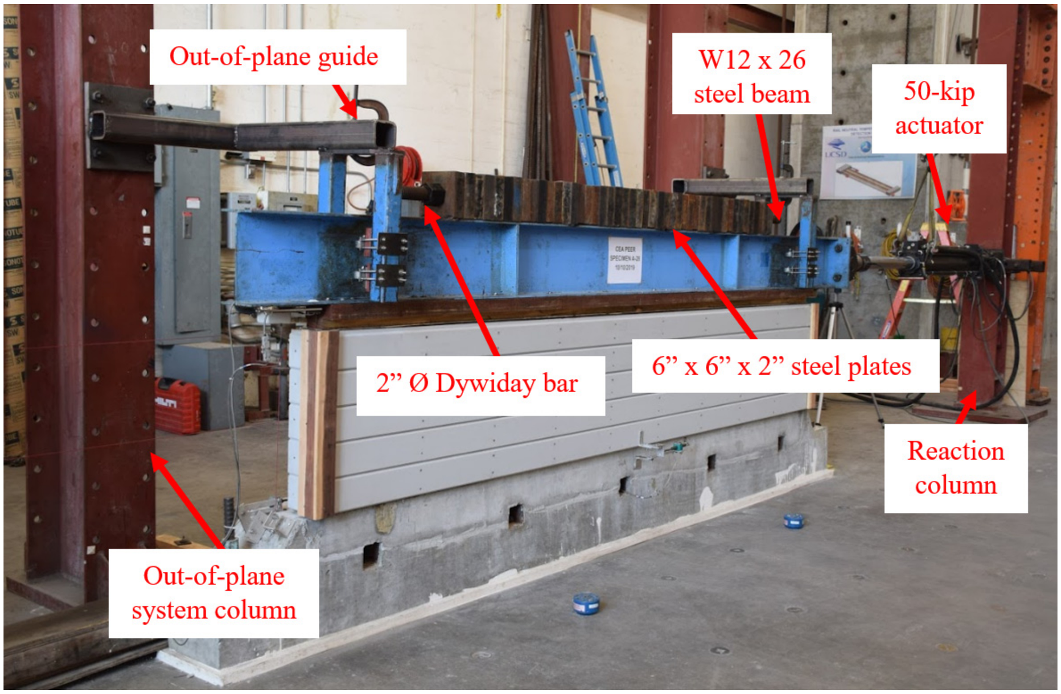
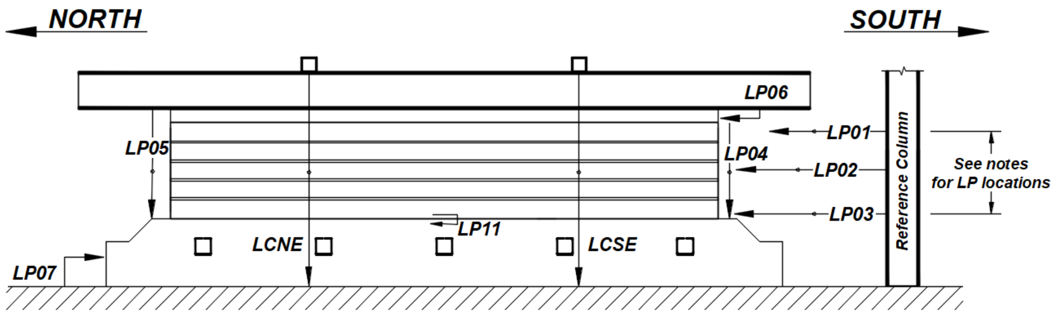


Figure 2.50 **Isometric view of the test setup for 2-ft-tall cripple walls with light vertical load.**

2.6 INSTRUMENTATION

Extensive measurements of displacements, rotations, and loads were performed on each cripple wall specimen. Each specimen had slight variations in instrumentation depending on its exterior finish and retrofitting condition. Figure 2.51 shows the instrumentation details for a 2-ft-tall cripple wall. Figure 2.51(a) and (b) show the exterior (finish) and interior (framing) elevations, respectively. Figure 2.51(c) provides a detail for the interior of a horizontal siding exterior finished cripple wall, and Figure 2.51(d) provides the same detail for a horizontal siding over diagonal sheathing cripple wall. Figure 2.52 shows the instrumentation details for Specimen A-13, a 6-ft-tall cripple wall with horizontal siding exterior finish.

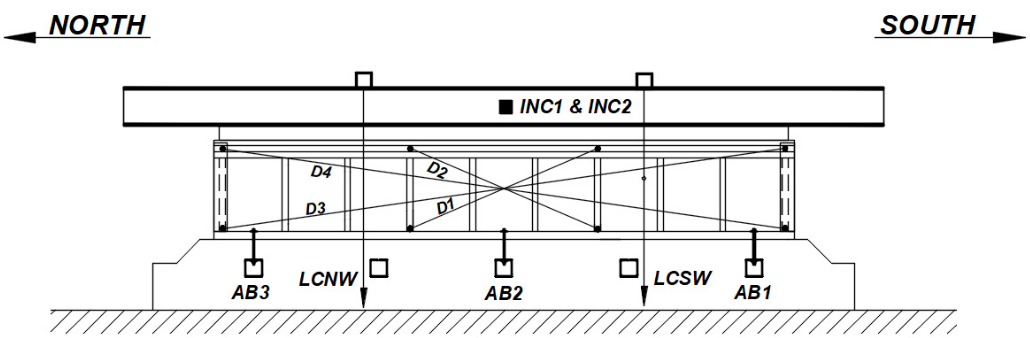
The overall response of the cripple wall was characterized using displacements measured by displacement transducer LP01. LP01, plus transducers LP02 and LP03, were connected to a stationary reference column tensioned into strong floor. For a 2-ft-tall cripple wall, LP01 was attached to the top of the middle top–24 in. from the top of the concrete footing–and captured the total displacement at the top of the cripple wall. LP02 was attached to the middle of the cripple wall, at a height of 12 in. from the top of the footing. This intermediate displacement transducer was used to define the deflected shape of the cripple wall. LP03 was attached to the middle of the sill plate and was used to measure the absolute displacement of the sill plate. For a 6-ft-tall cripple wall, LP01 was connected to the middle of the upper top plate–72 in. from the top of the concrete footing–and LP03 was connected to the middle of the sill plate. LP02 was placed in the middle of the cripple wall, 36 in. from the top of the concrete footing. By taking the difference between LP01 and LP03, the relative displacement of the cripple wall could be determined (neglecting sill slippage). Details of these transducers can be seen in Figure 2.51(a) and 2.52(a). A photograph of the placement of transducers LP01–LP03 can be seen in Figure 2.53(a) for a 2-ft-tall cripple wall and Figure 2.53(b) for a 6-ft-tall cripple wall.



Notes:
1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of second top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LCNE and LCSE monitor the axial load on the East Side of the wall.
6. LP11 monitors displacement of siding relative to the footing.

Instrumentation Elevation (E1)
Finish Face
2' Tall Cripple Wall

(a)

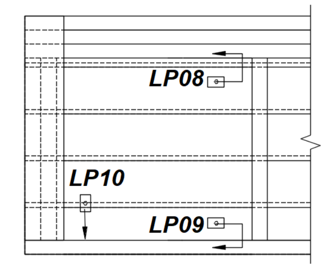


Notes:
1. AB1-AB3 are instrumented with 2" Ø Donut Load Cells measuring uplift loads.
2. Pairs of strength potentiometers are mounted on framing studs or plywood panels for retrofit cases.
3. INC1 measures longitudinal rotation of load transfer beam. INC2 measures the transverse rotation of the load transfer beam.
6. LCNW and LCSW measure the applied axial load.
7. D1-D4 monitor the diagonal distortion.

Instrumentation Elevation (E2)
Framing Face
2' Tall Cripple Wall

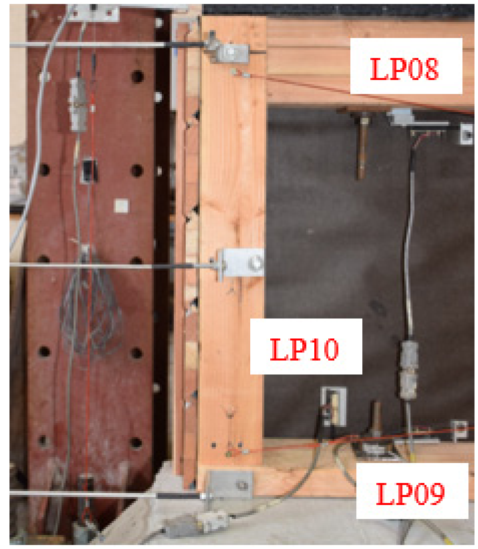
(b)

Figure 2.51 Instrumentation details for a 2-ft-tall cripple wall: (a) elevation for the horizontal siding finish face; (b) elevation for the framing face; (c) instrumentation details for horizontal siding; and (d) instrumentation details horizontal siding over diagonal sheathing.

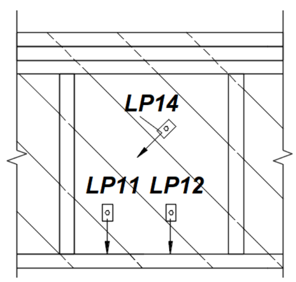


Notes:
1. LP08 and LP09 monitor the siding slip.
2. LP10 monitors uplift of bottom siding board.

Instrumentation Detail D1
Siding

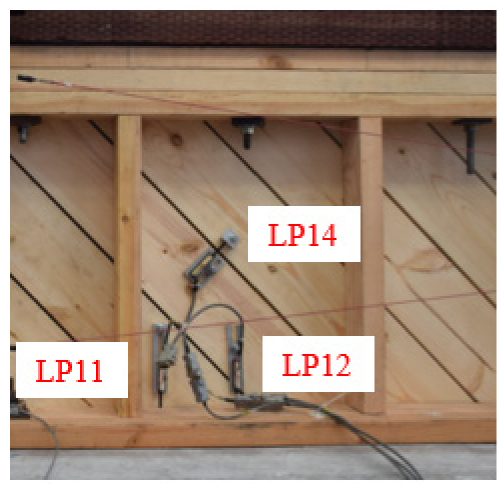


(c)



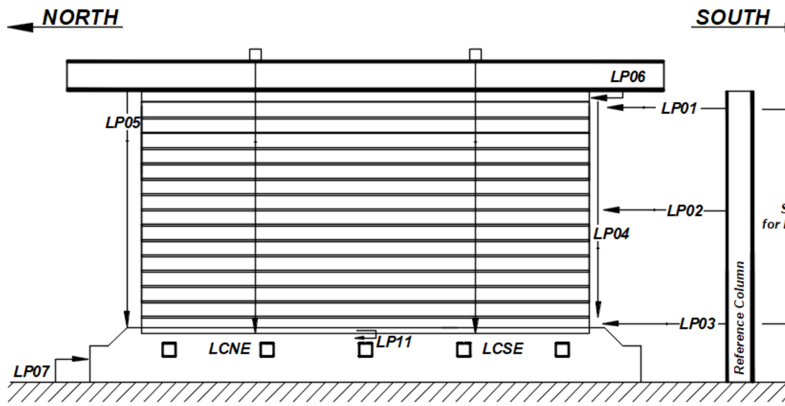
Notes:
1. LP11 and LP12 monitor the sheathing uplift.
2. LP14 monitors sheathing board gap.

Instrumentation Detail D2
Sheathing



(d)

Figure 2.51 (continued).

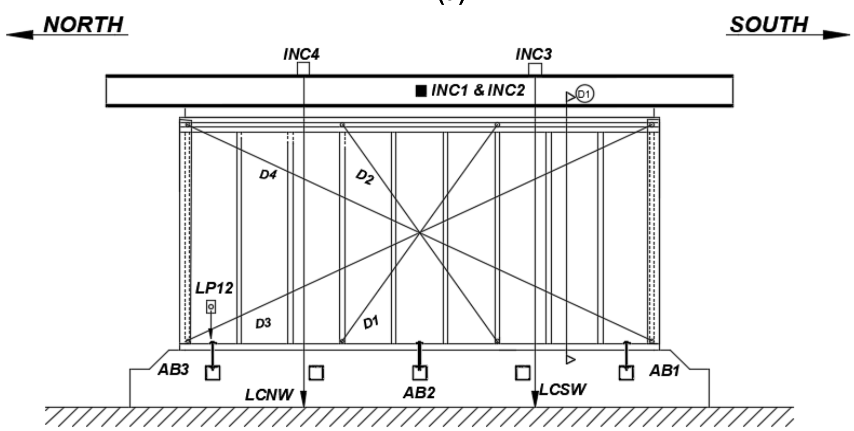


Notes:

1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of second top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LCNE and LCSE monitor the axial load on the East Side of the wall.
6. LP11 monitors displacement of siding relative to the footing.

Instrumentation Elevation (E1)
Finish Face
2' Tall Cripple Wall

(a)



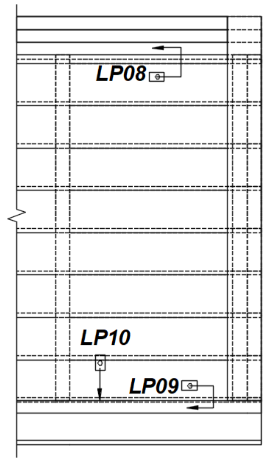
Notes:

1. AB1-AB3 are instrumented with 2" Ø Donut Load Cells measuring uplift loads.
2. Pairs of strength potentiometers are mounted on framing studs or plywood panels for retrofit cases.
3. INC1 measures longitudinal rotation of load transfer beam. INC2 measures the transverse rotation of the load transfer beam.
4. INC3 and INC4 measure the rotation of the transverse beams where axial load is applied.
5. LCNW and LCSW monitor the axial load on the West Side of the wall.
6. LP12 monitors the uplift of the bottom siding board relative to the sill plate.

Instrumentation Elevation (E2)
Framing Face
6' Tall Cripple Wall

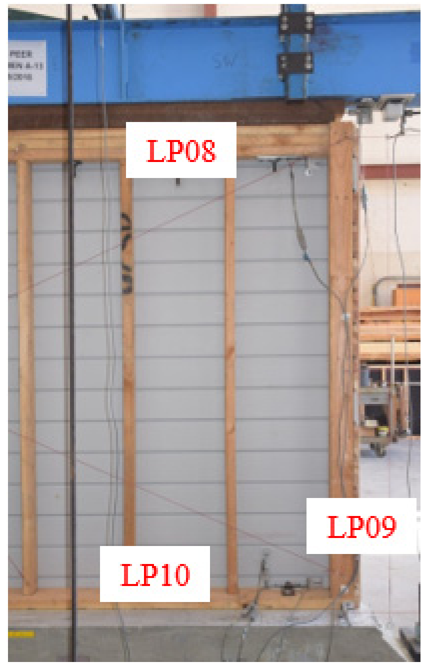
(b)

Figure 2.52 Instrumentation details for a 6-ft-tall cripple wall: (a) elevation for the horizontal siding finish face; (b) elevation for the framing face; and (c) instrumentation details for horizontal siding.



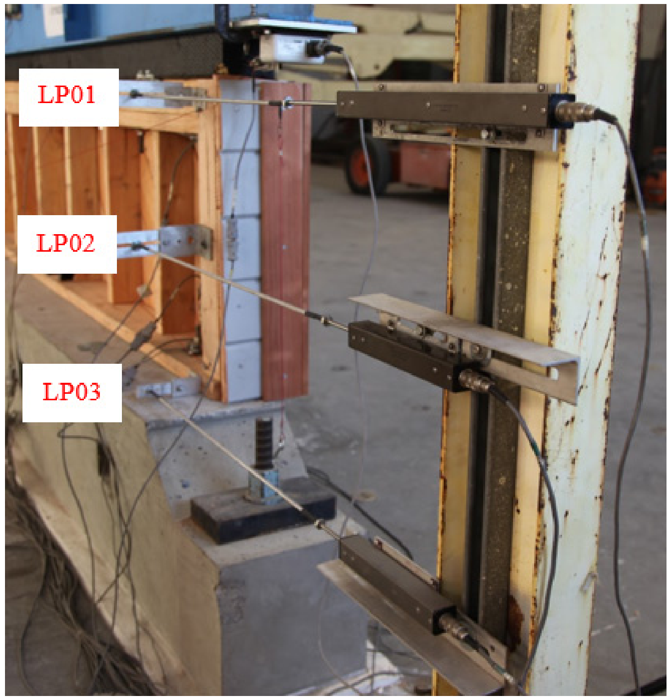
Notes:
1. LP08 and LP09 monitor the sheathing slip.
2. LP10 monitors the sheathing uplift

Instrumentation Detail D1
Sheathing

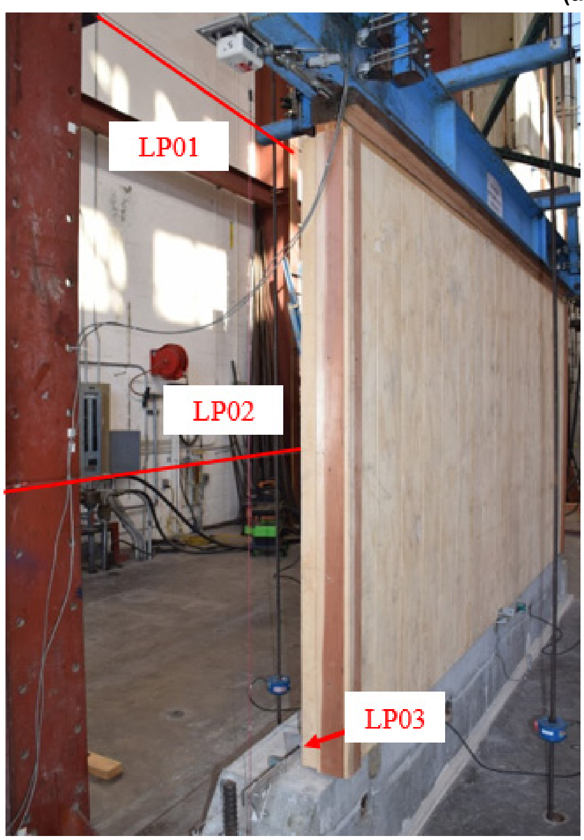


(c)

Figure 2.52 (continued).



(a)



(b)

Figure 2.53 Displacement transducers LP01–LP03 placement: (a) 2-ft-tall cripple wall; and (b) 6-ft-tall cripple wall.

Local deformations of the cripple wall were also measured. For retrofitted cripple walls, plywood panel deformations of the interior panel and all three panels were measured with two pairs of diagonal displacement transducers denoted as D1–D4. For existing cases, the diagonal transducers were fastened to the framing on the top and bottom of studs for the interior transducers and the flat corner studs for the outer transducers. The location of these diagonal transducers can be seen in Figures 2.51(b) and 2.52(b). The inner diagonal transducers, D1 and D2, characterized the distortion of the middle 4-ft section of the wall or the middle plywood panel when the cripple wall was retrofitted. For the retrofitted cases, however, the shear distortion of the middle panel was smaller than the resolution of the displacement transducers. The outer diagonal transducers, D3 and D4, characterized the overall distortion of the entire cripple wall.

Uplift of the cripple wall was measured at each end with displacement transducers LP04 and LP05, shown in Figures 2.51(a) and 2.51(a). For 2-ft-tall cripple walls, the uplift measurements were also out of the resolution range of the transducers. This was not expected to be the case with 6-ft-tall cripple wall specimens. The slip between the steel transfer beam and the uppermost top plate was measured by LP06. It should be noted that even if slip between steel transfer beam and top plate occurred, it did not affect the amount of displacement imposed on the cripple wall specimen as that was controlled by LP01, which is attached to the cripple walls itself. Displacement transducer LP07 was attached to the strong floor and measured the slip between the foundation and strong floor. The foundation was tensioned to ensure that no slip occurred at this interface.

Two inclinometers denoted as INC3 and INC4 were attached to the east end of the transverse vertical load beams to measure rotations of the beams during loading. Each transverse load beam was tensioned through a thread rod and a hydraulic jack fastened under the strong floor. Each thread rod was connected to a 10-kip load cell used to monitor the vertical load imposed. These load cells are shown in Figure 2.51(a) and (b) (for a 2-ft-tall cripple wall) and Figure 2.52(a) and (b) (for a 6-ft-tall cripple wall), and labeled according to their cardinal directional position (i.e., LCNW for the northwest load cell). The use of these four displacement transducers, two inclinometers, and four load cells worked to both monitor the vertical load applied to the specimen and determine lateral load imposed due to the horizontal component of the displacing vertical load. This artificial horizontal load component was taken out of the lateral responses of each cripple wall.

The tension in each anchor bolt is measured with a 10-kip donut load cell. These load cells monitored the uplift forces in the cripple wall. Refer to Section 2.2 for the setup of these load cells. Finally, two inclinometers, INC1 and INC2, were used to measure the rotation of the horizontal load transfer beam along the longitudinal and transverse axis of the loading direction. Additional displacement transducers were used to measure important displacements on various components of the cripple wall. As seen in Figures 2.51(c) and 2.52(c), LP10 measured the uplift of the bottom siding board. LP08 and LP09 measured the horizontal displacement of the top siding board and the bottom siding board, respectively.

As mentioned before, there were small variations in the instrumentation of some of the cripple wall specimens depending on the exterior finish and the retrofit condition; details are available in Appendix B.1.

2.7 CAMERA VIEWS

For each test, both extensive high-resolution digital photographs and videos documented the pre-test, during test, and after test state of each cripple wall specimen. During testing, photographs were taken at the push and the pull of the first cycle for each drift ratio level as well as at the end of the last cycle of the drift ratio level for the 0.2%–1.4% drift amplitudes. Five to six cameras were used to capture the live motion of the cripple wall during testing. Figure 2.54 shows the locations of each of the cameras used to record tests. The first two tests had the cripple wall facing the opposite direction; therefore, the arrangement of cameras was modified as shown in Figure 2.55. One of the cameras was a live web camera with views of the finish face of the cripple wall. These tests recorded the test continuously from start to finish. During video processing, the recordings of the webcams were edited and overlaid with the loading protocol as well as the lateral force–lateral displacement hysteresis of the cripple wall. The other three to four cameras worked to capture various angles of the walls that were deemed most important to help understand the specimen's behavior during testing. The framing face and finish face were often recorded with these cameras as well because the video resolution of these cameras is higher than that of the webcams. Other important areas to take video were the ends of the cripple walls. All of the cripple walls would bear on the foundation at their ends, which caused these areas to accumulate more significant damage than the framing or finish faces, especially at low drift amplitudes.

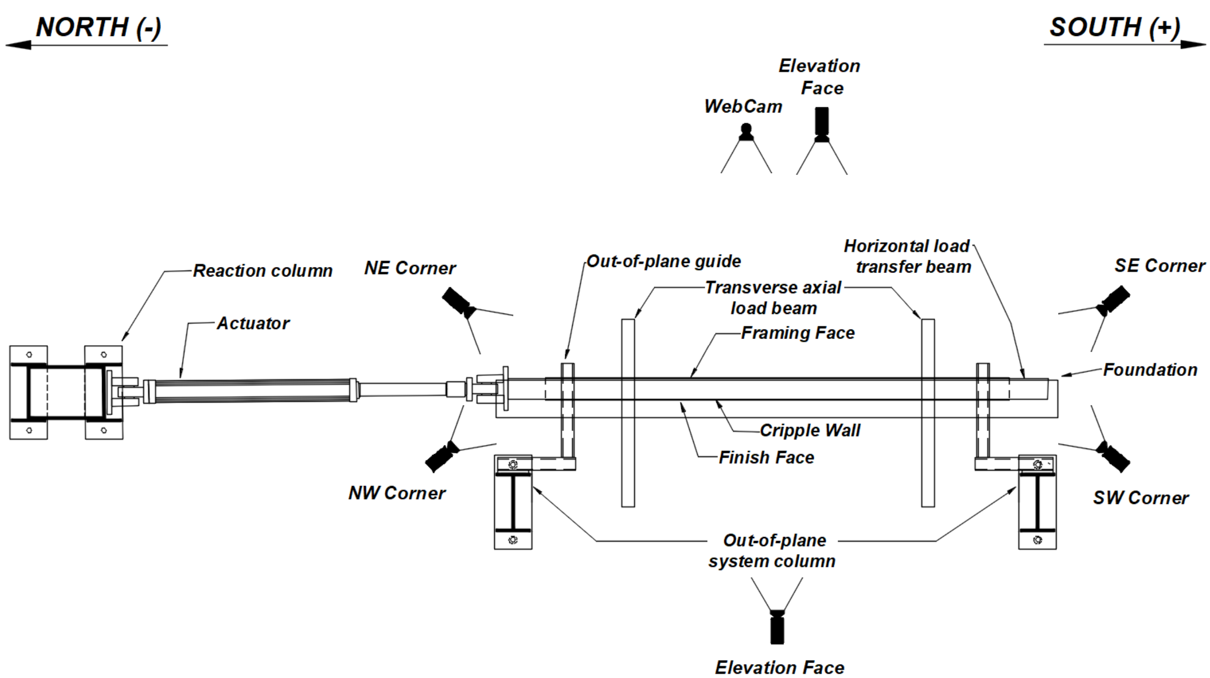


Figure 2.54 Layout of cameras and scope of view (for all but Specimens A-7 and A-8).

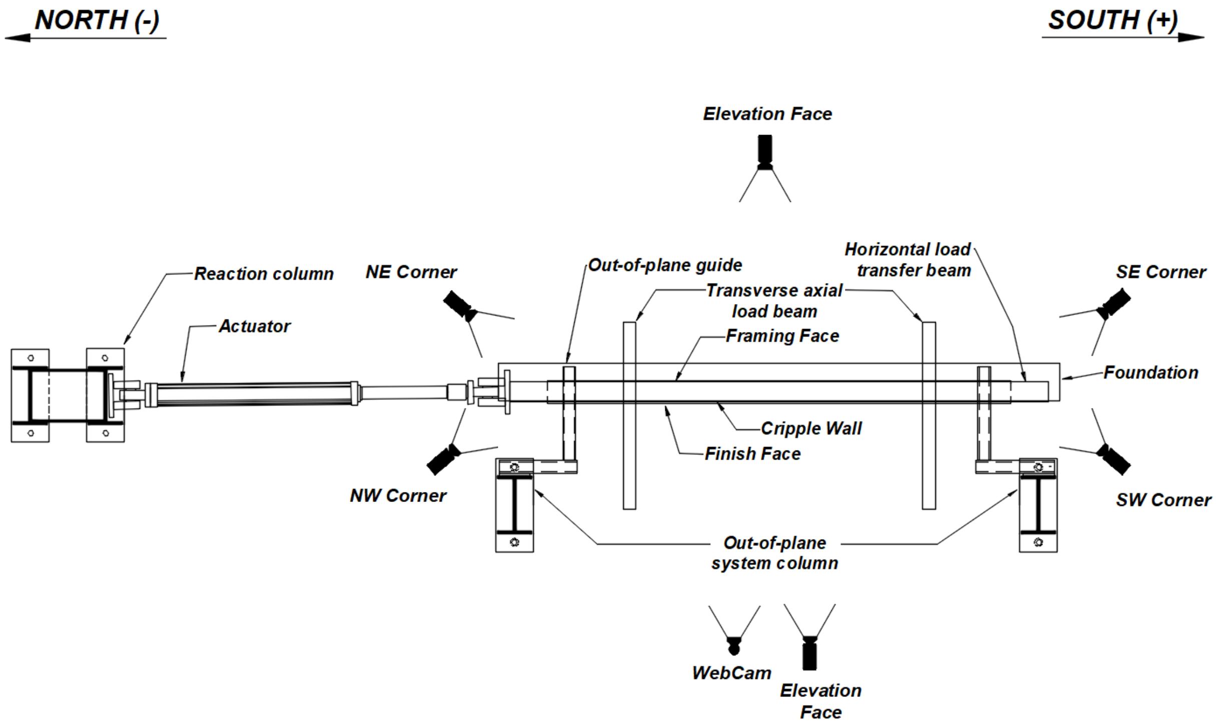


Figure 2.55 Layout of cameras and scope of view for Specimens A-7 and A-8.

2.8 LOADING PROTOCOL

The loading protocol for each test varied slightly depending on the rate of post-peak strength degradation of the individual specimen. All cripple walls underwent the same loading protocol up until the specimen realized a loss greater than 60% of its measured lateral strength. At this point in the protocol, the following and each subsequent drift ratio level was increased by 2%, rather than 1%. If the 60% loss in strength did not occur, each drift ratio level would remain at an increase of 1% per cycle grouping. The loading protocol would progress until an 80% loss in strength was realized. At this point, a monotonic push would be conducted, typically to a global drift of 20%. The amplitude of the monotonic push might vary slightly depending on instrumentation constraints. Figure 2.56 shows the loading protocol for Specimen A-7 (a 2-ft-tall cripple wall), and Table 2.4 gives details of the loading protocol. Figure 2.57 shows the loading protocol for Specimen A-13 (a 6-ft-tall cripple wall), and Table 2.5 gives details of the loading protocol. See Appendix A.4 for details of the loading protocol for each test.

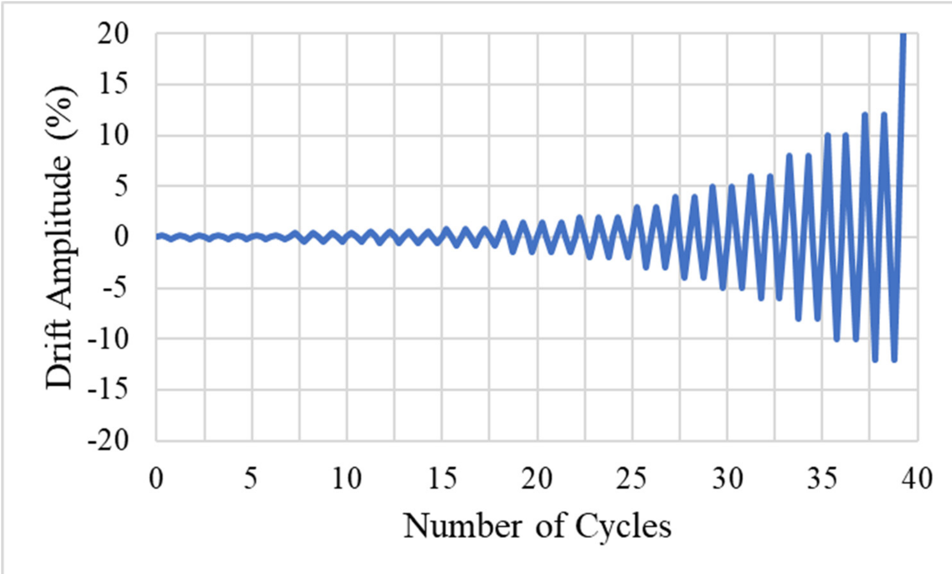


Figure 2.56 Loading protocol for the 2-ft-tall Specimen A-7.

Table 2.4 Summary of loading protocol for the 2-ft-tall Specimen A-7.

Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loadig rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.048	7	0.0064	30	210
2	0.4	0.096	4	0.0128	30	120
3	0.6	0.144	4	0.0192	30	120
4	0.8	0.192	3	0.0256	30	90
5	1.4	0.336	3	0.0448	30	90
6	2	0.48	3	0.064	30	90
7	3	0.72	2	0.096	30	60
8	4	0.96	2	0.128	30	60
9	5	1.2	2	0.16	30	60
10	6	1.44	2	0.192	30	60
11	8	1.92	2	0.256	30	60
12	10	2.4	2	0.16	60	120
13	12	2.88	2	0.192	60	120
14	Mono	5.0	--	0.333	60	60

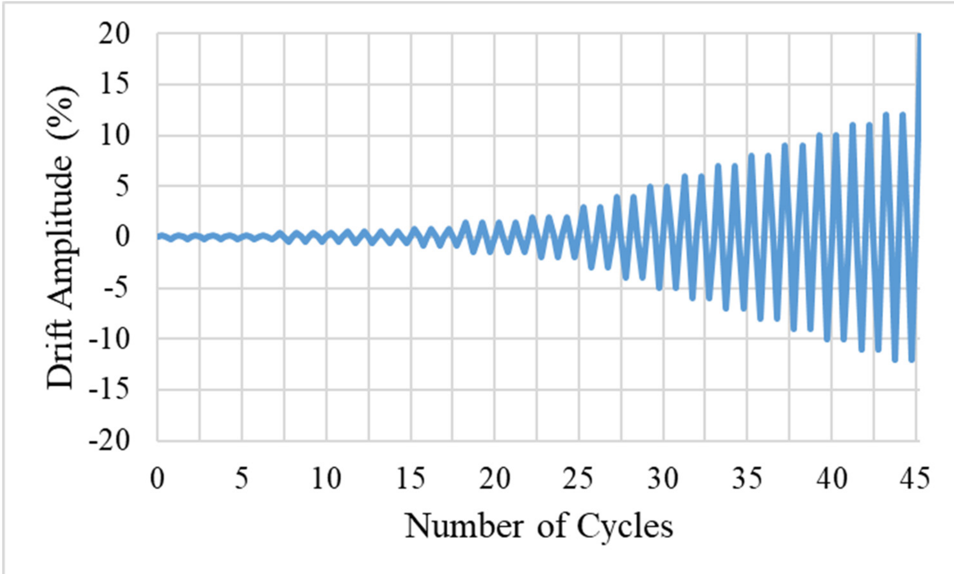


Figure 2.57 Loading protocol for the 6-ft-tall Specimen A-13.

Table 2.5 Summary of loading protocol for the 6-ft-tall Specimen A-13.

Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loading rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.144	7	0.0192	30	210
2	0.4	0.288	4	0.0384	30	120
3	0.6	0.432	4	0.0576	30	120
4	0.8	0.576	3	0.0768	30	90
5	1.4	1.008	3	0.1344	30	90
6	2	1.44	3	0.096	60	180
7	3	2.16	2	0.144	60	120
8	4	2.88	2	0.192	60	120
9	5	3.6	2	0.24	60	120
10	6	4.32	2	0.288	60	120
11	7	5.04	2	0.336	60	120
12	8	5.76	2	0.192	120	240
13	9	6.48	2	0.216	120	240
14	10	7.20	2	0.24	120	240
15	11	7.92	2	0.264	120	240
16	12	8.64	2	0.288	120	240
17	Mono	15.0	--	0.333	180	180

3 TEST RESULTS

3.1 GENERAL

This chapter presents the results of the reversed cyclic testing of the eleven dry (non-stucco) finished cripple walls considered in this testing program. As noted, the key parameters of this phase are exterior finish, height, vertical load, and retrofit condition of the cripple walls. The boundary conditions, loading protocol, anchorage condition, and length of the cripple walls remained constant. As stated in the previous chapter, each cripple wall was nominally 12 ft in length and 2 ft in height. All walls besides one were subjected to a vertical load of 450 lbs/ft, mimicking the gravity load of a typical two-story house, and all walls were constructed with top boundary condition B and bottom boundary condition c; see Schiller et al., [2020(a)]. The remaining wall had the same boundary conditions but was subjected to a vertical load of 150 plf, emulating a one-story home constructed with light building materials; see Section 2.2.2. for details of these boundary conditions. In this chapter evaluates the effects of the various exterior finishes, namely, horizontal siding, horizontal siding over diagonal sheathing, and T1-11 wood structural panels. In addition, the performance of a retrofitted cripple wall as well as variation of height, namely, the 2-ft-tall and 6-ft-tall walls, are evaluated. It is noted that each cripple wall contained a single parameter variation compared with another cripple wall, while the remaining parameters were controlled. This was done to ensure that the difference of results from one test with another would work could be easily compared to determine the effects of the changed parameter on the response of the wall. Table 3.1 presents the variables subject to change for each test. In addition, a pseudo-name is given to each of the specimens for purposes of clarity in the presentation of the results when needed.

Table 3.1 **Variables for each cripple wall tested and specimen pseudo-names.**

Specimen name	Test no.	Description of test	Specimen pseudo-name
A-7	7	Horizontal siding, existing, 2 ft tall	2-ft-tall HS[1]
A-8	8	Horizontal siding, retrofitted, 2 ft tall	2-ft-tall HS (Retrofitted)
A-9	11	Horizontal siding over diagonal sheathing, existing, 2 ft tall	2-ft-tall HS+DSh[2]
A-10	12	Horizontal siding over diagonal sheathing, existing, 2 ft tall	2-ft-tall HS+DSh (Retrofitted)
A-11	9	T1-11 WSP, existing, 2 ft tall	2-ft-tall T1-11
A-12	10	T1-11 WSP, retrofitted, 2 ft tall	2-ft-tall T1-11 (Retrofitted)
A-13	13	Horizontal siding, existing, 6 ft tall	6-ft-tall HS
A-14	14	Horizontal siding, retrofitted, 6 ft tall	6-ft-tall HS (Retrofitted)
A-23	23	T1-11 WSP, existing, 6 ft all	6-ft-tall T1-11
A-24	24	T1-11 WSP, retrofitted, 6 ft tall	6-ft-tall T1-11 (Retrofitted)
A-28	25	Horizontal siding over diagonal sheathing, existing, light vertical load 2 ft tall	2-ft-tall HS+Dsh (Light)

[1] Horizontal siding.
[2] Horizontal siding over diagonal sheathing.

3.2 LATERAL FORCE–LATERAL DISPLACEMENT RESPONSE

This section presents the global lateral force-displacement response of the specimens of interest in this report. For context, the presentation includes an overview photograph sequence of each specimen, followed by the lateral force-displacement hysteresis of the corresponding specimen; see Figures 3.2–3.33. It is noted that both global total and global relative displacement are presented, where the relative displacement accounts for the displacement of the cripple wall only, ignoring displacement between the foundation and the sill plate. In addition, secondary axes are incorporated in each plot to present the lateral load per lineal foot of wall length and the drift (i.e., displacement/cripple wall height). It should be noted that maximum lateral load in the positive and negative directions are identified in each hysteresis. While discussing the individual hysteresis is useful, a cross comparison amongst the various specimens is adopted herein, with particular emphasis on eliciting the impact of the parameters varied. In this regard, a cross-comparison of all dry finished specimens is first provided; subsequently, the effect of individual parameters considered in the Phase 1 matrix are discussed.

3.2.1 Hysteretic Response of all Dry (Non-Stucco) Specimens

Examining the global hysteresis of all specimens indicates that overall, the lateral load–lateral displacement responses for these specimens were nearly all symmetric in the push and pull directions, with the exception of Specimens A-9 and A-28, which were existing cripple walls with horizontal siding over diagonal sheathing exterior finish. The difference in characteristics between these specimens is the amplitude of vertical load applied, namely Specimen A-28 supported a light vertical load. The uncharacteristically asymmetric response of the specimens sheathed with diagonal sheathing and their sensitivity with vertical load amplitude can be attributed to the development of bearing between the sheathing boards upon closure of the gaps between the boards, and, conversely, the opening of the gaps between the boards led to reduction in lateral strength. The behavior of these pair of specimens will be discussed in greater detail below. For other, symmetrically sheathed finishes, in most cases the cripple wall response was nearly symmetric; nonetheless, the lateral force in the push direction was 0–15% larger in the push direction than the pull direction. This can be attributed initial damage induced during loading in the push direction.

In the sections below, the strongest and weakest dry finish specimens are discussed in particular detail, namely, specimens with diagonal sheathing underlain and horizontal siding (only). Subsequently, a synthesis of the key response parameters of strength and drift capacity across all dry finish specimens is presented and discussed.

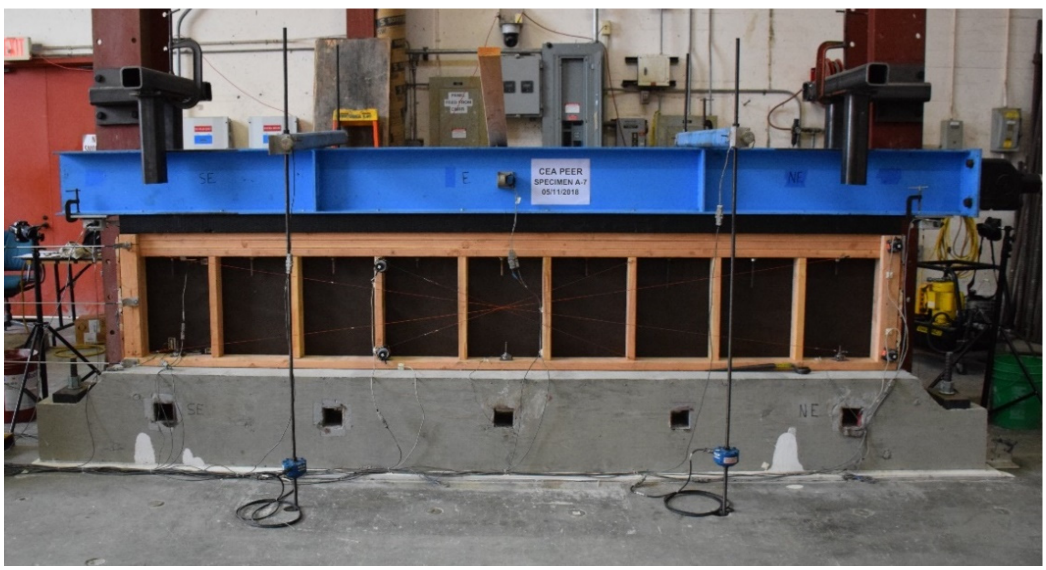
3.2.1.1 Impact of Diagonal Sheathing

For the horizontal siding over diagonal sheathing finished cripple walls, Specimens A-9 and A-28, the lateral strength was around 50% higher in the pull direction than the push direction; see Figures 3.8 and 3.32. As noted, this can be attributed to the orientation of the diagonal sheathing boards. Due to their orientation, the diagonal sheathing boards would separate upon pushing the specimen, whereas when the cripple wall was being pulled on, the diagonal sheathing boards closed-up. At later displacement cycles, the 1/8-in. gaps between the diagonal sheathing boards had completely closed. This caused the sheathing to bear on each other and act as a wood structural panel, resulting in a dramatic increase in capacity at later displacement cycles. It would be expected that the same phenomena would occur for the Specimen A-10, the retrofitted counterpart of Specimen A-9; however, the relative contribution of the diagonal sheathing was suppressed in the presence of the retrofit and dictated the response. In fact, the retrofit diagonal sheathing specimen was so strong, the case of Specimen A-10 failure in all seven anchor bolts occurred, prior to it gaining the added capacity due to the gaps in the diagonal sheathing closing. As a result, the response of Specimen A-10 was close to symmetric. It should be noted that Specimen A-9 also failed due to fractures of the anchor bolts, and Specimen A-28 failed due to a cross-grain split along the entire span of the sill plate. Further discussion on the anchor bolt failures in these specimens is presented in Section 3.3.

3.2.2.2 Characteristics of Horizontal Siding Specimens

Comparison amongst the various exterior finish materials shows that the horizontal siding was by far the weakest finish material. The average strength per linear foot between push and pull loading was 174 plf for the 2-ft-tall cripple wall and 93 plf for the 6-ft-tall cripple wall, compared with 1435 plf for the strongest (existing) specimen finishes (horizontal siding over diagonal sheathing). It could have been expected that the lateral strengths would have been similar between the two cripple walls with horizontal siding because the horizontal siding gains its capacity through the moment resistance of the nail couples at each stud, in addition to a small amount of resistance from the friction between the contact of the overlapping shiplap boards. While the 6-ft-tall cripple wall was subjected to three times the moment of the 2-ft-tall cripple wall, it had three times the nail couples, therefore, three times the moment resistance from the nail couples. Since the horizontal siding cripple walls were so weak, the framing provided a large portion of the moment capacity of the cripple walls due to both the withdrawal strength of the nails connecting framing members and the overturning resistance of the studs carrying the vertical load. Section 3.12 presents a static analysis of the two cripple walls with horizontal siding to give insight as to the discrepancy between their lateral load capacities.

(a)



(b)

Figure 3.1 Specimen A-7 pre-test photographs for 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.

(c)

(d)

Figure 3.1 (continued).

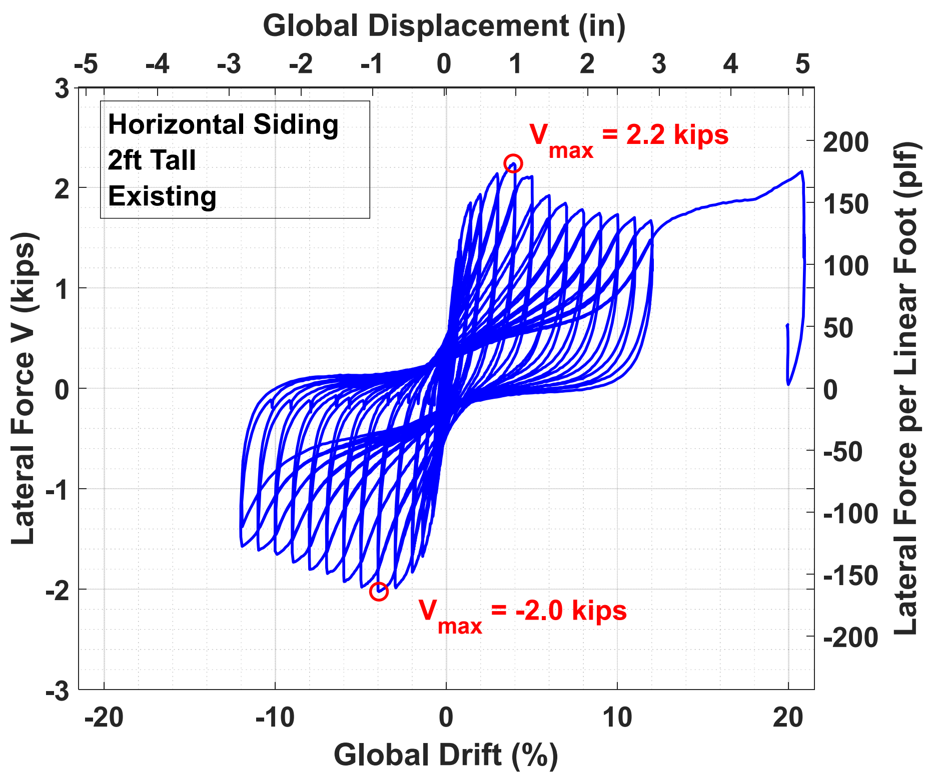


Figure 3.2 **Specimen A-7 lateral force versus *global* lateral drift and displacement hysteresis.**

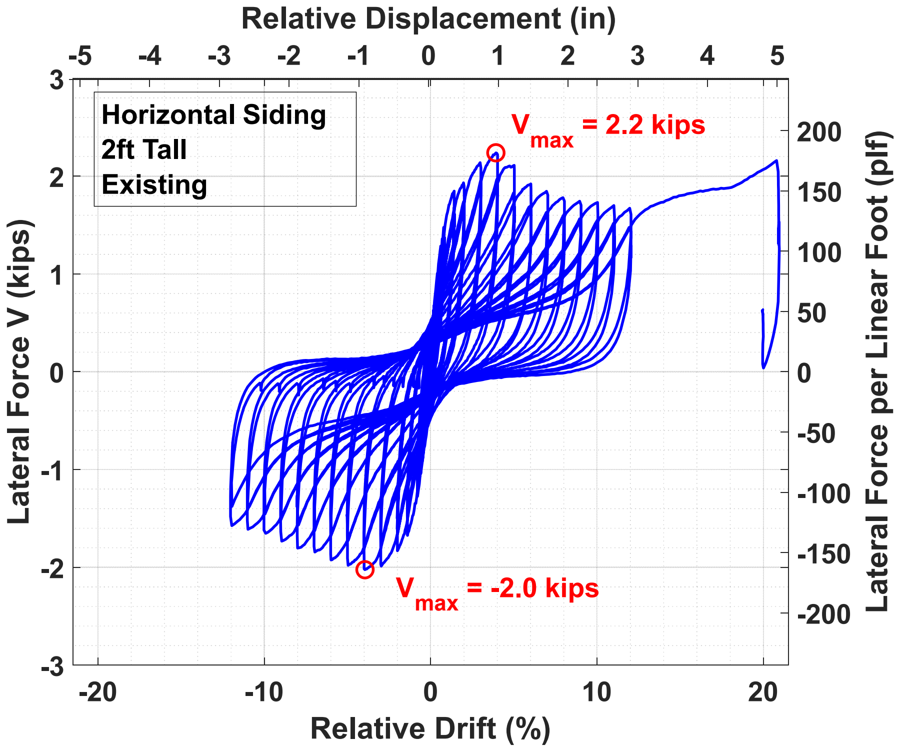
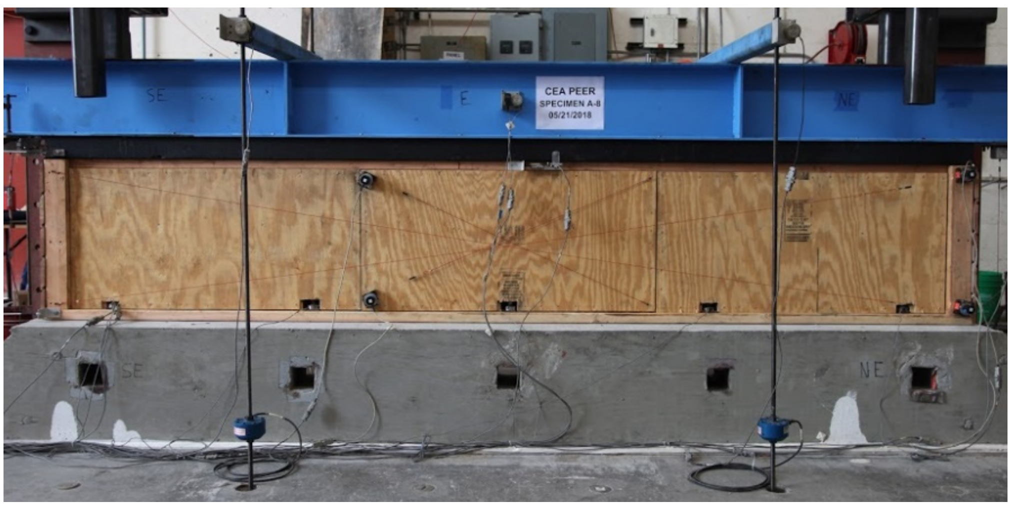


Figure 3.3 **Specimen A-7 lateral force versus *relative* lateral drift and displacement hysteresis.**

(a)



(b)

Figure 3.4 **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) south corner view; and (d) south interior corner.**

(c)

(d)

Figure 3.4 (continued).

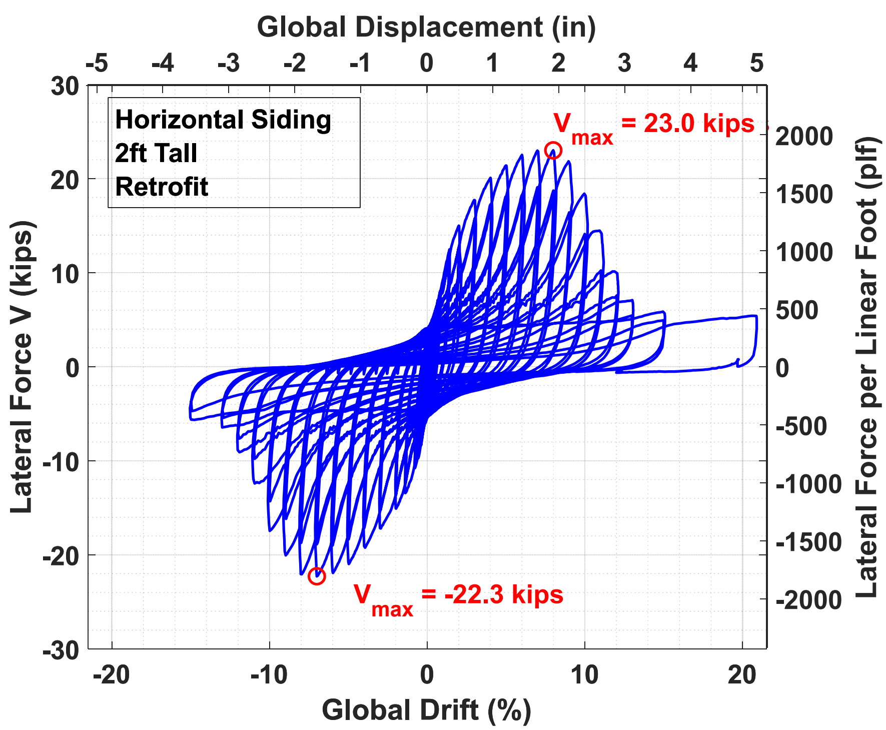


Figure 3.5 **Specimen A-8 lateral force versus *global* lateral drift and displacement hysteresis.**

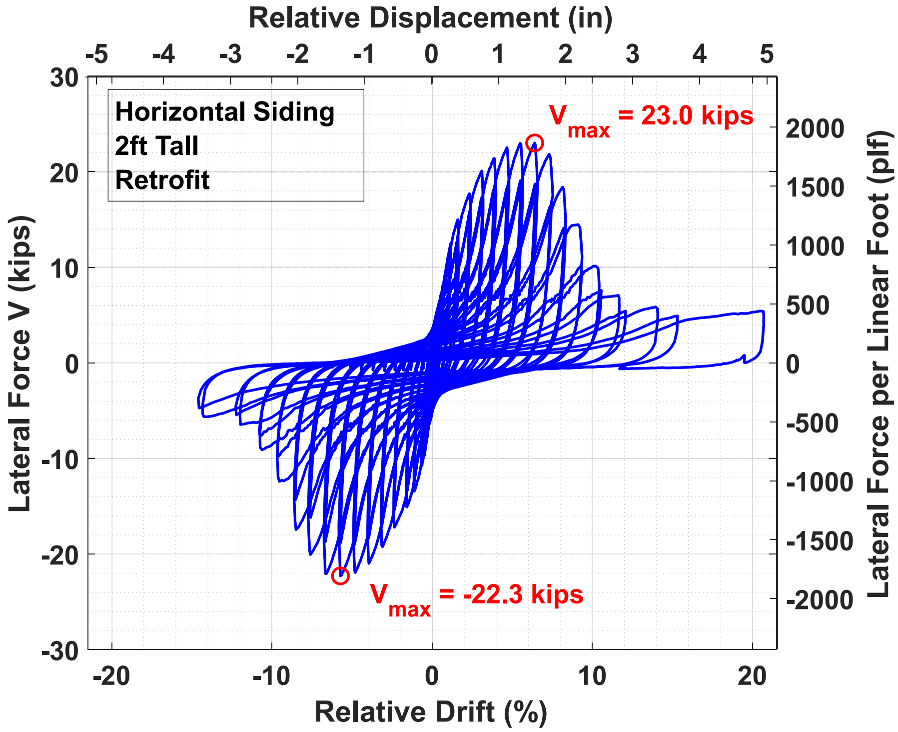


Figure 3.6 Specimen A-8 lateral force versus *relative* lateral drift and displacement hysteresis.

(a)

(b)

Figure 3.7 Specimen A-9 pre-test photographs for the existing 2-ft-tall cripple wall with horizontal siding over diagonal sheathing exterior finish, heavy vertical load: (a) exterior elevation; (b) interior elevation; (c) north exterior corner; and (d) north interior corner.

(c)

(d)

Figure 3.7 (continued).

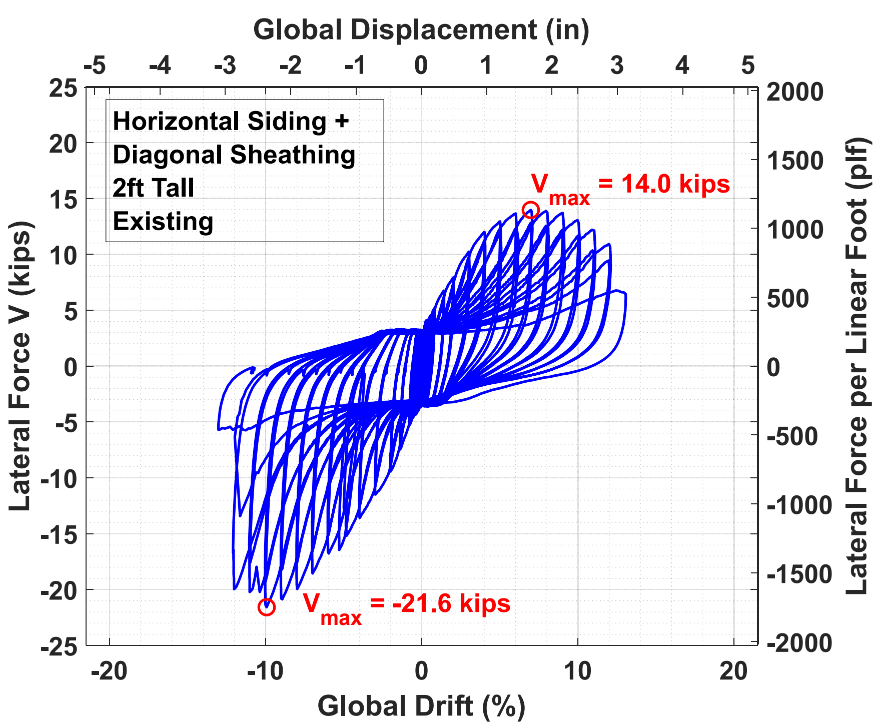


Figure 3.8 Specimen A-9 lateral force versus *global* lateral drift and displacement hysteresis.

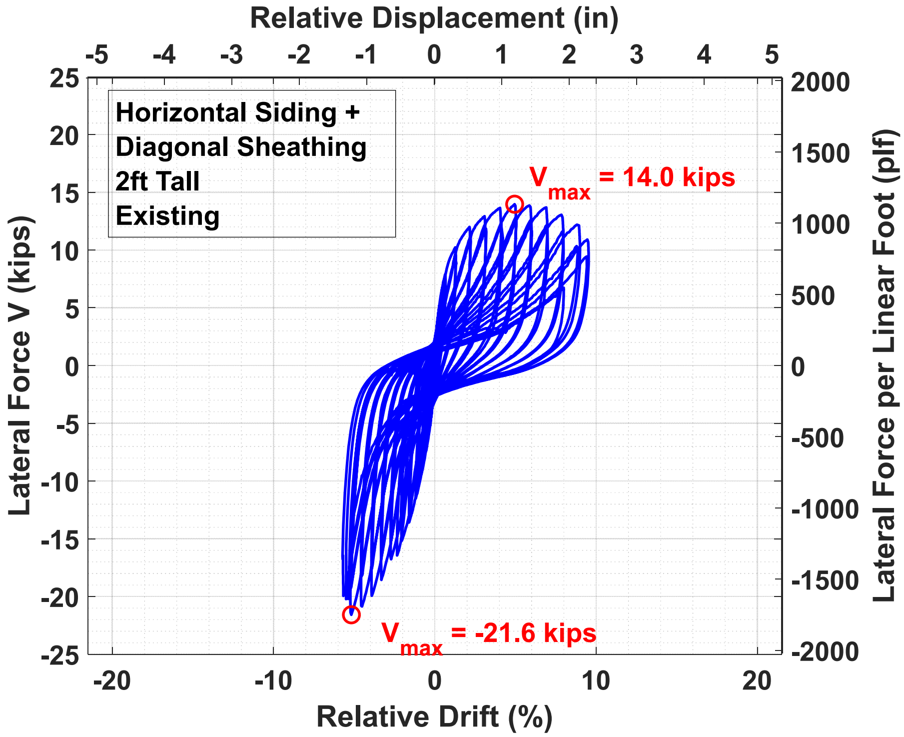


Figure 3.9 **Specimen A-9 lateral force versus *relative* lateral drift and displacement hysteresis.**

(a)

(b)

Figure 3.10 **Specimen A-10 pre-test photographs for the retrofitted 2-ft-tall cripple wall with horizontal siding over diagonal sheathing exterior finish): (a) exterior elevation; (b) interior elevation; (c) north exterior corner; and (d) north interior corner.**

(c)

(d)

Figure 3.10 (continued).

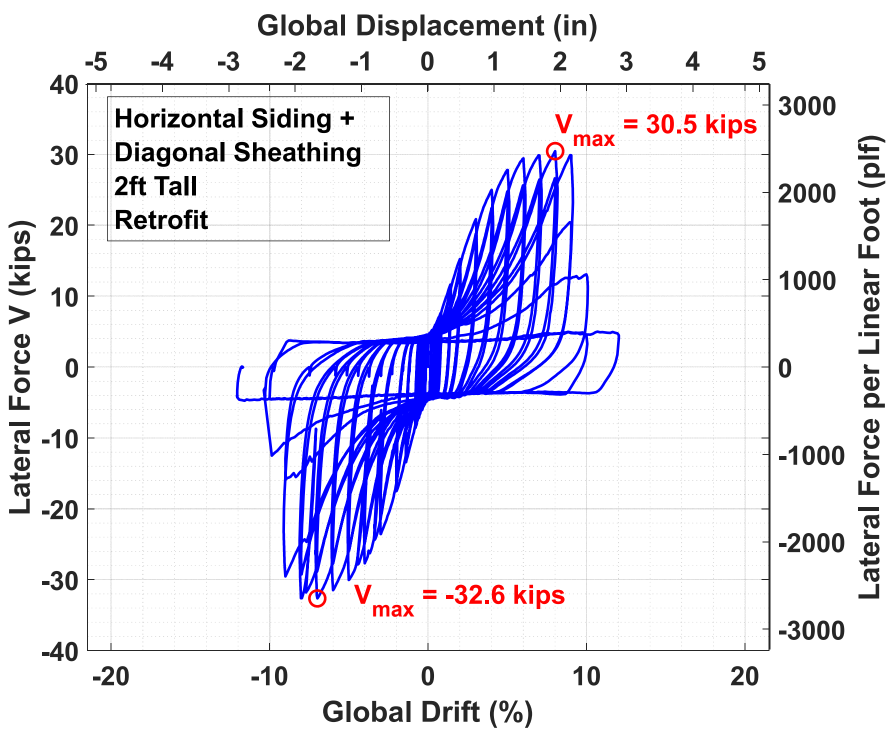


Figure 3.11 **Specimen A-10 lateral force versus *global* lateral drift and displacement hysteresis of.**

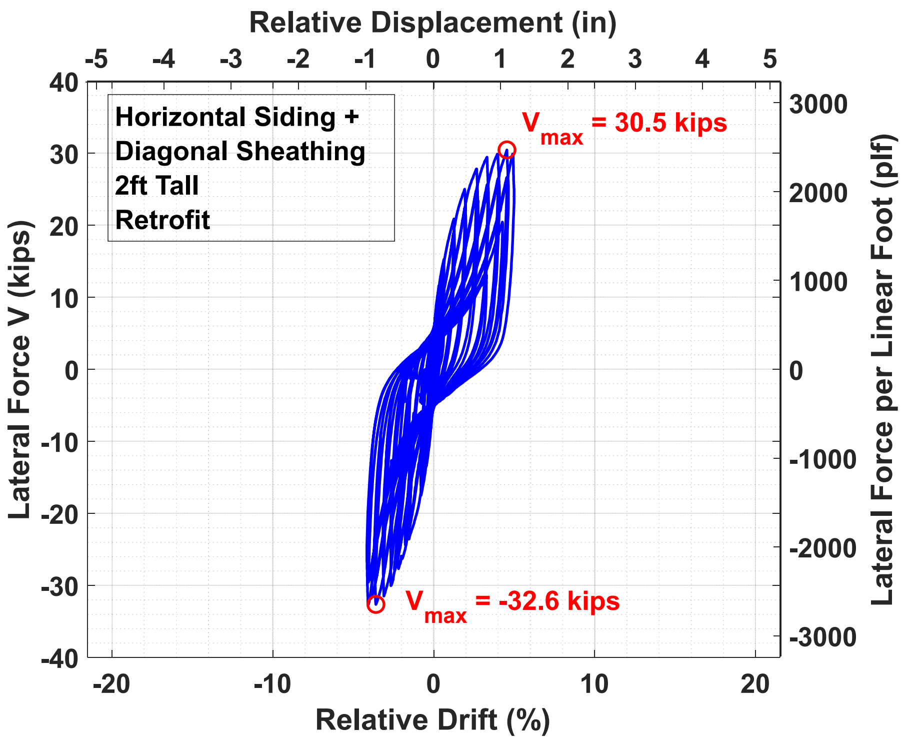
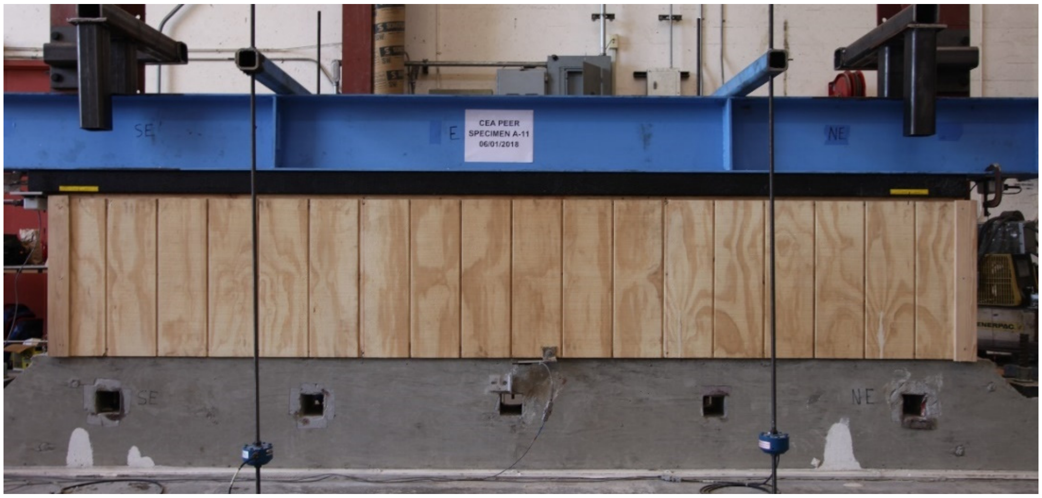
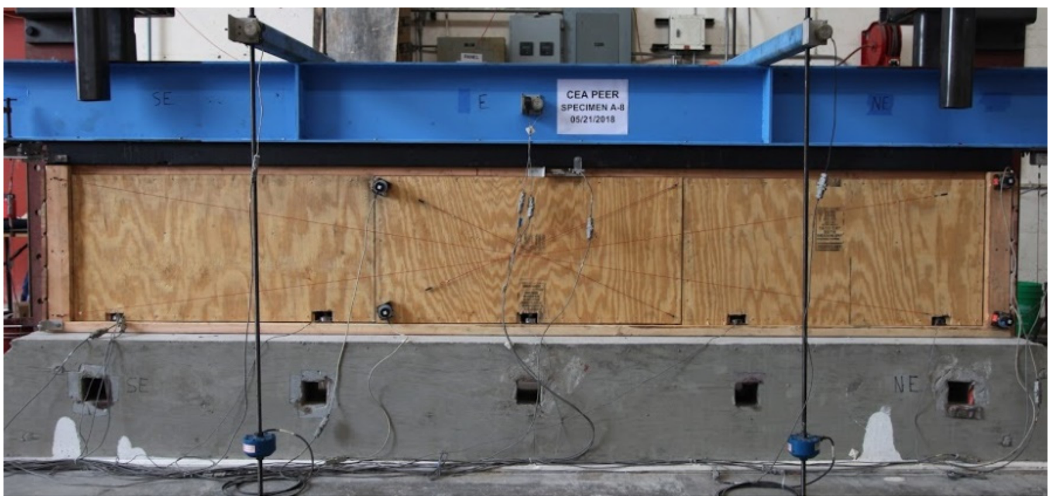


Figure 3.12 **Specimen A-10 lateral force versus *relative* lateral drift and displacement hysteresis.**



(a)



(b)

Figure 3.13 Specimen A-11 pre-test photographs for the existing 2-ft-tall cripple wall with T1-11 wood structural panel exterior finish: (a) exterior elevation; (b) interior elevation; (c) south exterior corner; and (d) nailing detail.

(c)

(d)

Figure 3.13 (continued).

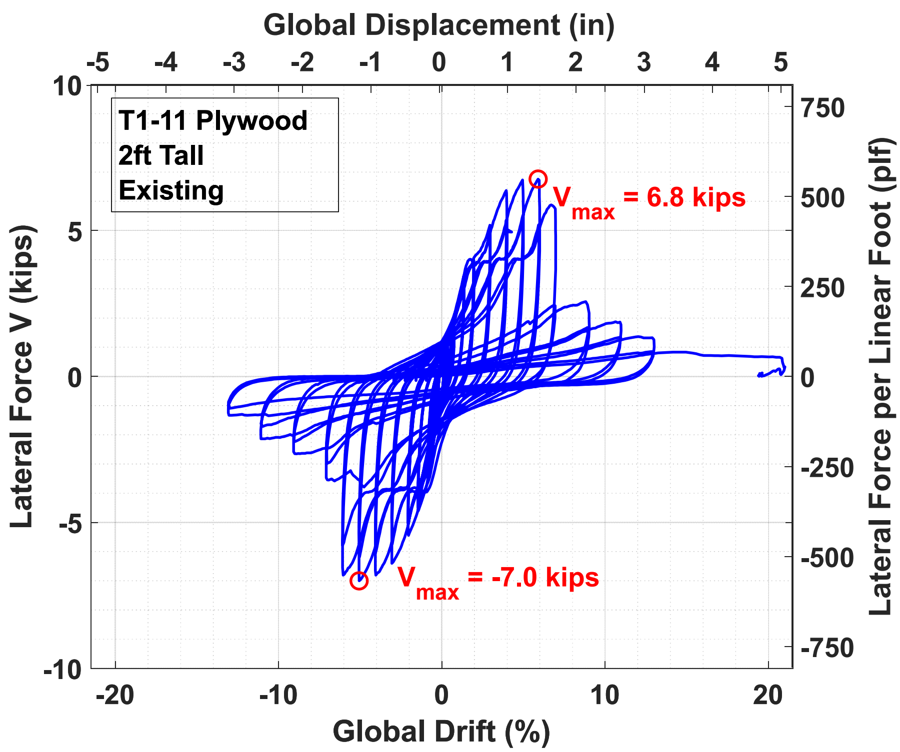


Figure 3.14 Specimen A-11 lateral force versus *global* lateral drift and displacement hysteresis.

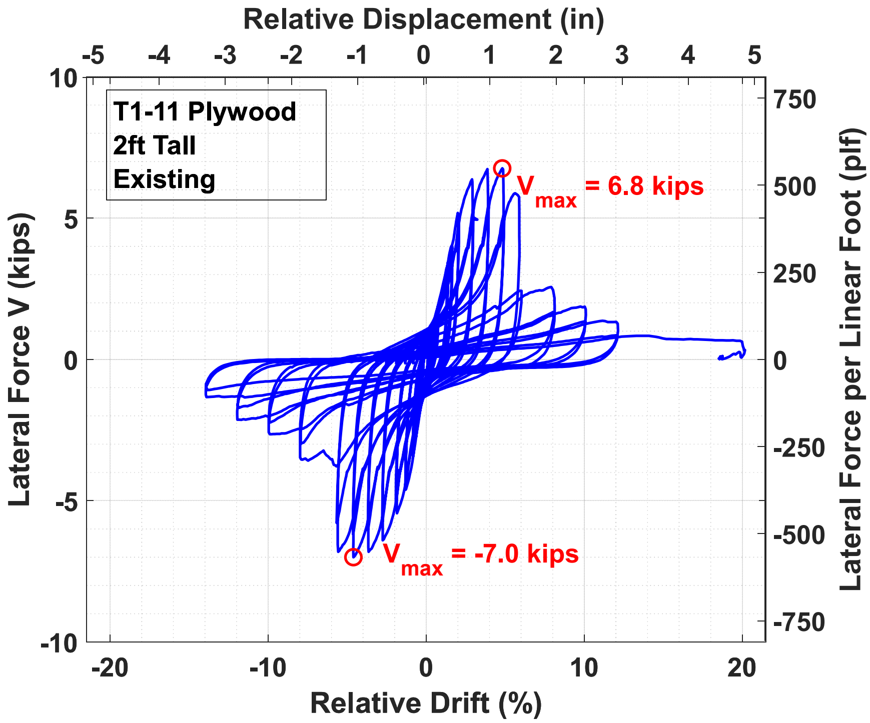
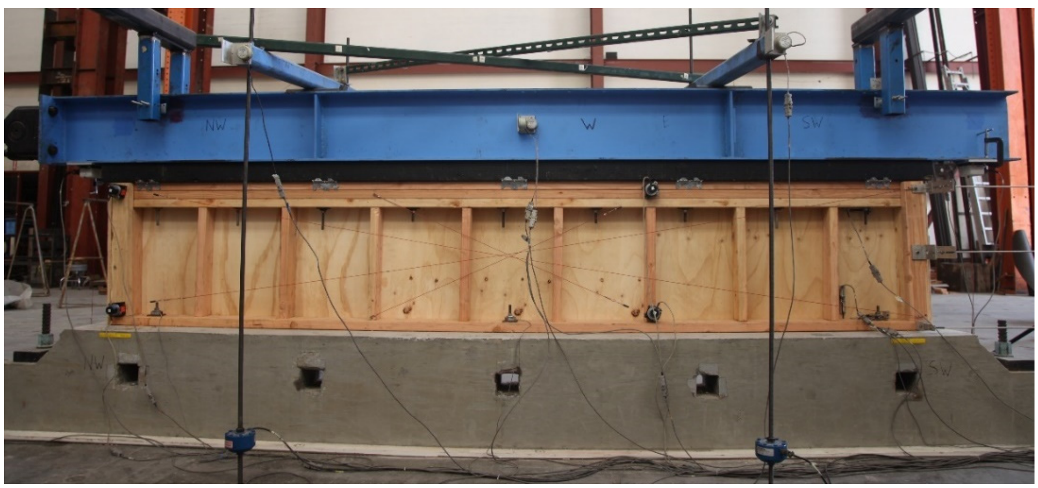


Figure 3.15 Specimen A-11 lateral force versus *relative* lateral drift and displacement hysteresis.

(a)



(b)

Figure 3.16 Specimen A-12 pre-test photographs for the retrofitted 2-ft-tall cripple wall with T1-11 wood structural panel exterior finish: (a) exterior elevation; (b) interior elevation; (c) north exterior corner; and (d) nailing detail.

(c)

(d)

Figure 3.16 (continued).

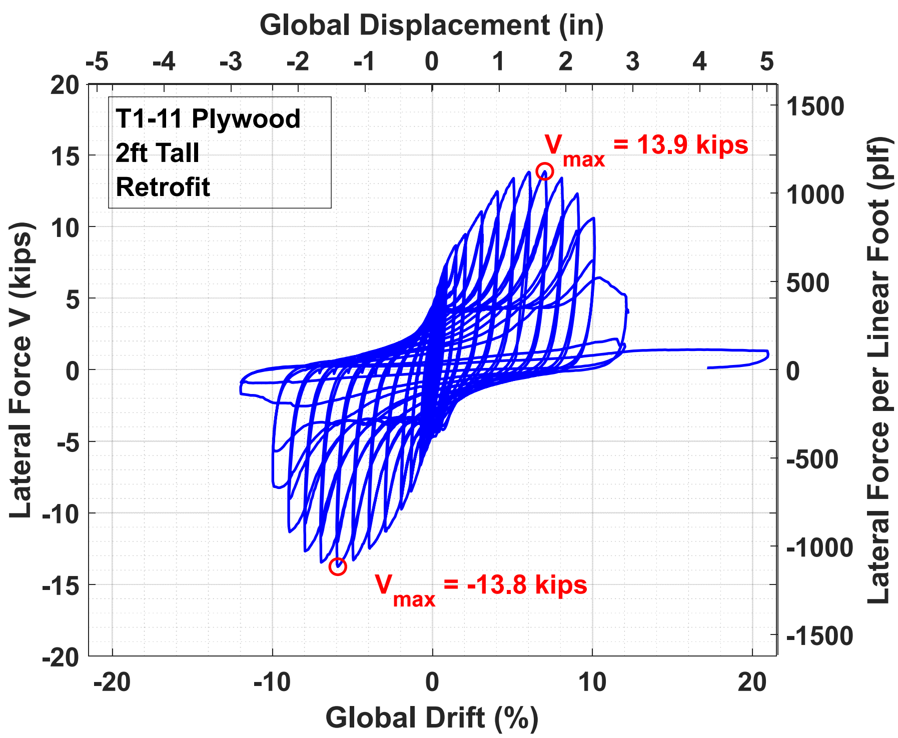


Figure 3.17 Specimen A-12 lateral force versus *global* lateral drift and displacement hysteresis.

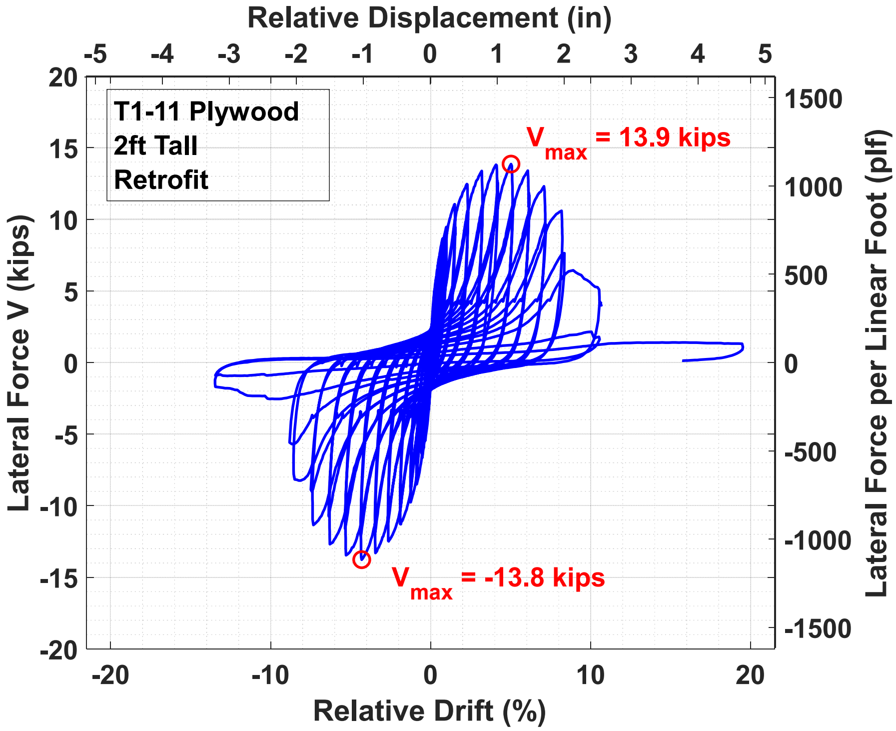


Figure 3.18 **Specimen A-12 lateral force versus *relative* lateral drift and displacement hysteresis.**

(a)

(b)

Figure 3.19 Specimen A-13 pre-test photographs for the existing 6-ft-tall cripple wall with horizontal siding exterior finish: (a) exterior elevation; (b) interior elevation; (c) north exterior corner; and (d) south interior corner.

(c)

(d)

Figure 3.19 (continued).

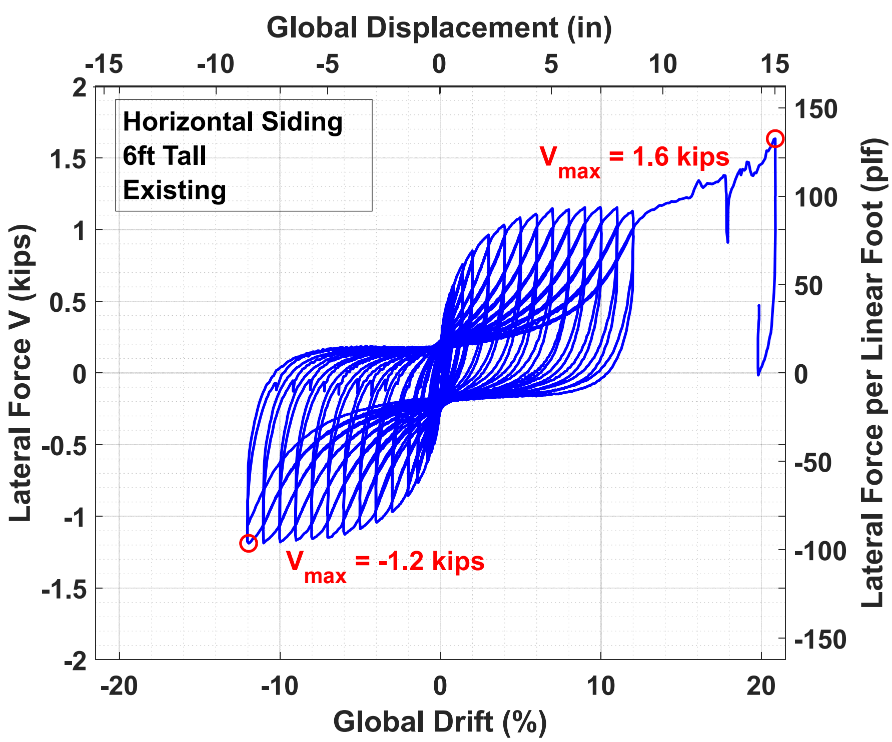


Figure 3.20 Specimen A-13 lateral force versus *global* lateral drift and displacement hysteresis.

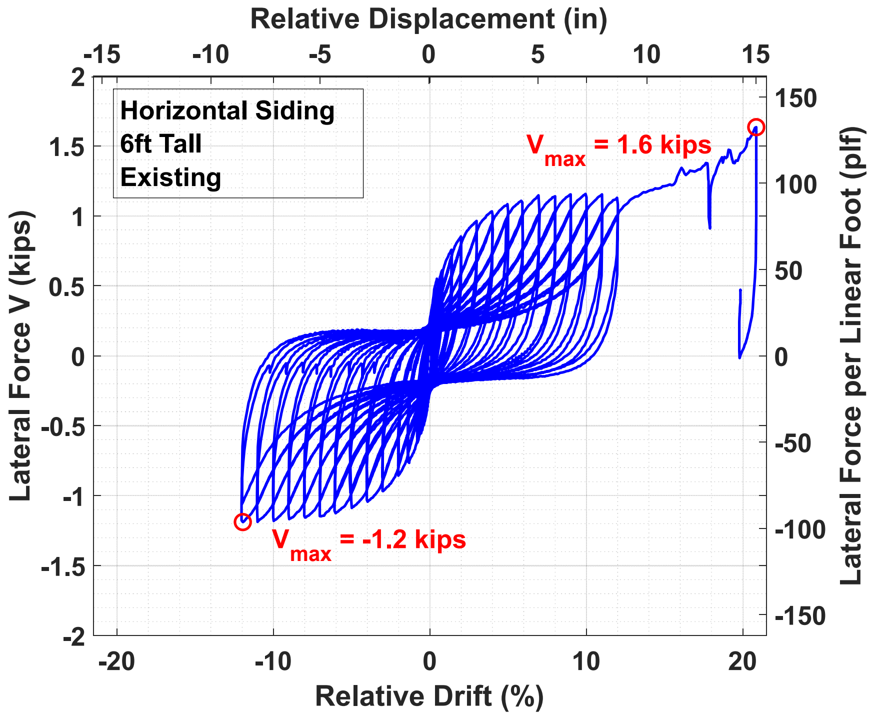
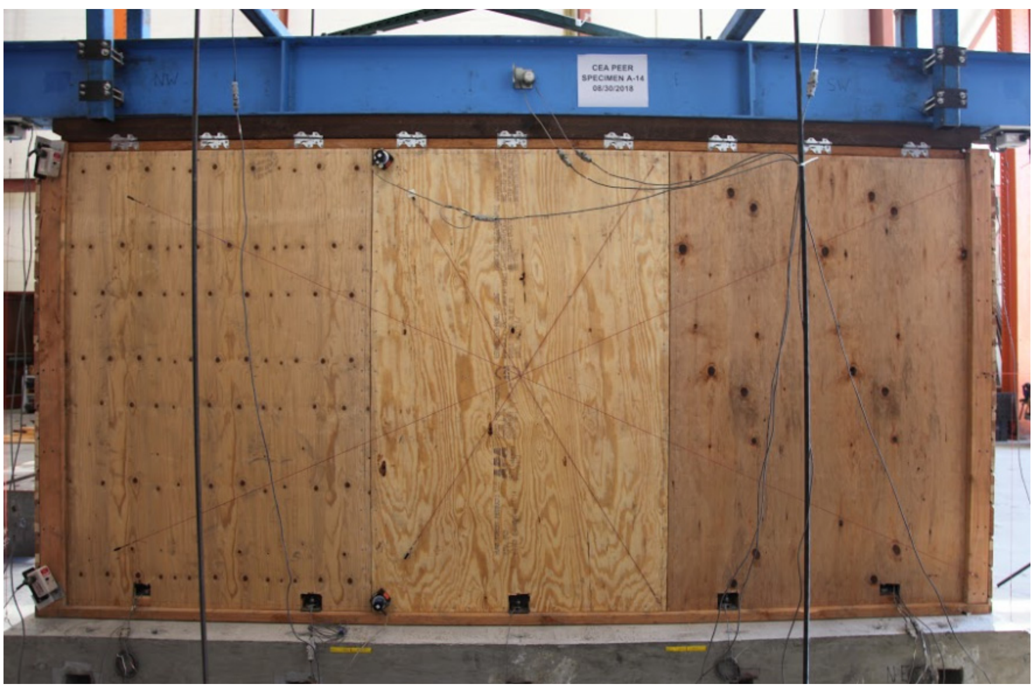


Figure 3.21 **Specimen A-13 lateral force versus *relative* lateral drift and displacement hysteresis.**

(a)



(b)

Figure 3.22 **Specimen A-14 pre-test photographs for the retrofitted 6-ft-tall cripple wall with horizontal siding exterior finish: (a) exterior elevation; (b) interior elevation; (c) north exterior corner; and (d) north interior corner.**

(c) (d)

Figure 3.22 (continued).

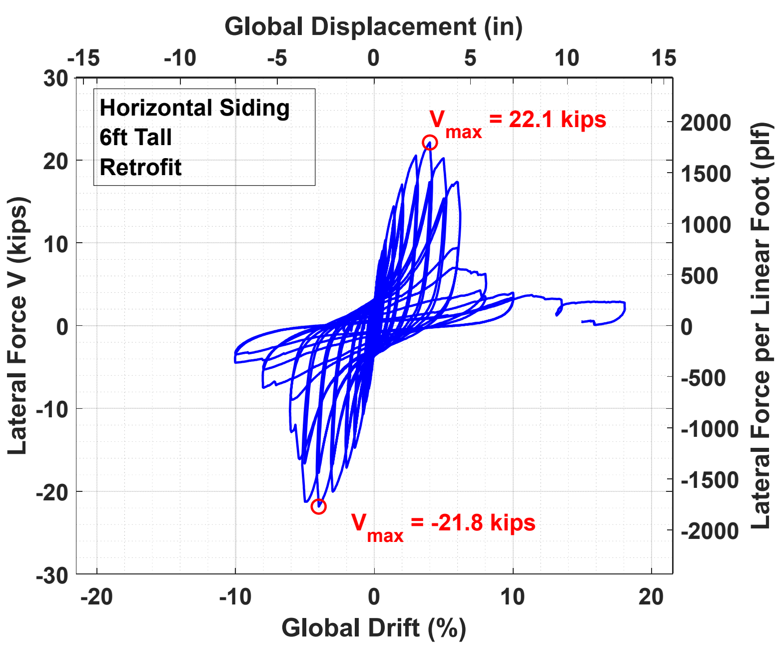


Figure 3.23 Specimen A-14 lateral force versus *global* lateral drift and displacement hysteresis.

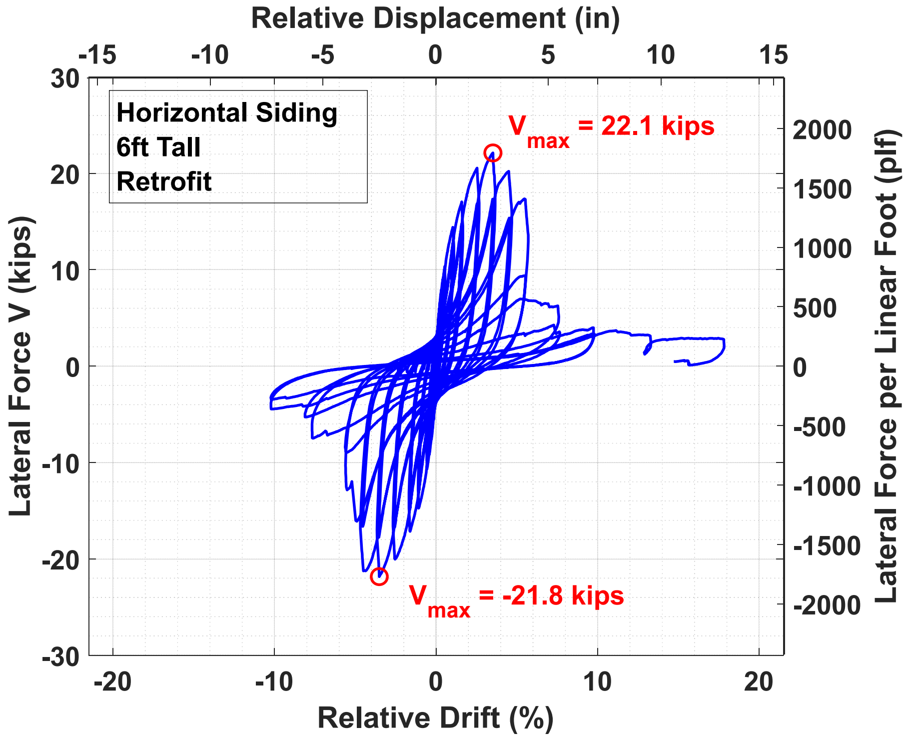
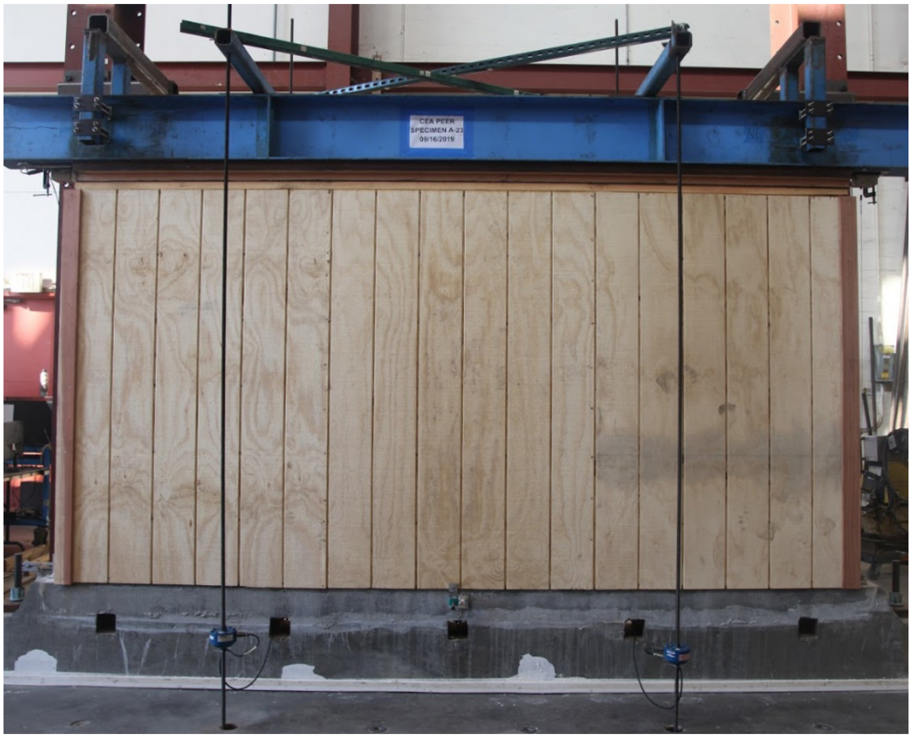


Figure 3.24 **Specimen A-14 lateral force versus *relative* lateral drift and displacement hysteresis.**



(a)

(b)

Figure 3.25 Specimen A-23 pre-test photographs for the existing 6-ft-tall cripple wall with T1-11 wood structural panel exterior finish: (a) exterior elevation; (b) interior elevation; (c) south exterior corner; and (d) south interior corner.

(c)

(d)

Figure 3.25 (continued).

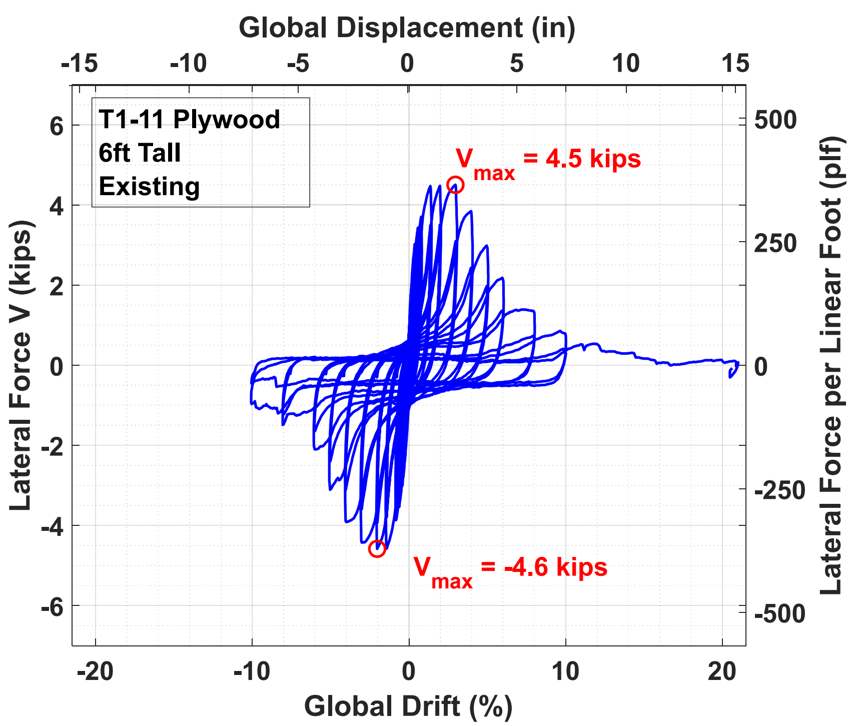


Figure 3.26 Specimen A-23 lateral force versus *global* lateral drift and displacement hysteresis.

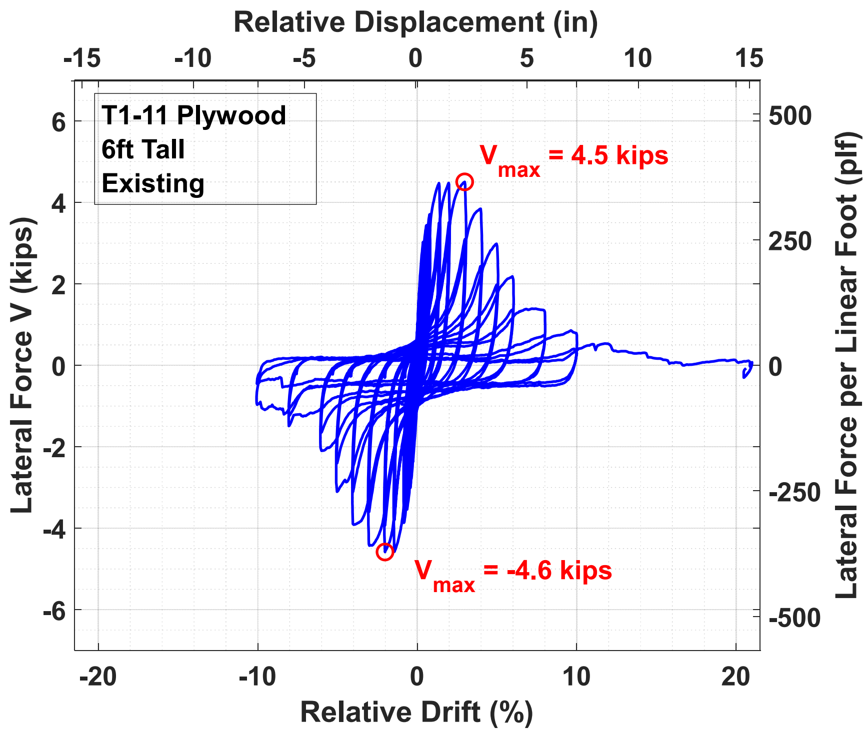


Figure 3.27 **Specimen A-23 lateral force versus *relative* lateral drift and displacement hysteresis.**

(a)

(b)

Figure 3.28 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.

(c)

(d)

Figure 3.28 (continued).

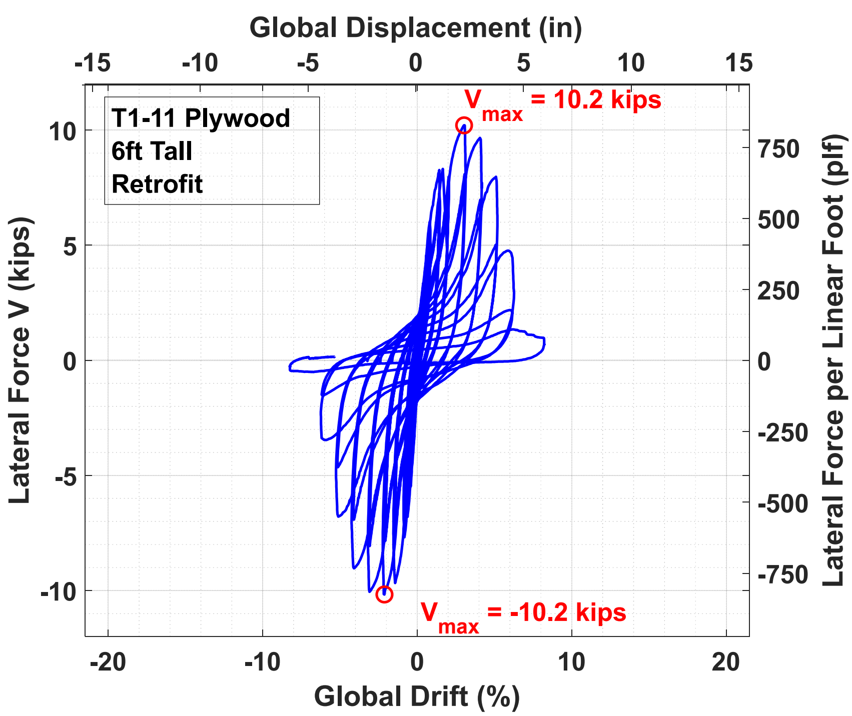


Figure 3.29 **Specimen A-24 lateral force versus *global* lateral drift and displacement hysteresis.**

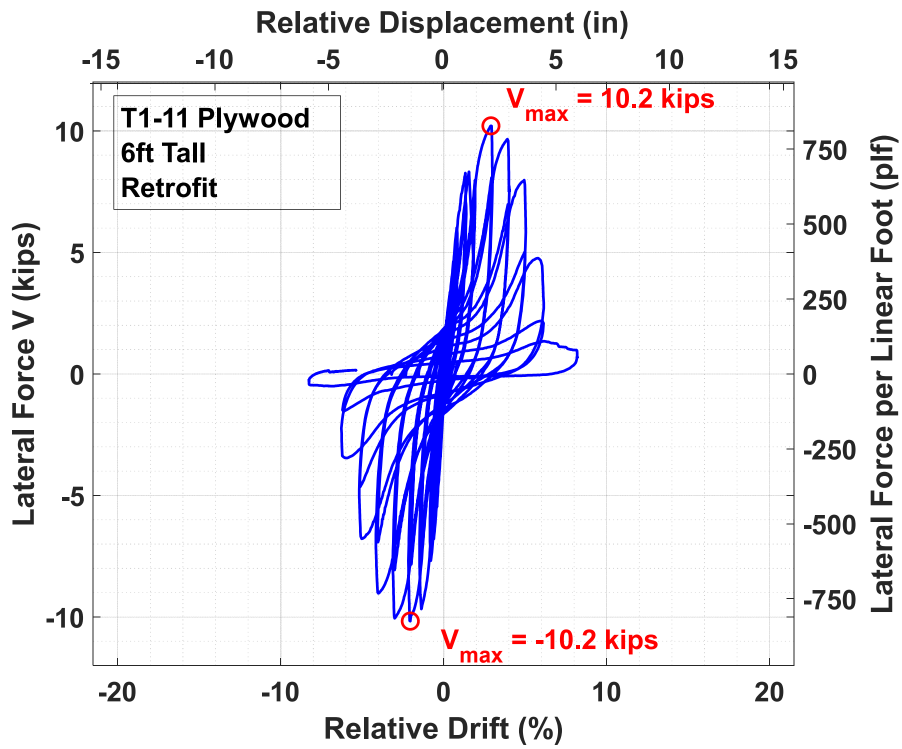


Figure 3.30 **Specimen A-24 lateral force versus *relative* lateral drift and displacement hysteresis.**

(a)

(b)

Figure 3.31 **Specimen A-28 pre-test photographs for the existing 2-ft-tall cripple wall with horizontal siding over diagonal sheathing exterior finish, light vertical load: (a) exterior elevation; (b) interior elevation; (c) north exterior corner; and (d) north interior corner.**

(c)

(d)

Figure 3.31 (continued).

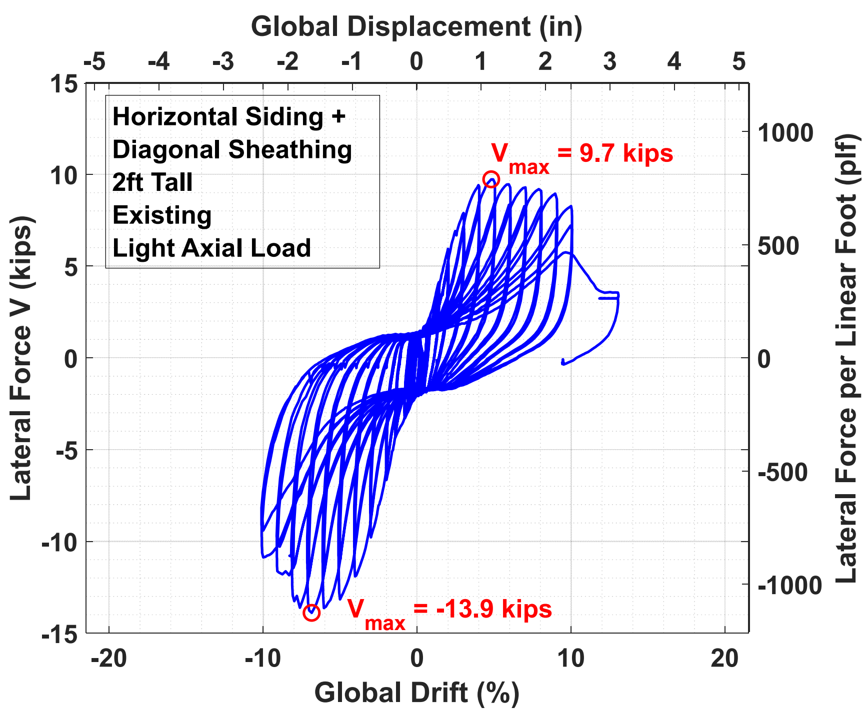


Figure 3.32 Specimen A-28 lateral force versus *global* lateral drift and displacement hysteresis.

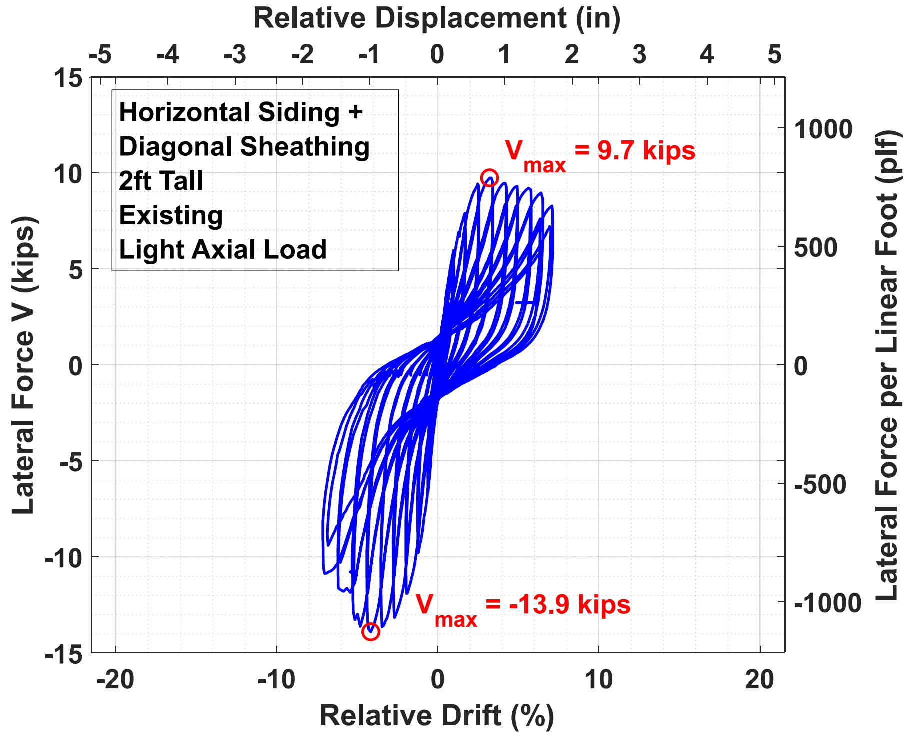


Figure 3.33 Specimen A-28 lateral force versus *relative* lateral drift and displacement hysteresis.

3.2.2 Synthesis of Response of All Dry (Non-Stucco) Specimens

Figures 3.34–3.36 summarize the key response parameters of all dry (non-stucco) specimens, namely, the lateral strength, and drift capacities at strength. Additional parameters of interest are also identified and summarized in the following figures, as defined in Figure 3.37.

Analysis of the summary of lateral strengths (Figure 3.34) indicates that of the three exterior finishes tested, the combination of horizontal siding over diagonal sheathing exhibited by far the highest strengths of any of the finishes. In the existing condition, Specimen A-9 had a lateral strength per linear foot of 1,156 plf in the push direction and 1713 plf in the pull direction. This amounted to a 724% increase in capacity, on average, between the push and pull directions of loading, over Specimen A-7, the existing 2-ft-tall cripple wall with horizontal siding exterior finish, and a 157% increase in capacity over Specimen A-11, the existing 2-ft-tall cripple wall with T1-11 plywood exterior finish. Assuming the horizontal siding contributes 179 plf (per results obtained from Specimen A-8), and assuming the diagonal sheathing contributes the remaining strength, its contribution is 970 plf in the push direction and 1551 plf in the pull direction. On average between push and pull directions, the lateral strength contribution from the diagonal sheathing was 1261 plf, exceeding the average strength of the retrofitted cripple wall with T1-11 plywood exterior finish. While wood structural panels like T1-11 plywood typically provide the largest amount of shear resistance in shear walls and cripple walls, the panels contain most of their nailing on the perimeter, whereas the diagonal sheathing boards contain two nails on each stud, which is an overall increase in the amount of nails across the cripple wall. This caused an increase in capacity of the diagonal sheathing over the T1-11 plywood, regardless of the direction of loading. Comparing the diagonal sheathing to the horizontal siding, the orientation of the diagonal boards (45° across the framing) provided high resistance to lateral movement as the sheathing boards were put into tension and compression based on the loading direction; in contrast, the orientation of the horizontal siding (parallel to the framing) did little to resist the lateral movement. The nails attaching the diagonal sheathing were much more engaged than those attaching the horizontal siding. Specimen A-9, the existing T1-11 2-ft-tall cripple wall, exhibited lateral strengths of 541 plf in the push direction and 574 plf in the pull direction, and Specimen A-23, the 6-ft-tall counterpart, had peak strengths of 375 plf in the push direction and 382 plf in the pull direction. For existing specimens, T1-11 wood structural panels were in the middle in terms of strength of dry finishes tested.

All cripple walls showed significant differences in the global and relative response due to the displacement of the sill plate along the foundation, as demonstrated when comparing Figures 3.35 and 3.36. Three cripple walls tested did not have the capacity to overcome the frictional resistance between the sill plate and foundation. In the previous report, an analysis was made of the required force to initiate sill to foundation displacement. For cripple walls with a 450 plf vertical load, the required force is around 5.5 kips. Three cripple walls, all existing specimens, did not exhibit degradation in their lateral load; thus, it may be assumed their strength was not strictly attained. These include: the 2-ft-tall and 6-ft-tall wall with horizontal siding, and the 6-ft-tall wall with T1-11 wood structural panel specimens. In the case of the horizontal siding specimens, it is noted that their lack of lateral load degradation is due to the relatively low load carrying capacity and correspondingly large lateral displacement. While very large lateral displacements could have caused the specimen to attain a failure mechanism and thus degradation in lateral load, such large displacements would have been an artifact of the test setup.

Overall, the global drifts at strength ranged from 2% to 12% global ratio drift. By omitting the displacement of the sill plate relative to the foundation, the range drastically reduced to 2% to 6.4% drift ratio. This range excludes Specimen A-13, the existing 6-ft-tall cripple wall with horizontal siding exterior finish, whose strength occurred at 11% drift ratio in the push direction and 12% drift ratio in the pull direction (global and relative). Figures 3.20 and 3.21 show the global and relative response of Specimen A-13; it can be seen that the lateral load continued to increase until 12% drift and further increased when a monotonic push was initiated. The choice was made to initiate a monotonic push after the 12% drift ratio cycle as the lateral strength was converging on 1.2 kips. Although this decision did not strictly follow the loading protocol, which requires a monotonic push be initiated upon attainment of an 80% reduction in lateral load, it allowed attainment of comparable drift amplitudes of prior specimens (i.e., ~20%) while also minimizing the potential for damage to instrumentation. During this push, the lateral load increased 45% up to 1.6 kips. It should be noted that the responses were corrected for "P-Δ" effects from the applied vertical load, meaning that the lateral contribution of the vertical load, which is inherent due to the mechanism of vertical load application, was accounted for in the data in the presented results. The increase in load is largely due to the gaps between the siding boards closing at large displacements. While the existing 2-ft-tall cripple wall with horizontal siding exterior attained lateral strength at 4% drift with 186 plf in the push direction and 4% drift ratio with 162 plf in the pull direction, this specimen maintained lateral strength at 11% drift with 89 plf in the push direction and 12% drift ratio with 97 plf in the pull direction, respectively, which was only a reduction of 51% and 40% capacity in the push and pull directions, respectively; see Figure 3.2. In addition, the secant stiffness for both existing cripple walls with horizontal siding were the lowest of any specimens. Note that the secant stiffness is defined as the slope extending from the origin to a point on the pre-peak portion of the envelope curve that is equal to 80% of the maximum lateral load for the relative displacement response; see Figure 3.37, which also provides graphical interpretations of relative displacement at 80% of strength, pre-peak, and relative displacement at 40% strength, post-peak. The results of both existing cripple walls with horizontal siding demonstrate the large flexibility of this type of finished wall. It is noted for contrast, that by 12% relative drift, all other cripple wall had lost at least 80% of their capacity or failed due to fractured anchor bolts or cracked sill plates.

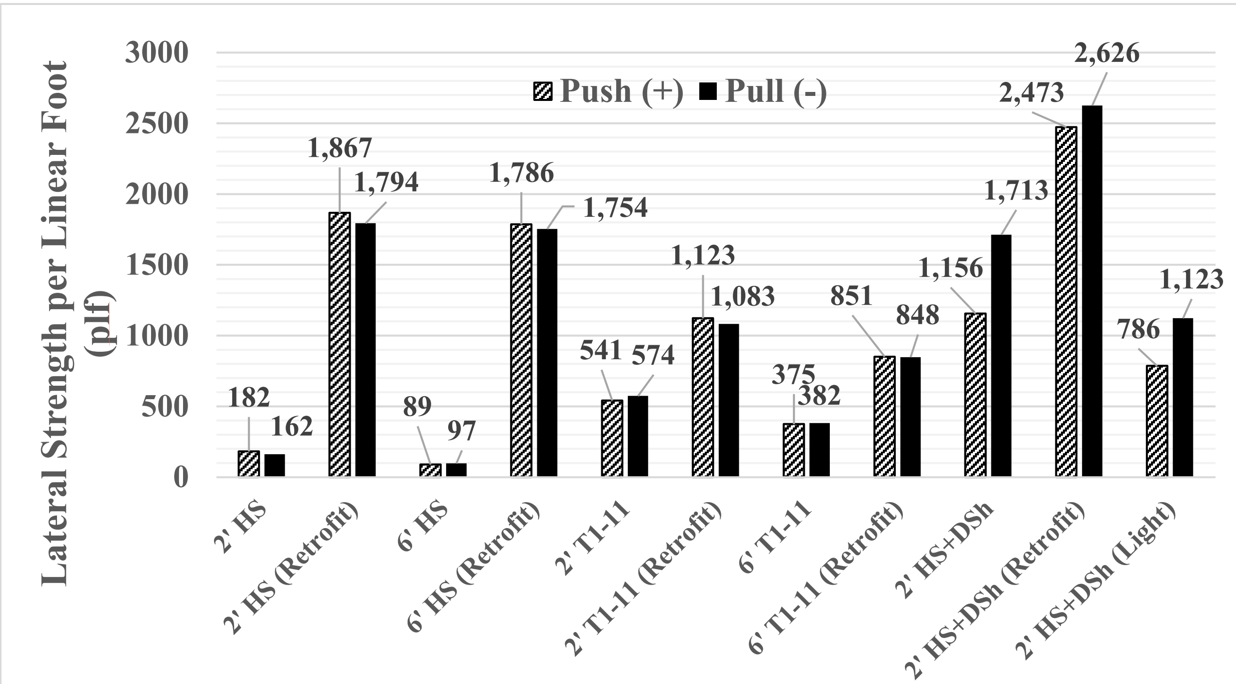


Figure 3.34 Comparison of lateral strength per linear foot of cripple walls.

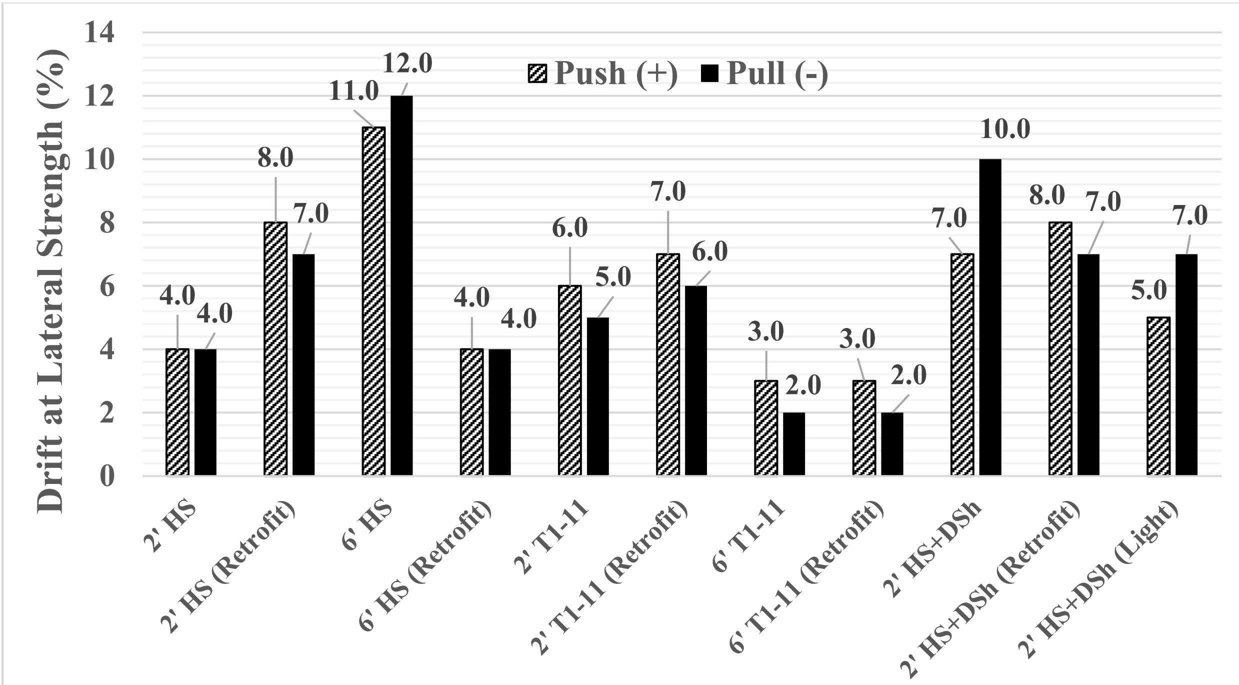


Figure 3.35 **Comparison of *global* drifts at lateral strength.**

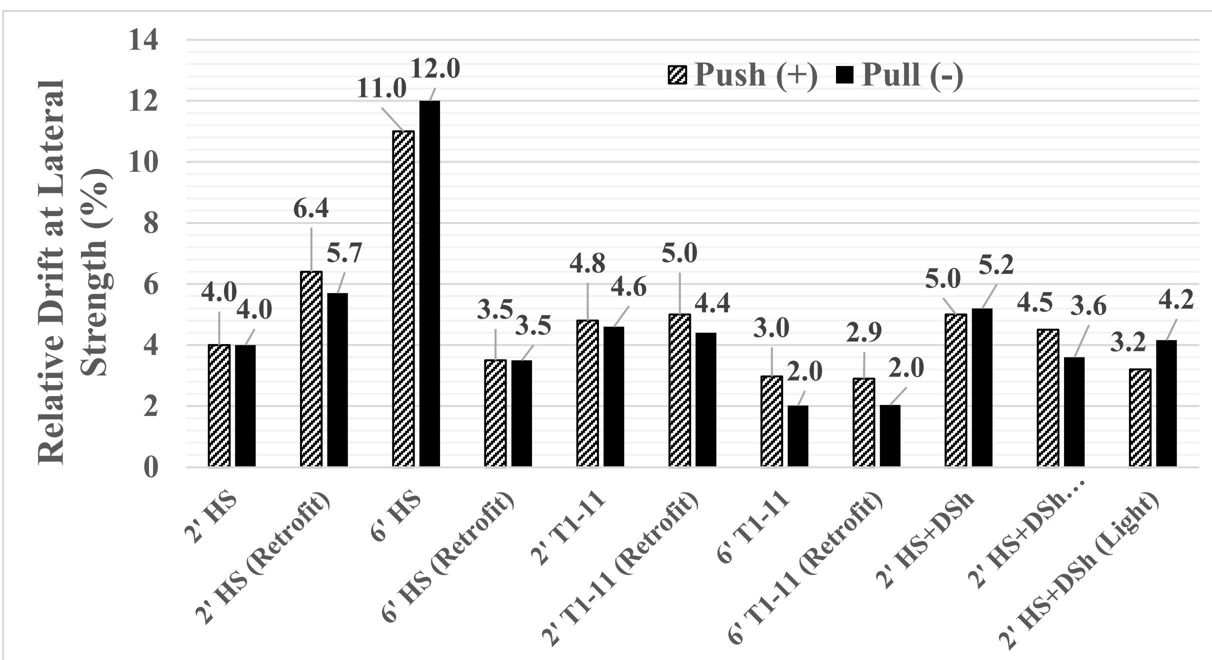


Figure 3.36 **Comparison of *relative* drifts at lateral strength.**

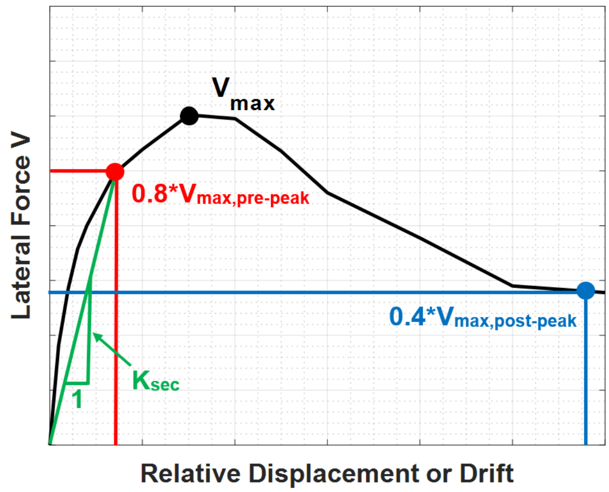


Figure 3.37 **Schematic defining key parameters cross-compared amongst specimens in report: initial secant stiffness, relative drift at 80% lateral strength (pre-strength), and relative drift at 40% lateral strength (post-strength) from a monotonic envelope of the response.**

Figure 3.38 shows the relative drift at 80% of the pre-peak strength. Cross comparison of all dry finish specimens shows that all of the cripple walls achieved 80% strength in at least one direction between 1.5% and 2.3% drift ratio. The 6-ft-tall cripple walls with the T1-11 finish gained strength much earlier, from 0.7–1.3% drift ratio, while the existing 6-ft-tall cripple wall with horizontal siding was at 80% strength by 3% drift ratio. This is an important metric for understanding the behavior of the cripple walls because it provides a sense of how much displacement is required to achieve most of the capacity.

Figure 3.39 provides the relative drift of cripple walls at 40% of strength post-peak. Eight of the eleven dry finished specimens had a 60% drop in strength. Both existing cripple walls with 2-ft-tall and 6-ft-tall horizontal siding never crossed this threshold. The 2-ft-tall specimen began losing strength after 4% drift, but only lost 40% strength by 12% drift. The 6-ft-tall specimen never had a reduction in load during the entire loading protocol. It should also be noted that the cripple wall with horizontal siding over diagonal sheathing and light vertical load lost load abruptly due to a crack forming along the entire length of its sill plate. There was a large range in the relative drift ratios where this criterion was attained: from 2.2% to 10.6%. The most consistent response when considering the retrofit and existing specimens amongst the exterior finishes were the T1-11 specimens, especially for the 6-ft-tall specimens. For both the existing and retrofitted cripple walls, a 60% loss in strength occurred between 6.0% and 6.1% drift. For the T1-11 2-ft-tall specimens, it occurred between 7.9% and 9.9% drift. It should be noted that the horizontal siding over diagonal sheathing specimens only achieved a 60% loss in strength due to its anchor bolts fracturing; these values would have been higher had loss of strength resulted from the actual wall losing capacity.

Figure 3.40 shows the initial secant stiffness for all specimens. It should be noted that the initial stiffness values provided in Figure 3.40 are based on relative drift values. A comparison of all dry specimens demonstrates that the secant stiffness of the existing cripple wall with horizontal siding over diagonal sheathing and heavy vertical load, namely, Specimen A-9, was the largest, with a value of 27.2 kip/in in the push direction and 21.6 kip/in in the pull direction. In contrast, the existing 2-ft-tall cripple wall with horizontal siding, namely Specimen A-7, had the lowest secant stiffness, with values of 4.7 kip/in in the push direction and 4.4 kip/in in the pull direction. The T1-11 specimens observed a secant stiffness of 11.6 kip/in. and 15.4 kip/in. for the existing condition in push and pull directions, respectively. On average between both directions of loading, the increase in secant stiffness from horizontal siding to horizontal siding over diagonal sheathing was nearly 600% and from T1-11 wood structural panels to horizontal siding over diagonal sheathing was around 80%, showing diagonal sheathing alone contributes more to strength and stiffness of any of the dry finish materials. It would be expected that a taller cripple wall would be more flexible than a shorter one; this is confirmed in a comparison of the existing 2-ft- to the 6-ft-tall specimens. For the horizontal siding finish, there was nearly a 90% reduction in stiffness from the 6-ft- to the 2-ft-tall specimen, and for the T1-11 finish there was nearly a 50% reduction in stiffness from the 6-ft- to the 2-ft-tall specimen.

An additional important response parameter, which helps characterize the wall specimen's capacity, is its drift at 80% post-strength. This drift amplitude may be considered important in terms of characterizing the propensity of the specimen towards a global failure mechanism, as the wall's strength will only continue to degrade beyond this amplitude. Hereafter, the drift amplitude at 80% post-strength is referred to as the drift defining near failure. Moreover, when such a load drop is realized, a monotonic push would be implemented in the loading protocol.

For both existing cripple walls with horizontal siding exterior finishes, an 80% drop in load never occurred. Even when the monotonic push was initiated after the 12% drift amplitude, both cripple walls showed an increase in capacity from their last drift cycle due to the gaps between the siding boards closing-up and the boards beginning to bear on one another. For Specimen A-9 (horizontal siding over diagonal sheathing with a heavy vertical load), an 80% drop in load occurred when all of the anchor bolts fractured by the 12% drift ratio cycle. For the same type of cripple wall with a light vertical load, the failure of the sill plate occurred at the 10% drift ratio. It is evident that diagonal sheathing provides so much strength that the anchor bolts and sill plate became the weak point of the wall. The difference in failure mode, anchor bolt fracture versus sill plate splitting, is due to the amount of vertical load. With a light vertical load, the wall had less resistance to uplift. In combination with the diagonal sheathing boards displacing upward during loading and only fastened to one side of the sill plate, large forces were experienced by the sill plate, which led to a cross-grain crack along the entire span of the sill plate. Chapter 5 discusses the damage characteristics of the cripple walls throughout the testing. A discussion of retrofitted specimens is given in Section 3.7.

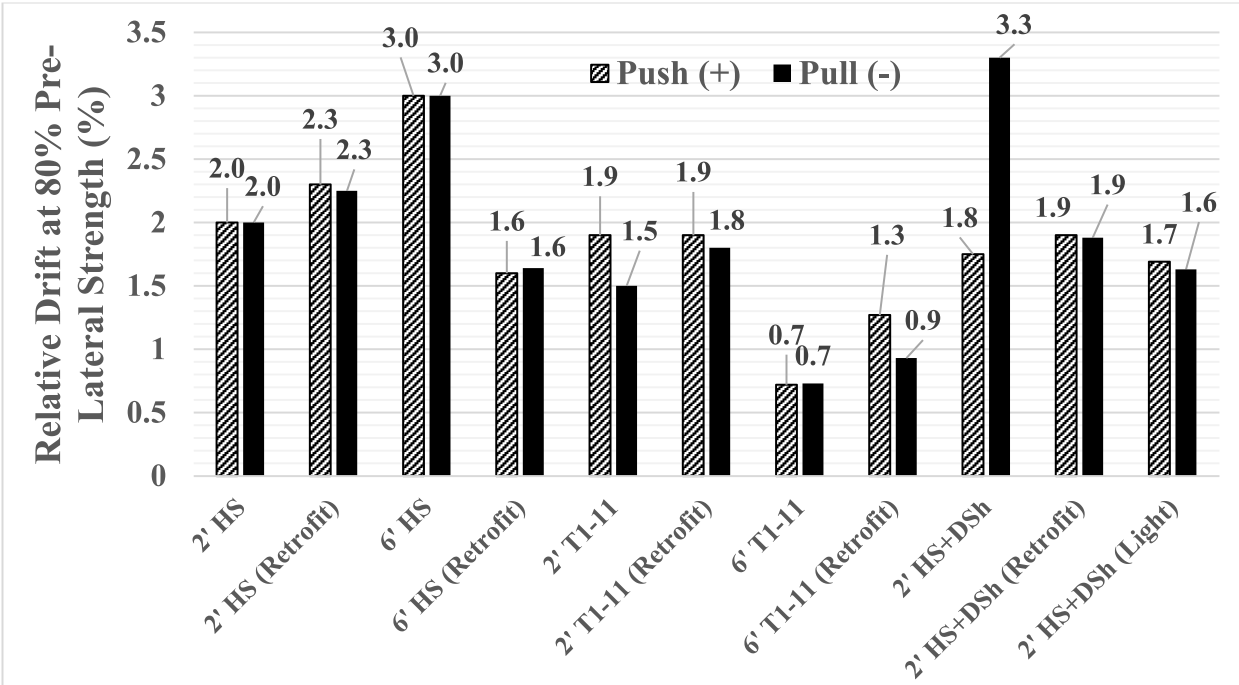


Figure 3.38 **Comparison of *relative* drift at 80% lateral strength, pre-strength (0.8 V_{max}).**

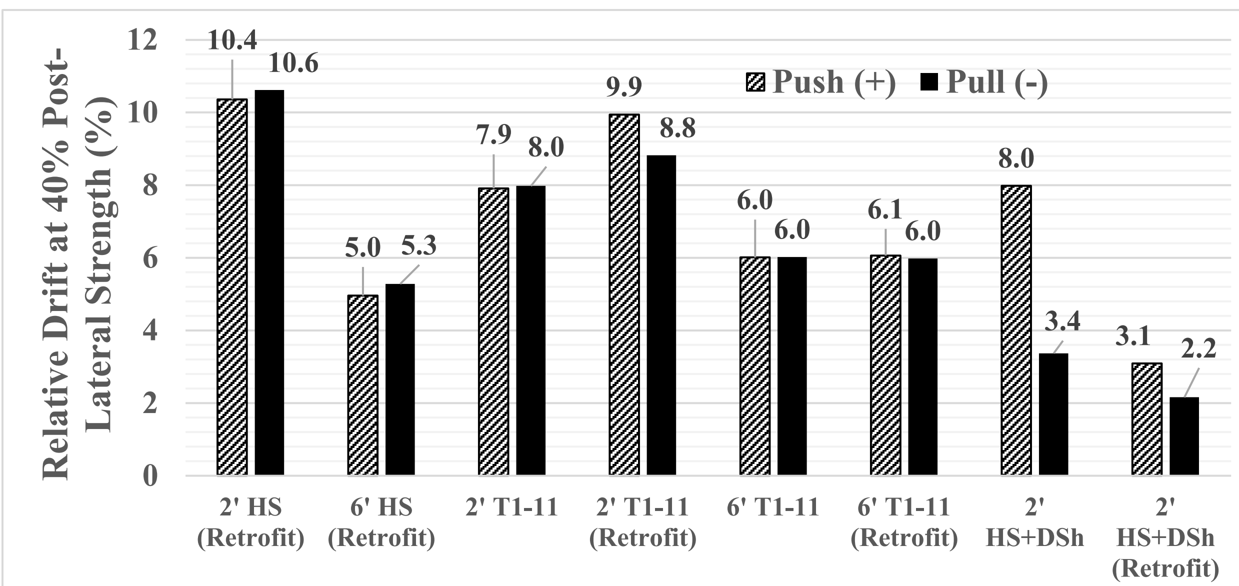


Figure 3.39 **Comparison of *relative* drift at 40% lateral strength, post-strength (0.4 V_{max}); note that select specimens did not observe a drop in lateral load and thus are not included in this plot.**

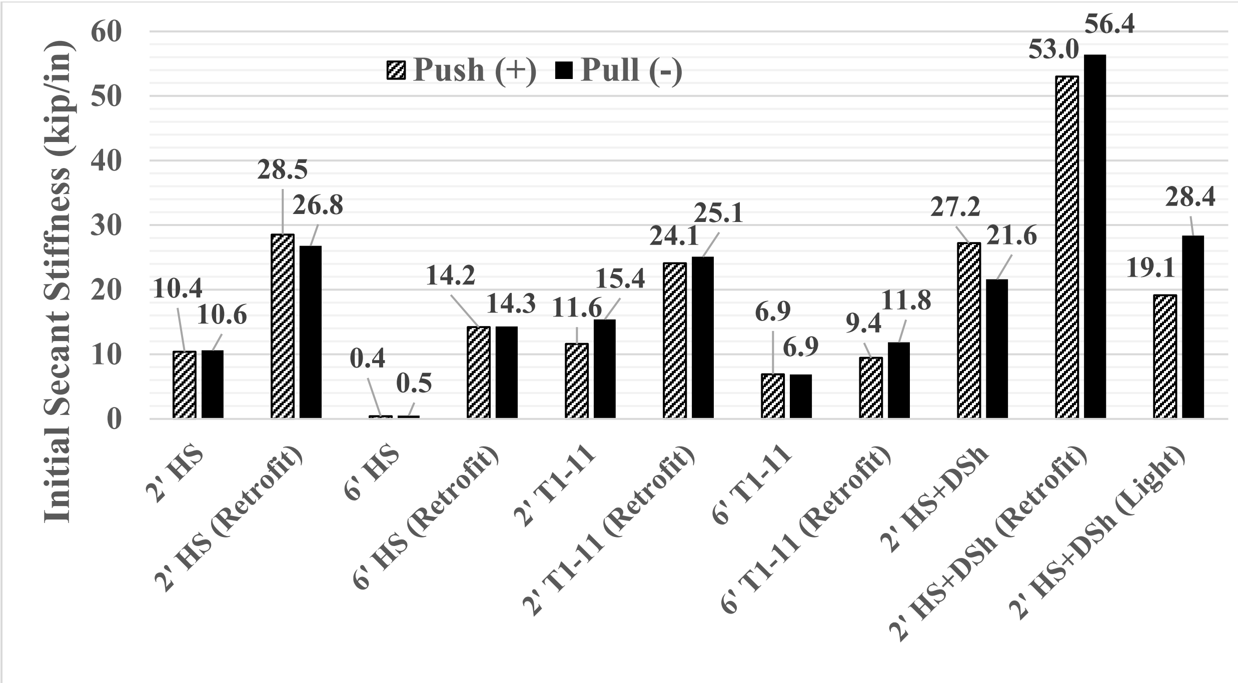


Figure 3.40 Secant stiffness associated with the *relative* drift at 80% pre-strength.

3.3 SILL PLATE DISPLACEMENT RELATIVE TO FOUNDATION

In addition to the lateral response, it is important to know where the contributions of the displacement are coming from. As shown in figures in Section 3.2, often the global response–displacement of the sill plate relative to the foundation and displacement of the cripple wall structure—dramatically differs from the relative response—displacement of the cripple wall structure only. For all cripple walls tested, the anchor bolt holes were oversized by 1/4 in. besides epoxied anchor bolts that were added during retrofitting. This is a common construction practice in California wood-frame dwellings because it alleviates some of the precision needed to frame walls, ultimately leading to quicker construction and is prone to fewer mistakes. Because the anchor bolt holes are oversized, there is less resistance to sliding of the sill plate on the foundation, as the anchor bolts will not immediately resist the sliding. The resistance to this sliding will initially come from the frictional resistance between the bottom of the sill plate and the top of the foundation. It should be noted that the foundation has a smooth trowel finish, which would result in a lower resistance to sliding compared with a foundation with a rougher finish. Through a static analysis, it can be predicted when sliding should occur based off the normal force provided by the cripple wall with the imposed vertical load and the coefficient of friction between wood and concrete. Through the tests in Phase 1 and Phase 2, significant sliding was initiated around 5–6 kips of lateral load. This value varies slightly as a function of the weight of the cripple wall, as the imposed vertical load was kept constant for all walls.

Of the eleven cripple walls tested, eight underwent considerable displacements of the sill plate during loading. The three cripple walls had negligible displacement of the sill plate relative to the foundation: (1) existing Specimens A-7 and A-13 finished with horizontal siding; and (2) Specimen A-23, an existing 6-ft-tall cripple with T1-11 wood structural panel exterior finish. For these cripple walls, the lateral load imposed on the cripple wall during testing was not enough to overcome the frictional resistance keeping the cripple wall in place. The lateral load for Specimen A-7 peaked at 2.24 kips, Specimen A-13 peaked at 1.23 kips, and Specimen A-23 peaked at 4.46 kips which are too low to overcome the frictional resistance between the sill plate and the foundation. In the other tests, sliding of the sill plate began after around 6 kips of lateral load were applied to the wall.

Figures 3.30 through 3.41 show the displacement of the sill plate relative to the foundation versus the lateral load for all cripple walls, with the exception of Specimens A-7, A-13, and A-23, which did not observe significant sill plate movement. It would be expected that cripple walls with larger load capacities would have the most displacement of the sill plate relative to the foundation occur, and this is generally the trend. This was the case with all retrofitted specimens. Specimens A-8 and A-14 experienced the lowest amount of sill movement for retrofitted specimens with horizontal siding exterior finishes. The sill-to-foundation displacement was similar for both, 0.44 in. in the push direction and 0.36 in. in the pull direction for the 2-ft-tall cripple wall and 0.37 in. in the push direction and 0.36 in. in the pull direction for the 6-ft-tall cripple wall. Specimen A-12, the retrofitted T1-11 cripple wall, had a lower peak capacity compared to Specimen A-8 and Specimen A-14. The sill sliding displacement was larger: 0.48 in. in the pull direction for the 2-ft-tall cripple wall and 0.40 in. in the push direction. The increased displacement was due to the reduced amount of anchor bolts resisting the movement of the sill plate. Specimen A-12 contained three anchor bolts while Specimen A-8 contained five anchor bolts, and Specimen A-14 contained seven anchor bolts. This helps to explain why the sill plate displacement for Specimen A-8 was larger than that of Specimen A-14, despite the two walls having similar peak loads. The lowest displacement of the cripple walls with appreciable displacement was for Specimen A-11, the existing T1-11 cripple wall. The peak sill plate displacement for Specimen A-11 was 0.26 in. in the push direction and 0.11 in. in the pull direction. The large discrepancy between the push and pull displacements is partly due to the peak strength exceeding the frictional resistance required for sliding to occur for less drift cycles than the higher strength cripple walls. Another possible factor is due to the alignment of the anchor bolts within the sill plate. If the anchor bolts are located towards one side of the slot than the other, the ability for the sill plate to slide in one direction of the anchor bolt hole is less than in the opposite direction. And lastly, as stated before, Specimen A-11 was tested in two sessions due to issues with the controller for the actuator. The imposed displacement was a push to 2% drift before the test stopped and a 2% drift pull could be imposed. The test resumed after the weekend, which allowed time for the wall to relax and led to an asymmetry in early drift cycles, as seen by the large difference in peak sill displacement values.

By far the largest sill-to-foundation relative displacements occurred in the retrofitted and existing cripple walls with horizontal siding over diagonal sheathing. For the existing cripple wall, Specimen A-9, the peak displacement in the push direction was 1.22 in. and in the pull direction was 2.30 in. For the retrofitted cripple wall, Specimen A-9, the peak displacement in the push direction was 2.77 in. and 2.72 in. in the pull direction. These displacements are so much larger than the rest of the cripple walls because both cripple walls experienced anchor bolts failures during the test. Therefore, once the anchor bolt failures occurred, nearly all of the imposed lateral displacement was culminated in the form of sill plate sliding. This is evident when looking at the lateral force versus global displacement hysteresis compared with the lateral force versus relative displacement hysteresis in Figures 3.8 and 3.9 for Specimen A-9 and Figures 3.11 and 3.12 for

Specimen A-10; the hysteretic loops begin pinching inward in drift cycles shortly after peak load occurs.

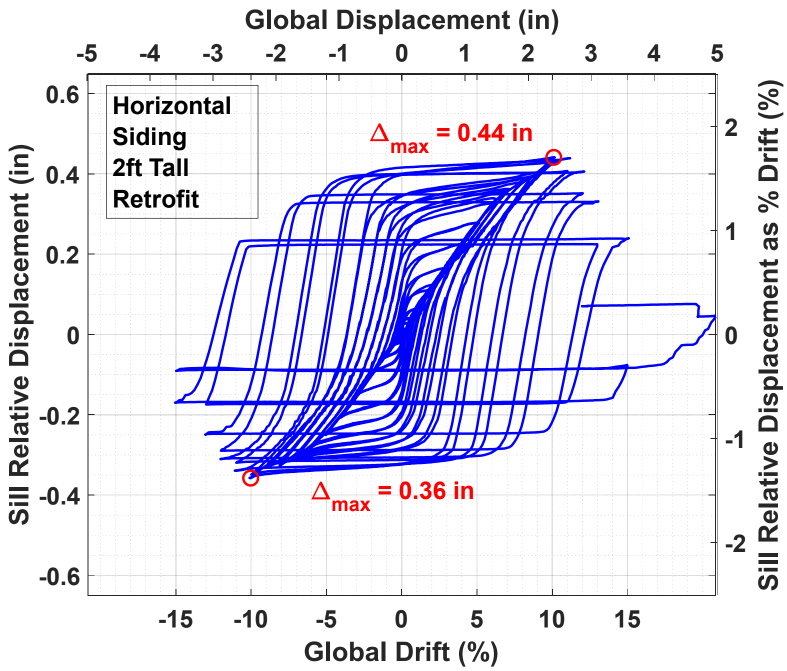


Figure 3.41 Specimen A-8 sill plate to foundation relative displacement versus *global* drift.

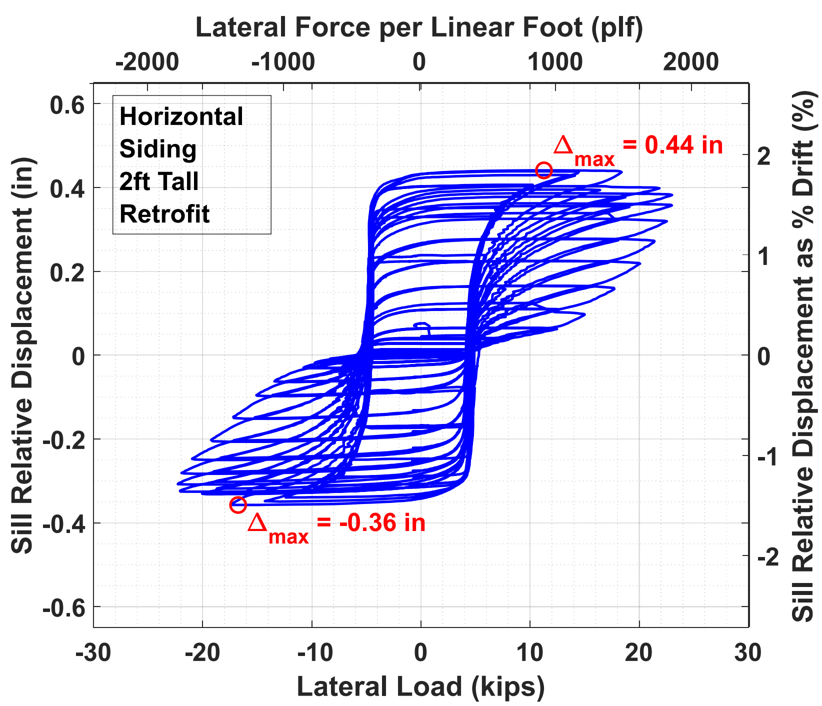


Figure 3.42 Specimen A-8 sill plate to foundation relative displacement versus lateral strength.

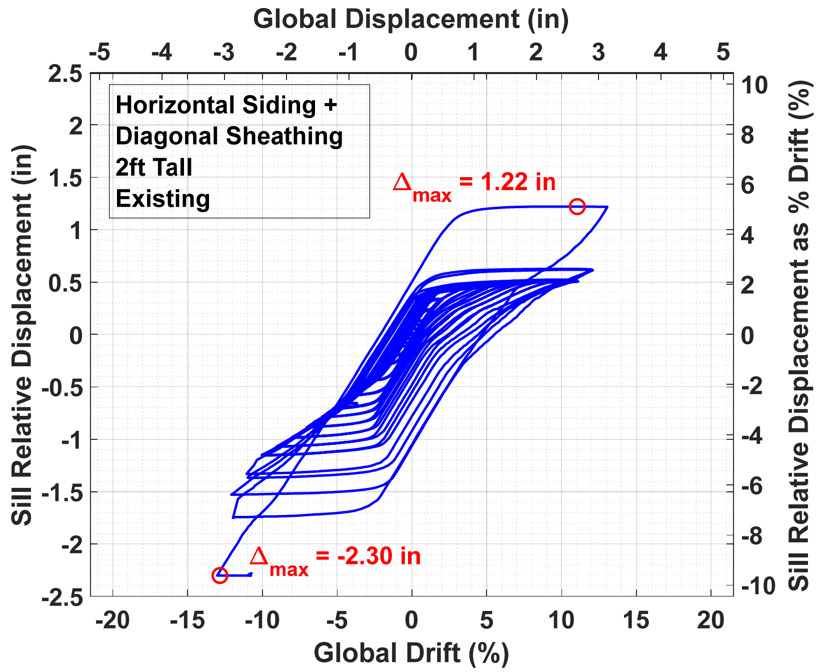


Figure 3.43 Specimen A-9 sill plate to foundation relative displacement versus *global* drift.

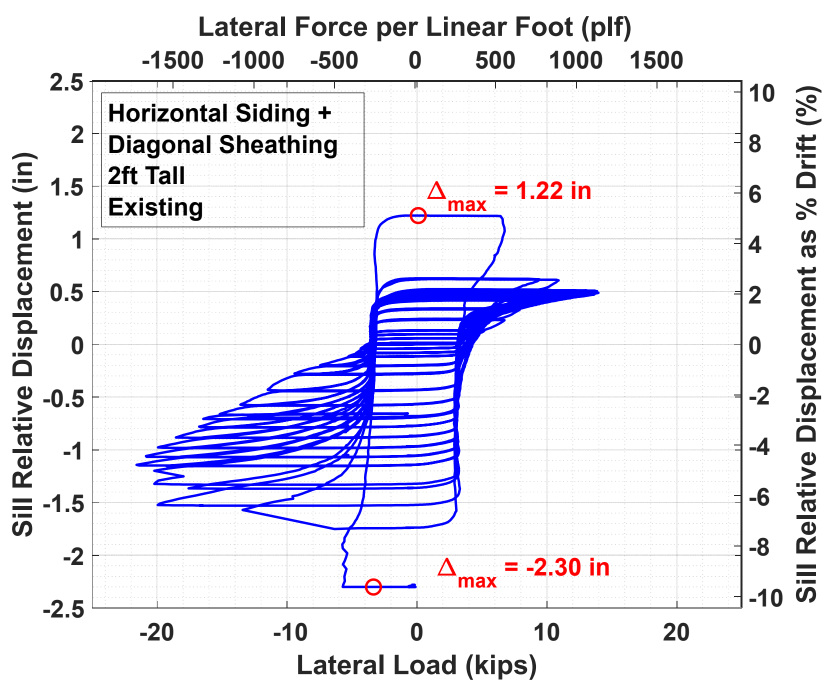


Figure 3.44 Specimen A-9 sill plate to foundation relative displacement versus lateral strength.

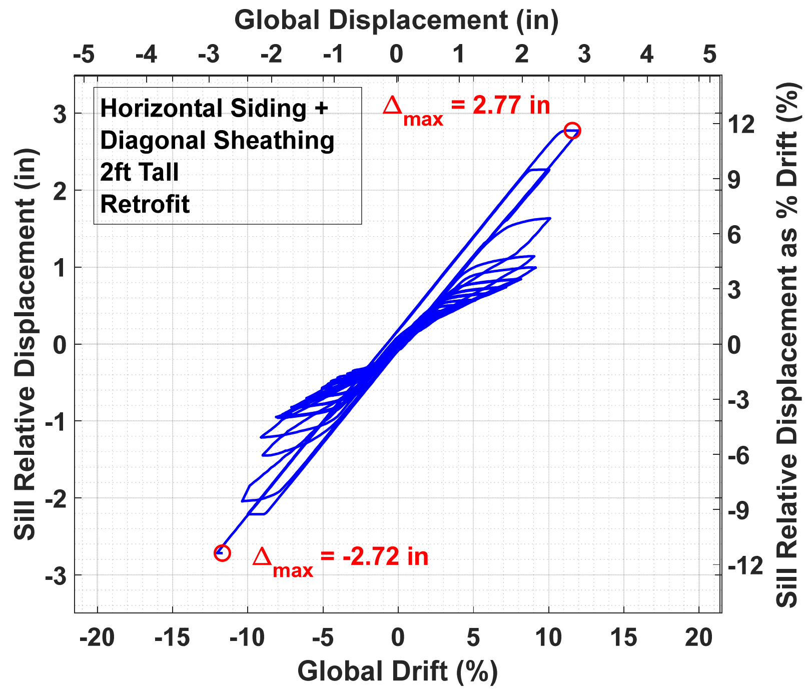


Figure 3.45 Specimen A-10 sill plate to foundation relative displacement versus *global* drift.

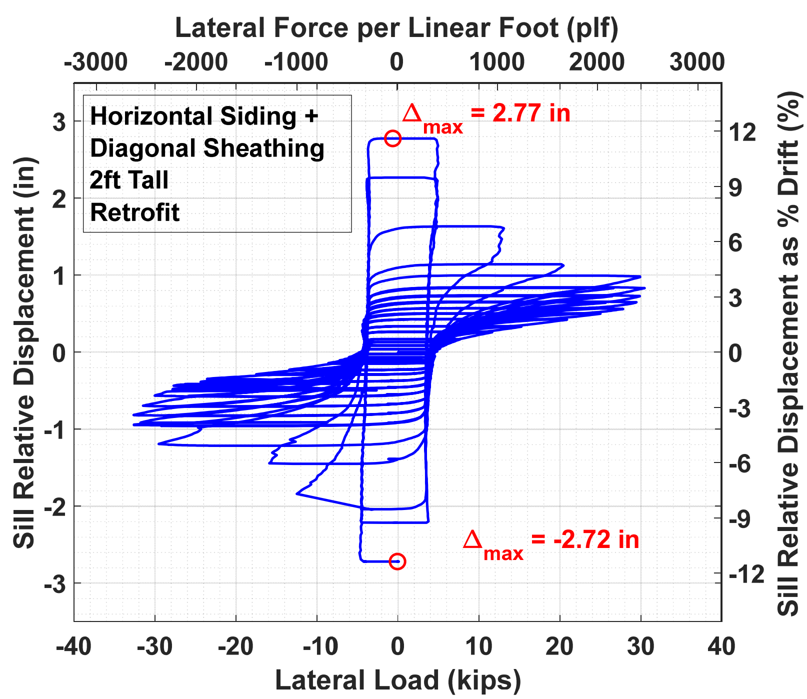


Figure 3.46 Specimen A-10 sill plate to foundation relative displacement versus lateral strength.

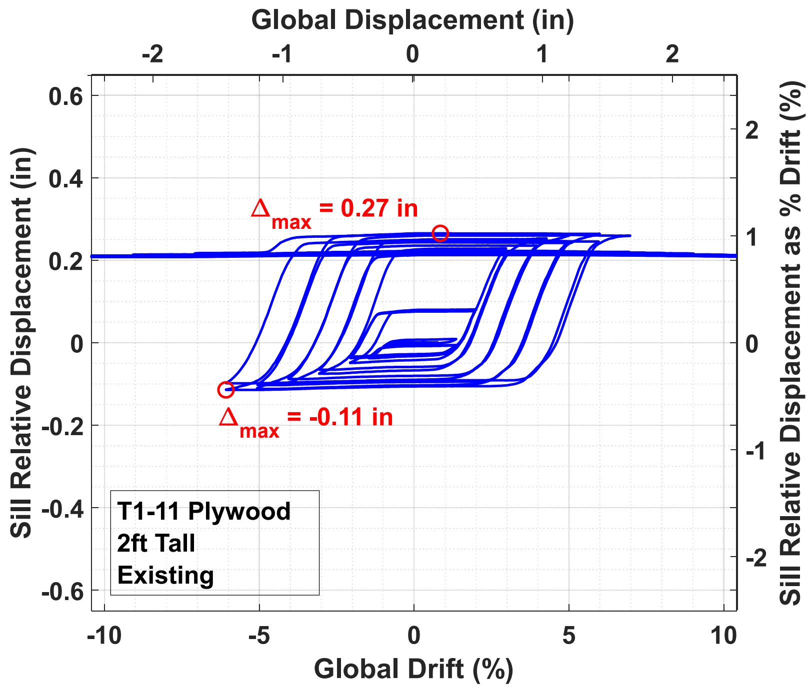


Figure 3.47 Specimen A-11 sill plate to foundation relative displacement versus *global* drift.

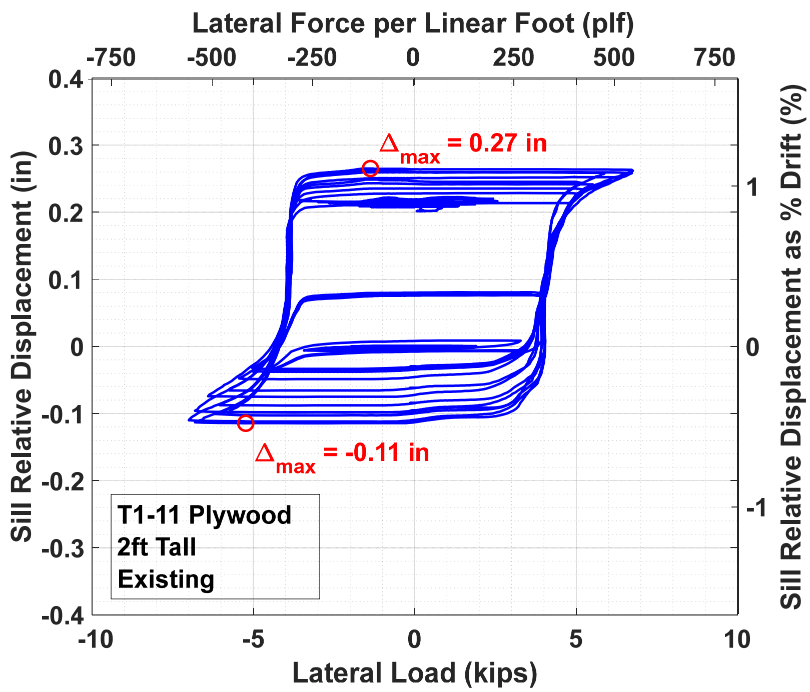


Figure 3.48 Specimen A-11 sill plate to foundation relative displacement versus lateral strength.

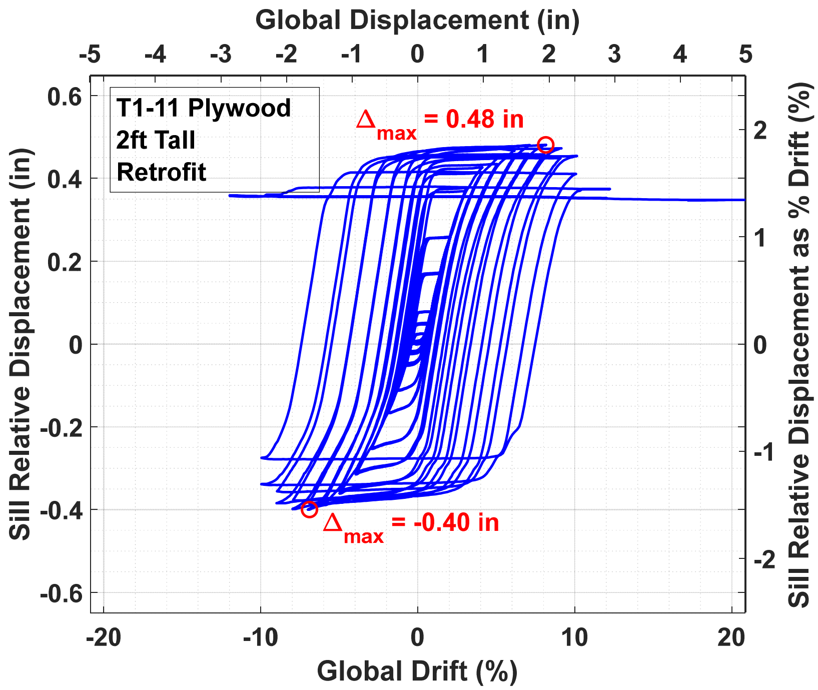


Figure 3.49 Specimen A-12 sill plate to foundation relative displacement versus *global* drift.

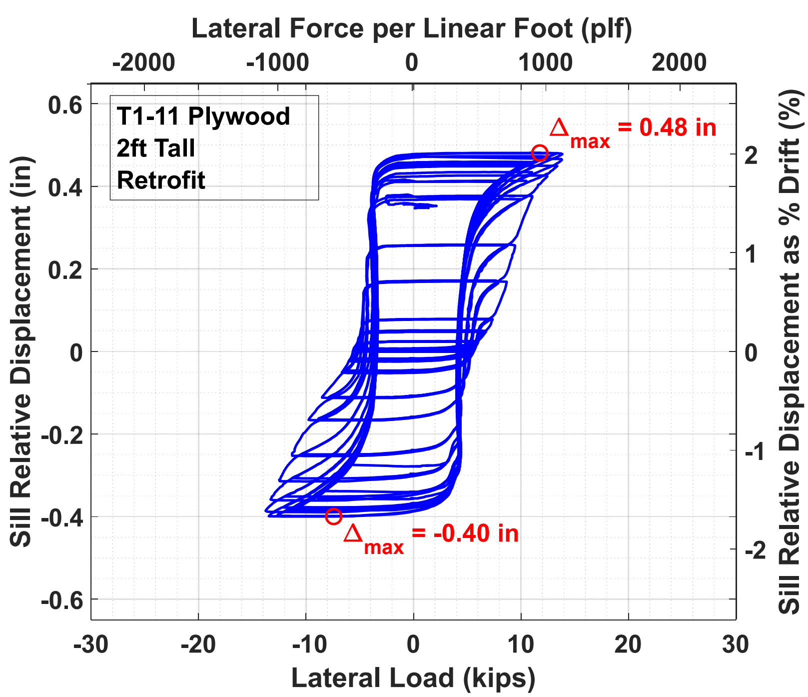


Figure 3.50 Specimen A-12 sill plate to foundation relative displacement versus lateral strength.

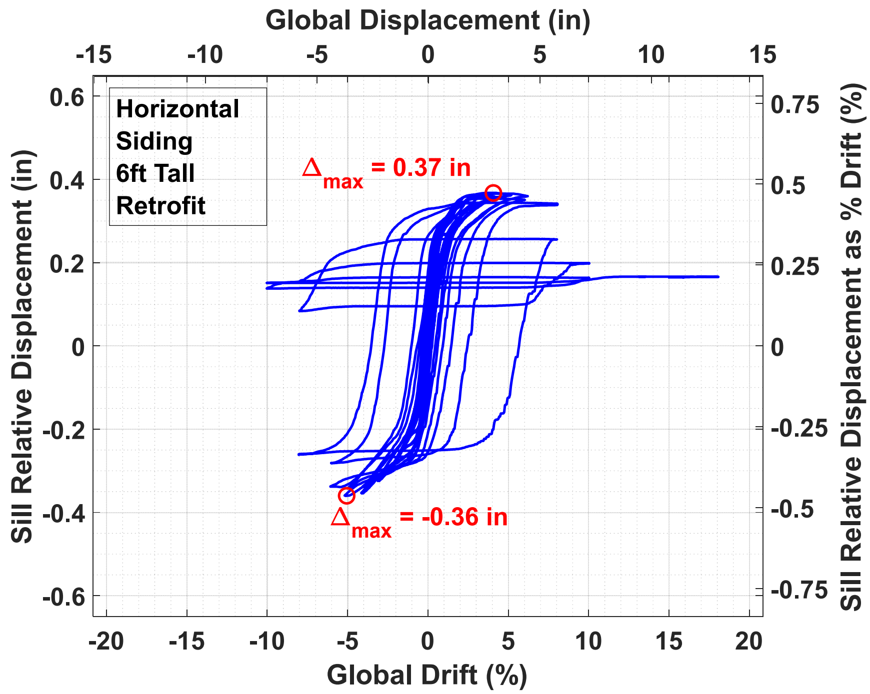


Figure 3.51 **Specimen A-14 sill plate to foundation relative displacement versus *global* drift.**

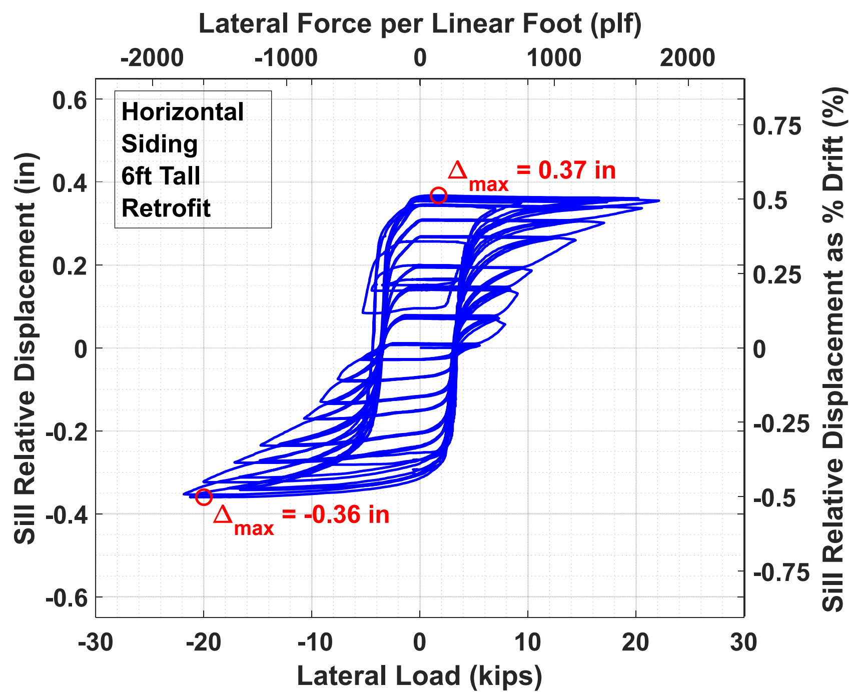


Figure 3.52 **Specimen A-14 sill plate to foundation relative displacement versus lateral strength.**

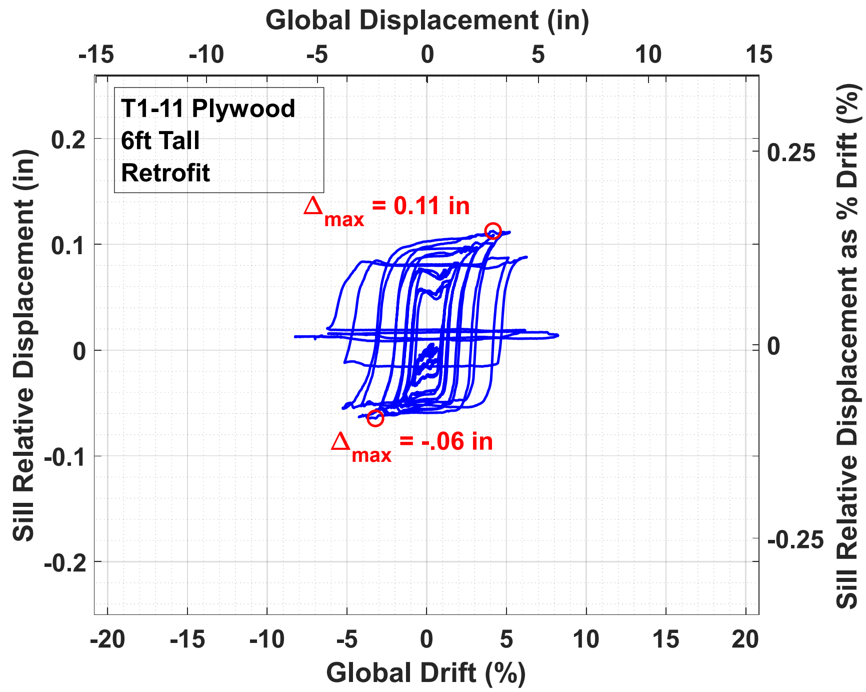


Figure 3.53 Specimen A-24 sill plate to foundation relative displacement versus *global* drift.

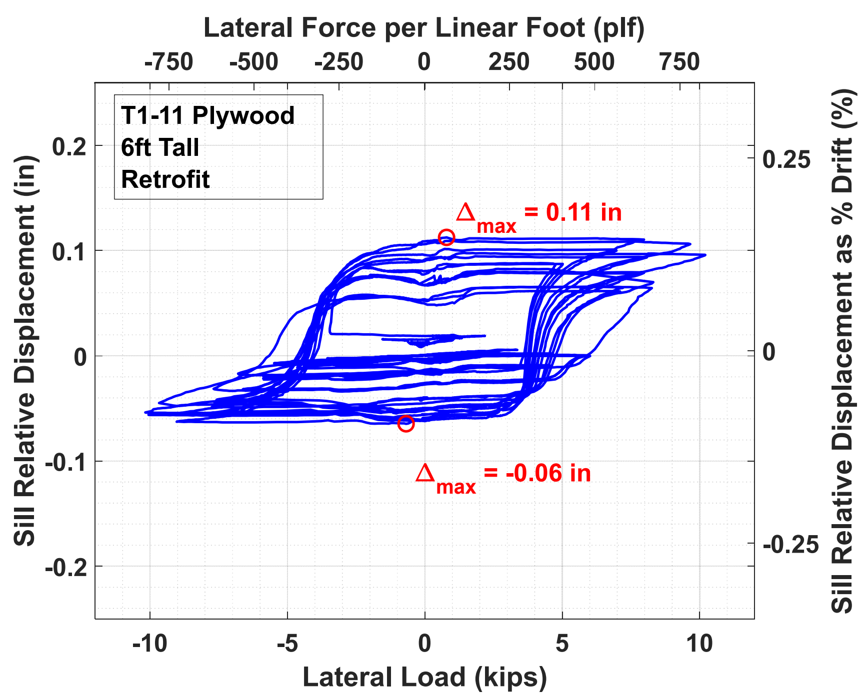


Figure 3.54 Specimen A-24 sill plate to foundation relative displacement versus lateral strength.

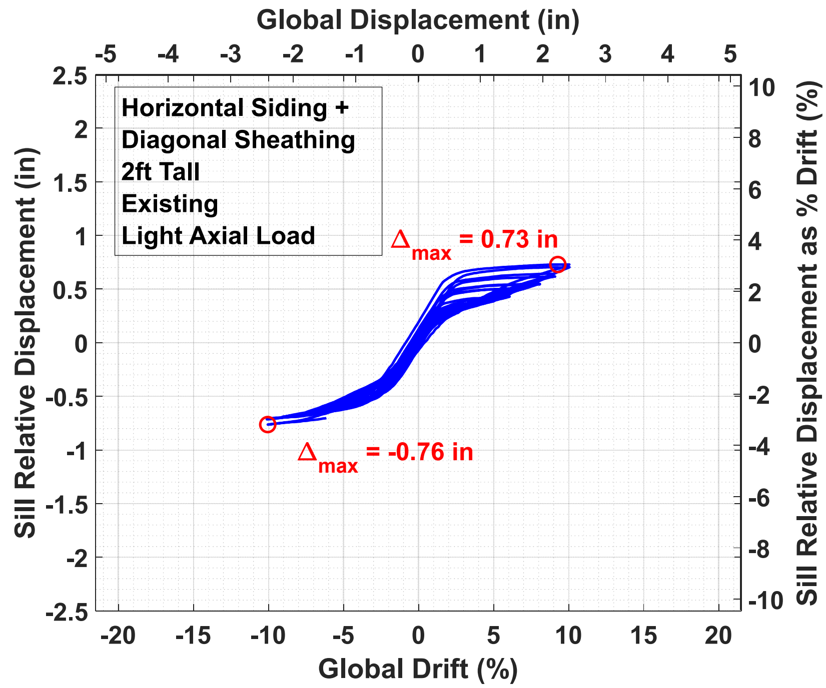


Figure 3.55 Specimen A-28 sill plate to foundation relative displacement versus *global* drift.

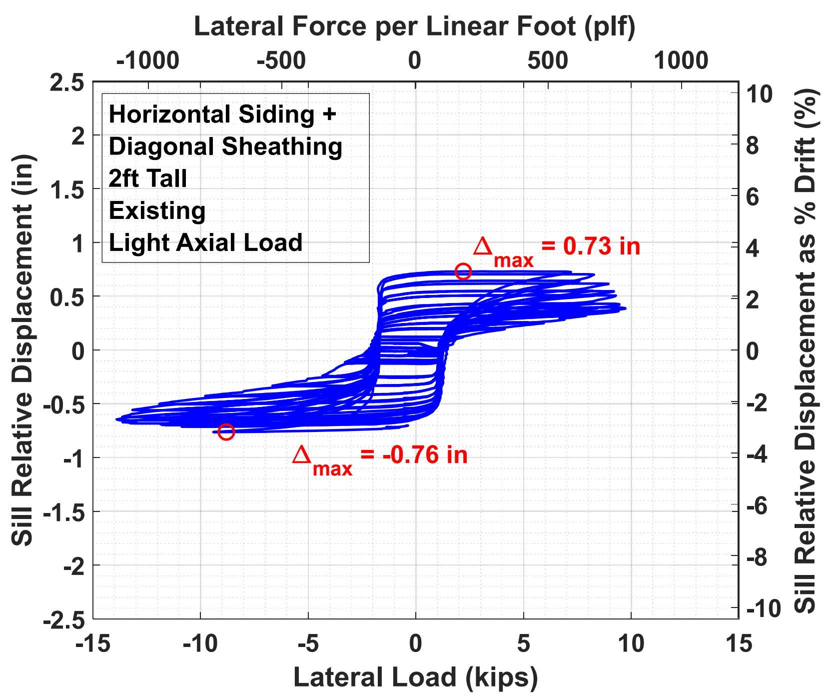


Figure 3.56 Specimen A-28 sill plate to foundation relative displacement versus lateral strength.

3.3.1 Sill Plate to Foundation Friction

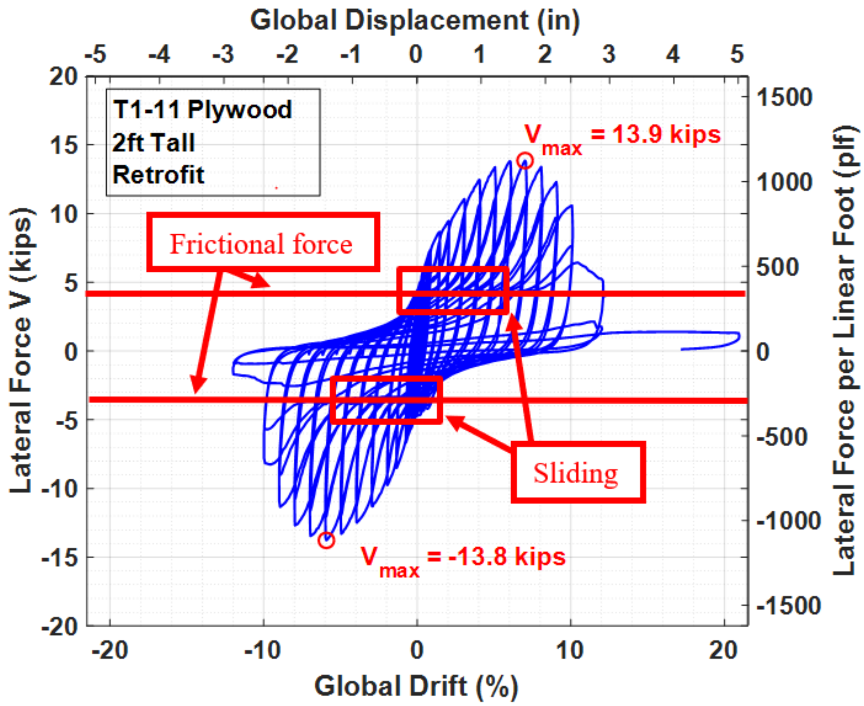
In many of the tests, the global and relative response varied significantly due to the displacement of the sill plate relative to the foundation. To facilitate sliding of the sill plate along the foundation, the load imposed on the cripple wall must overcome the frictional force between the sill plate and foundation. This frictional force is dependent on the weight of the cripple wall, the vertical load on the cripple wall, the roughness of the top of the foundation, and the tensile forces in the anchor bolts fastening the sill plate to the foundation. The foundations were smooth trowel finished; therefore, the foundation surface roughness was not a variable in determining the static coefficient of friction to characterize the resistance between the sill plate and foundation. The static coefficient of friction and hence resistance to sliding would increase with a rougher finish of the foundation. Figure 3.57 gives a visual of the difference between the global and relative response and the associated frictional force preventing the cripple wall from displacing relative to the sill plate. Since the normal force and the lateral force are known, the coefficient of friction between the cripple wall sill plate and the foundation can be estimated by using the following equation:

$$V = \mu N$$

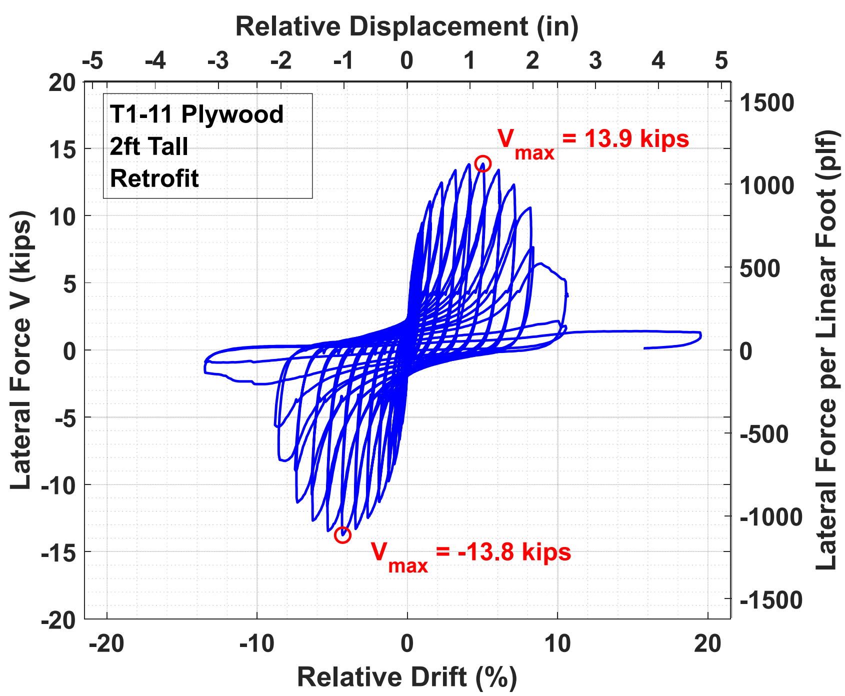
$$where, V = lateral\ load, N = normal\ force, \mu = static\ coefficient\ of\ friction$$

$$\therefore \mu = {}^{V}/_{N}$$

The vertical load for all specimens is 450 plf or 5.5-kips, with the exception of Specimen A-28 that had a vertical load of 150 plf. The weight of the cripple walls varied depending on the construction details and density of the lumber. Higher moisture contents in the lumber equates to a slightly heavier specimen. The amount of material used for all cripple walls that had displacements of the sill plate relative to the foundation was equal, with the exception of the cripple wall with the return walls. The weight was 0.14-0.26 kips for the 2-ft-tall cripple walls and 0.25-0.46 kips for the 6-ft-tall cripple walls. The anchor bolts were tensioned to around 0.2-kip. The amount of lateral load imposed to initiate sliding also varied. These variations can be attributed to the different tension in anchor bolts from test to test, the fluctuations in vertical load, and the anticipated range in static coefficient due to nominal material interface variability. Accounting for the variations in anchor bolt tensions, Table 3.2 shows the static coefficient of frictions between the sill plate and the foundation for all 2-ft-tall specimens that had displacements between the sill plate and foundation. The data is not given for the 6-ft-tall walls or the light vertical load wall as it was not as clear when the cripple walls overcame the frictional resistance between the sill plate and the foundation. The average static coefficient of friction for all specimens that had displacement of the sill plate relative to the foundation is approximated to be 0.64 with a range of values from 0.59 to 0.68. The static coefficient of friction between dry wood and concrete has been measured as 0.62 [Aira et al., 2014].



(a)



(b)

Figure 3.57 **Global and relative responses showing the frictional force between the sill plate and foundation; (a) *global* response; and (b) *relative* response.**

Table 3.2 Static coefficient of friction calculation.

Specimen	Total vertical load (kips)	Total anchor bolt loads (kips)	Frictional force (kips)	μ_{static}
A-8	5.48	1.0	4.42	0.68
A-9	4.82	0.6	3.18	0.59
A-10	5.06	1.2	3.70	0.59
A-11	5.28	0.6	3.90	0.66
A-12	5.14	0.6	3.75	0.65
Average static coefficient of friction = 0.64				

3.4 ANCHOR BOLT LOADS AND FAILURES

In order to measure the tension developed in each anchor bolt, 10-kip donut load cells were placed on top of the square plate washers. For existing cripple walls, three anchor bolts were used, spaced at 64 in. on center. The anchor bolt layout for these cripple walls can be seen in Figure 3.58. Specimen A-12 (the retrofitted 2-ft-tall cripple wall with T1-11 wood structural panel finish) had the same three anchor bolt arrangement as all existing cripple walls. For all other retrofitted cripple walls, additional anchor bolts were added as per the *FEMA P-1100* retrofit guidelines. Specimens A-10, A-14, and A-24 implemented four additional anchor bolts. The location of anchor bolts for Specimen A-10 can be seen in Figure 3.60, for Specimen A-14 can be seen in Figure 3.61, and Specimen A-24 can be seen in Figure 3.62. For Specimen A-8, the locations of anchor bolts can be seen in Figure 3.59. The typical spacing for anchor bolts in the retrofitted cripple was 32 in. on center. For Specimens A-10, A-14, and A-24, two additional anchor bolts were epoxied into place 12 in. inward of the outermost anchor bolts. The embedment depth of the epoxied anchor bolts was 10 in. into the foundation. Specimen A-14 utilized two tie-downs fastened to the innermost end stud, which changed the spacing slightly as seen in Figure 3.45. Specimen A-8 was tested prior to the publication of the *FEMA P-1100* guidelines; therefore, the retrofit details were derived from engineering calculations based on ATC-110 retrofit guidelines [ATC 2014]. All anchor bolts were tensioned to around 200 lbf prior to testing as observed in the field. Initial anchor bolt loads are provided in Table 3.4.

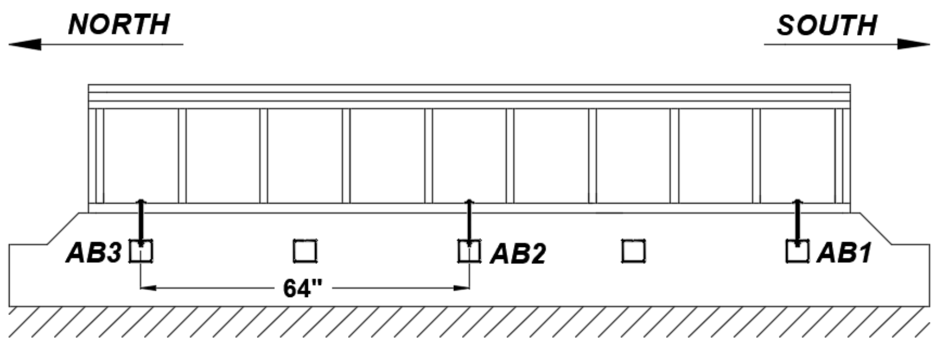


Figure 3.58 Specimen A-12 three anchor bolt layout for existing cripple walls and the retrofitted 2-ft-tall T1-11 cripple wall.

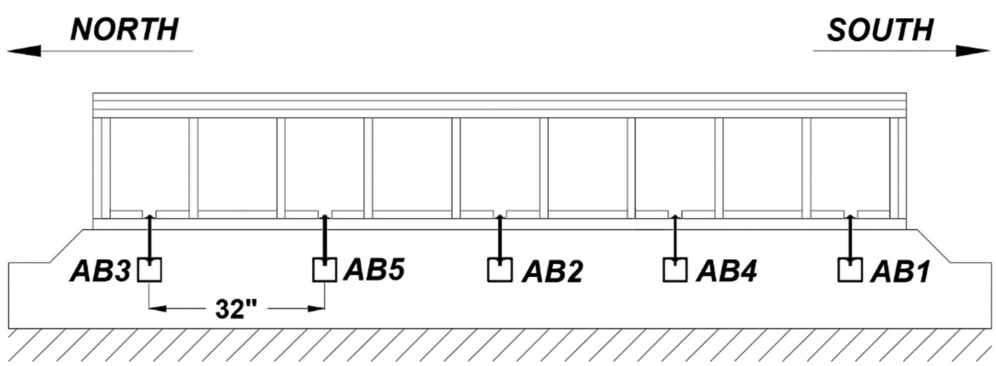


Figure 3.59 Specimen A-8 five anchor bolt layout used for the retrofitted 2-ft-tall cripple wall with horizontal siding.

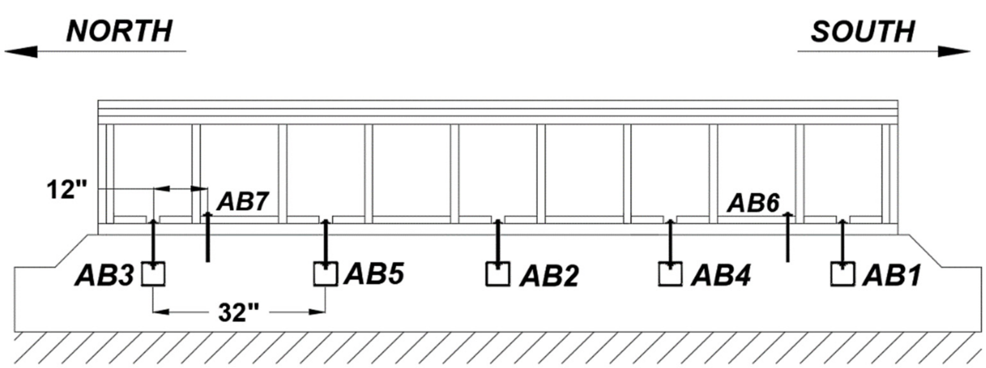


Figure 3.60 Specimen A-10 anchor bolt layout for the retrofitted cripple wall with horizontal siding over diagonal sheathing.

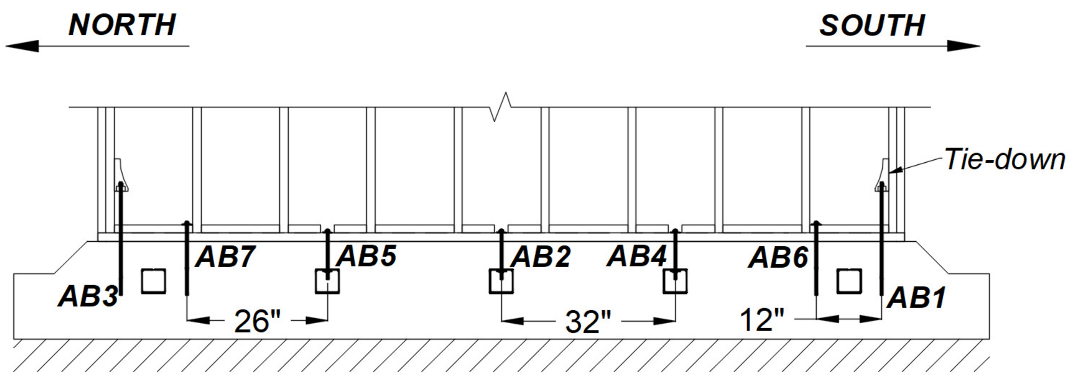


Figure 3.61 Specimen A-14 anchor bolt layout for the retrofitted 6-ft-tall cripple wall with horizontal siding.

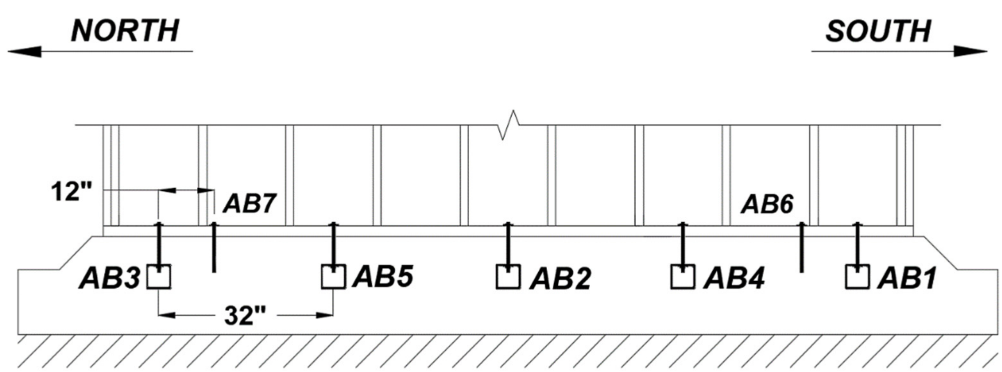


Figure 3.62 Specimen A-24 anchor bolt layout for the retrofitted 6-ft-tall T1-11 cripple wall.

The maximum loads experienced by each anchor bolt during testing is shown in Table 3.3. In general, retrofitted cripple walls experienced the largest anchor bolt loads with the exception of the existing cripples wall with horizontal siding over diagonal sheathing (Specimen A-9 and A-28). The increased anchor bolt loads for these cripple walls were due to the large lateral loads that these cripple walls experienced during testing. The lowest anchor bolt loads were for Specimen A-7 and Specimen A-13, the existing cripple walls with horizontal siding, which again followed the trend of increased anchor bolt loads with increased lateral capacity. The largest anchor bolt loads were experienced by the retrofitted 6-ft-tall cripple wall with horizontal siding. From Table 3.3, it can be seen that AB1 and AB3, the end anchor bolts, peaked at 7.96 kips and 7.35 kips, respectively, while the interior anchor bolts were all less than 2.5 kips. The outermost anchor bolts had the largest amount of tension due to the presence of tie-downs attached to the anchor bolts. The tie-downs resisted the uplifting force of both the sill plate and the end framing, whereas typical anchor bolts only resist the uplift of the sill plate. Due to this, the anchor bolts fastened to tie-downs tended to be subjected to more tension than anchor bolts without tie-downs.

Tables 3.5 and 3.6 provide the anchor bolt loads experienced at peak loading in the push and pull directions, respectively. Note that the cripple walls were pushed in South and pulled north.

When loaded in the push direction, the anchor bolts on the north end of the walls saw increases in load as they resisted the uplift and sliding of the cripple wall, and vice versa for when loaded in the pull direction. All cripple walls exhibited this trend with the exception of the existing 2-ft-tall and 6-ft-tall cripple walls with horizontal siding finishes. These specimens had the lowest capacity of any specimens tested and experienced a reduction of the anchor bolt tension as the nuts on the bolts loosened throughout the test. Finally, Tables 3.7 and 3.8 show differences in anchor bolt loads in the push and pull directions from the start of the test, respectively.

Table 3.3 Anchor bolt maximum loads (in kips) for all cripple walls.

Specimen	South			Center	North		
	AB3	AB7	AB5	AB2	AB4	AB6	AB1
A-7	0.19			0.44			0.24
A-8	2.18		0.86	0.87	1.16		2.07
A-9	5.42			3.39			6.05
A-10	4.04	2.20	2.52	3.82	2.53	1.25	4.30
A-11	1.12			0.63			1.01
A-12	2.75			1.32			2.35
A-13	0.22			0.17			0.18
A-14	7.35	2.43	1.90	1.66	1.31	2.04	7.99
A-23	2.06			0.23			1.31
A-24	2.85	1.01	0.69	1.36	1.51	1.13	2.45
A-28	2.49			4.68			3.08

Table 3.4 Initial anchor bolt tension (in kips) at start of test.

Specimen	South			Center	North		
	AB3	AB7	AB5	AB2	AB4	AB6	AB1
A-7	0.17			0.18			0.16
A-8	0.21		0.19	0.20	0.15		0.22
A-9	0.20			0.18			0.19
A-10	0.19	0.21	0.18	0.16	0.17	0.10	0.20
A-11	0.21			0.19			0.20
A-12	0.19			0.21			0.20
A-13	0.20			0.17			0.18
A-14	0.05	0.19	0.18	0.17	0.17	0.19	0.09
A-23	0.18			0.17			0.19
A-24	0.13	0.17	0.18	0.18	0.16	0.27	0.18
A-28	0.19			0.22			0.16

Table 3.5 Anchor bolt load (in kips) at peak load in the *push* loading direction.

Specimen	South			Center	North		
	AB3	AB7	AB5	AB2	AB4	AB6	AB1
A-7	0.07			0.06			0.14
A-8	1.98		0.19	0.71	1.19		1.81
A-9	2.42			1.83			1.74
A-10	3.51	0.88	2.47	3.82	1.93	2.94	2.29
A-11	0.90			0.62			0.01
A-12	2.75			1.25			0.27
A-13	0.15			0.16			0.14
A-14	7.35	2.40	1.90	1.66	0.74	1.44	0.01
A-23	2.06			0.14			0.00
A-24	2.75	0.75	0.68	0.34	0.03	0.01	0.02
A-28	1.96			1.60			1.13

Table 3.6 Anchor bolt load (in kips) at peak load in the *pull* loading direction.

Specimen	South			Center	North		
	AB3	AB7	AB5	AB2	AB4	AB6	AB1
A-7	0.00			0.12			0.18
A-8	0.00		0.86	0.38	1.06		0.59
A-9	2.71			3.35			4.76
A-10	2.89	0.99	2.52	3.18	2.53	3.36	3.39
A-11	0.16			0.02			0.95
A-12	0.01			0.86			2.35
A-13	0.14			0.16			0.12
A-14	0.00	0.35	1.44	1.20	1.15	2.04	7.99
A-23	0.04			0.22			1.25
A-24	0.01	0.03	0.29	1.04	1.43	0.82	2.44
A-28	1.12			4.25			3.00

Table 3.7 **Difference in anchor bolt loads (in kips) at peak *push* load to initial anchor bolt loads.**

Specimen	South			Center	North		
	AB3	AB7	AB5	AB2	AB4	AB6	AB1
A-7	-0.10			-0.12			-0.02
A-8	1.77		0.50	0.51	1.04		1.59
A-9	2.22			1.65			1.55
A-10	3.32	0.67	2.29	3.66	1.76	2.84	2.09
A-11	0.69			0.43			-0.19
A-12	2.56			1.03			0.07
A-13	-0.05			-0.01			-0.04
A-14	7.30	2.21	1.72	1.49	0.56	1.25	-0.08
A-23	1.88			-0.03			-0.19
A-24	2.62	0.58	0.50	0.16	-0.13	-0.26	-0.16
A-28	1.77			1.38			0.97

Table 3.8 **Difference in anchor bolt loads (in kips) at peak *pull* load to initial anchor bolt loads.**

Specimen	South			Center	North		
	AB3	AB7	AB5	AB2	AB4	AB6	AB1
A-7	0.01			-0.07			-0.16
A-8	-0.21		0.67	0.18	0.90		0.37
A-9	2.51			3.17			4.57
A-10	2.70	0.78	2.34	3.02	2.36	3.26	3.19
A-11	-0.05			-0.17			0.75
A-12	-0.18			0.65			2.15
A-13	-0.06			-0.01			-0.06
A-14	-0.05	0.16	1.26	1.03	0.97	1.86	7.90
A-23	-0.14			0.05			1.06
A-24	-0.12	-0.14	0.11	0.56	1.27	0.55	2.26
A-28	0.93			4.02			2.84

Two cripple walls failed due to fracture of their anchor bolts. Both specimens were finished with horizontal siding over diagonal sheathing. All cripple walls with this finish had large increases in anchor bolts loads as the test progressed, regardless of the direction of loading. The strong combination of materials increased the relative displacements between the sill plate and the foundation, thus increasing the anchor bolt loads as they would bear on both the sill plate and the foundation while the wall continued to displace. In addition, as the specimens moved laterally in the pull direction (north), the diagonal sheathing moved upward and produced additional uplift forces in the sill plate due to the sheathing boards being fastened to the sill plate. The result produced significant increases in the anchor bolt loads during pull loading compared with during push loading.

Figure 3.63 shows the anchor bolt load versus displacement for existing cripple wall Specimen A-9, and Figure 3.66 shows the anchor bolt load versus displacement for retrofitted cripple wall Specimen A-10. In Figures 3.64 and 3.67, the locations of the anchor bolt fractures on the hysteretic response can be seen for the existing and retrofitted cripple walls, respectively. Figures 3.65 and 3.68 provide a reference to the locations of the anchor bolts for the existing and retrofitted cripple walls, respectively. The fractures are a result of both shear and flexural forces acting on the anchor bolts. Once one anchor bolt fracture occurred, the shear and flexural forces would increase on the other anchor bolts, and as the cripple walls continued to gain strength, the anchor bolts could not resist these forces, eventually causing all anchor bolts to fracture. Although the load on Specimen A-14's anchor bolts was the highest of all the specimens, there were no fractures. These anchor bolts were connected to the tie-downs; therefore, they were more in tension from the cripple wall uplifting than flexure and shear from the wall displacing.

Figures 3.69 and 3.70 show images of the anchor bolt failures for both specimens. During testing of many cripple walls, cross-grain cracks developed in the sill plates. In the case of Specimen A-28, cross-grain cracks propagated across the entire span of the sill plate. This specimen also had a horizontal siding over diagonal sheathing exterior finish, but the applied vertical load was a third of the other cripple walls with this finish (150 plf versus 450 plf). The reduction in vertical load reduced the uplift resistance on the wall. Combined with the diagonal sheathing uplifting during testing, large stresses were developed in the sill plate. Since the sheathing material was only nailed to the exterior of the cripple wall, the sill was subjected to cross-grain bending, and resulted in a full span crack of the sill through all anchor bolt slots. Because there was no resistance from the sill plate, the cripple wall strength dramatically decreased even through there was not significant damage to the finish materials or framing. Photographs of the damage to the sill plate are provided in Figure 3.71. All anchor bolt load versus global drift hysteresis can be found in Appendix C.1.

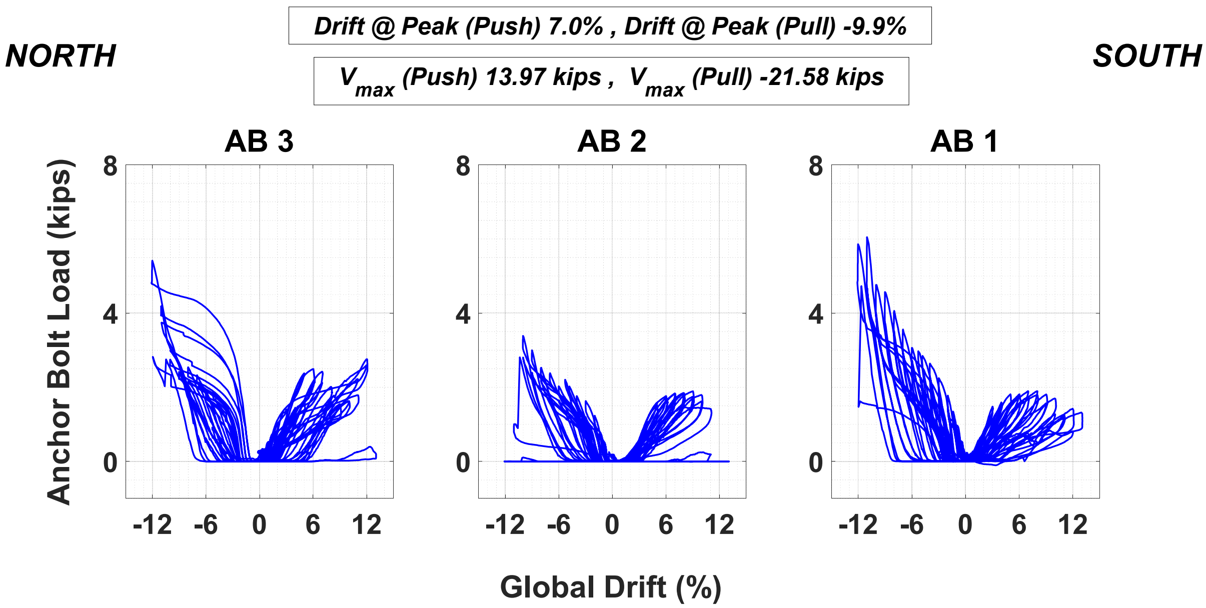


Figure 3.63 Specimen A-9 anchor bolt load versus global drift for the existing cripple wall with horizontal siding over diagonal sheathing.

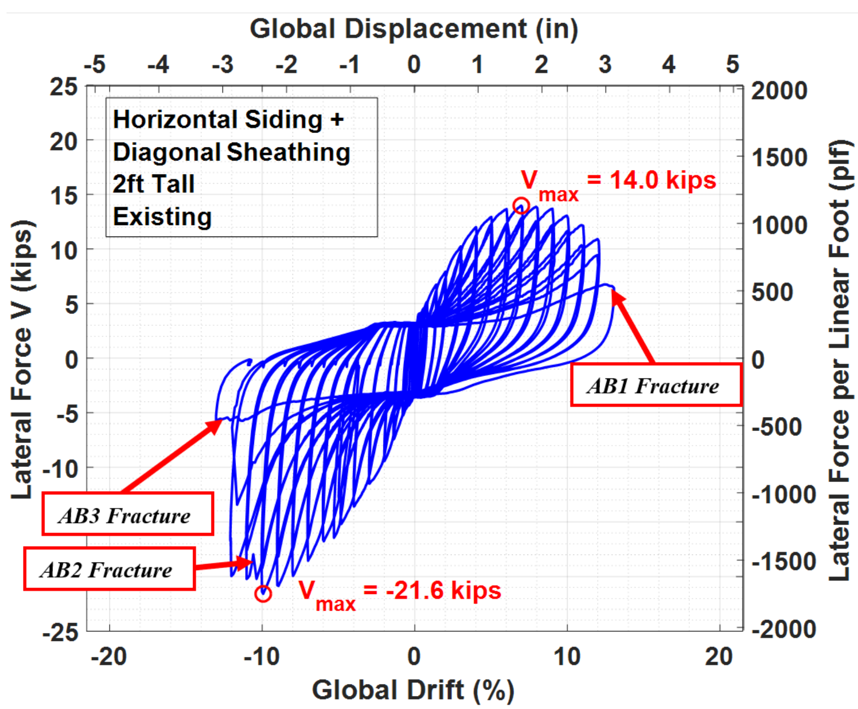


Figure 3.64 Specimen A-9 location of anchor bolt fractures on lateral force versus global lateral drift and displacement hysteresis.

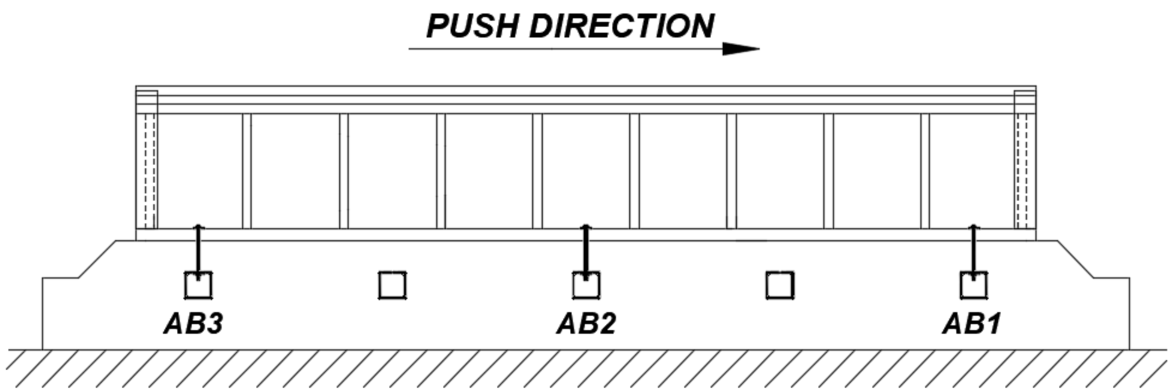


Figure 3.65 Specimen A-9 anchor bolt locations.

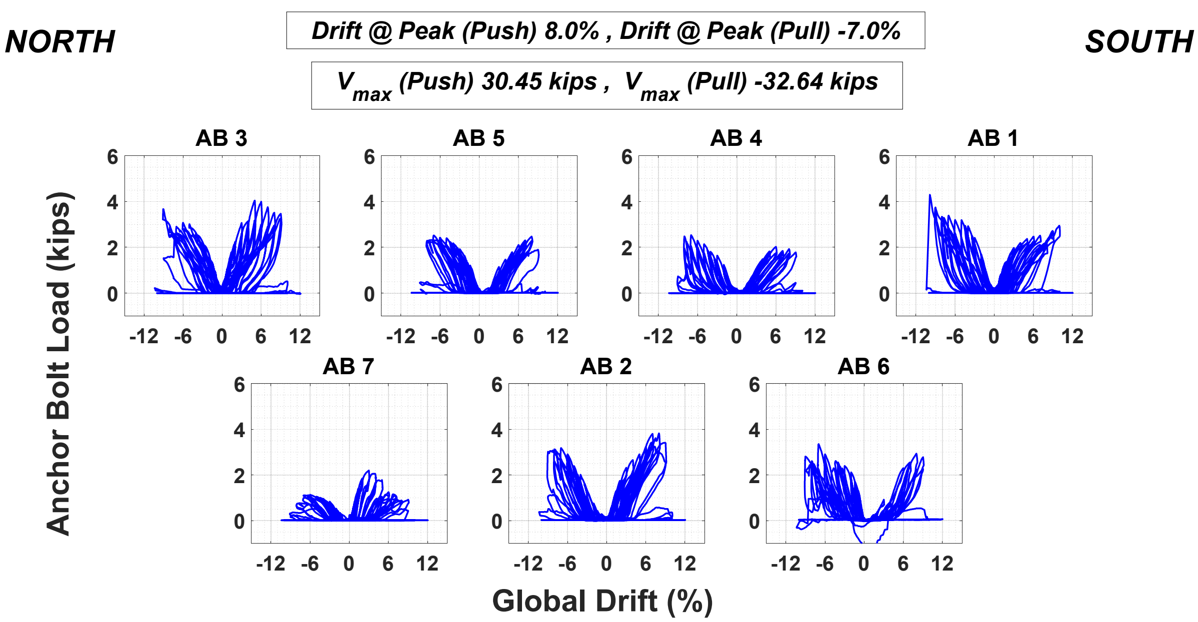


Figure 3.66 Specimen A-10 anchor bolt load versus global drift for the retrofitted cripple wall with horizontal siding over diagonal sheathing.

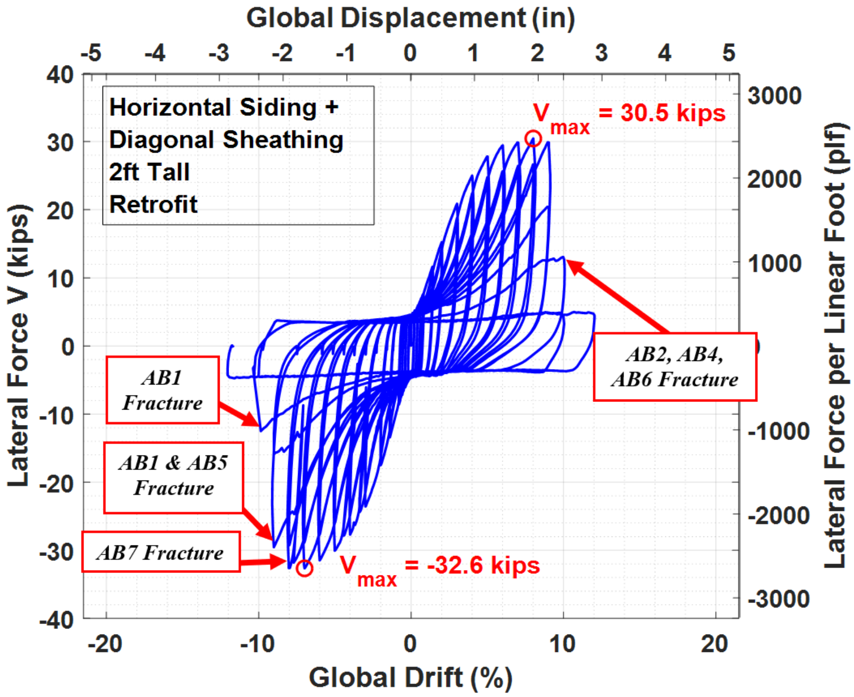


Figure 3.67 Specimen A-10 instances of anchor bolt fractures overlaid with the lateral force versus global lateral drift and displacement hysteresis.

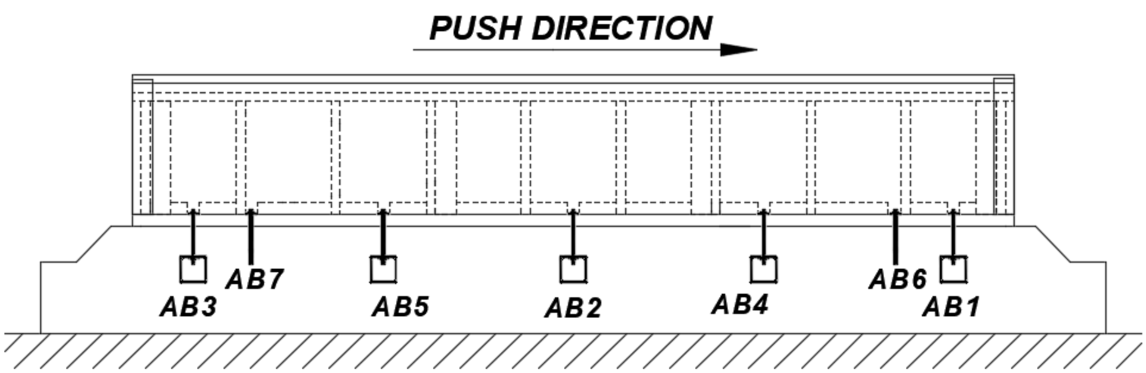


Figure 3.68 Specimen A-10 anchor bolt locations.

Figure 3.69 Specimen A-9 fractured anchor bolt (left) and damage to the sill plate post-testing (right).

Figure 3.70 Specimen A-10 fractured anchor bolt (left) and damage to the sill plate post-testing (right).

Figure 3.71 Specimen A-28 cross-grain sill plate split.

3.5 DIAGONAL MEASUREMENTS

Two sets of potentiometers were used to take measurements of the displacement across the diagonals of the cripple wall. One pair of potentiometers measured the distortion across the entire cripple wall, while the other pair measured the distortion of the middle third of the cripple wall. The purpose of these measurements was to determine the amount of shear distortion within the cripple wall. These measurements were used to determine whether the applied lateral displacement could be resolved using the diagonal and end uplift measurements. Figure 3.72 shows the linear potentiometers used to calculate the resolved lateral displacement of the cripple wall. Figure 3.73 shows the how the resolved lateral displacements from diagonal and uplift measurements were derived.

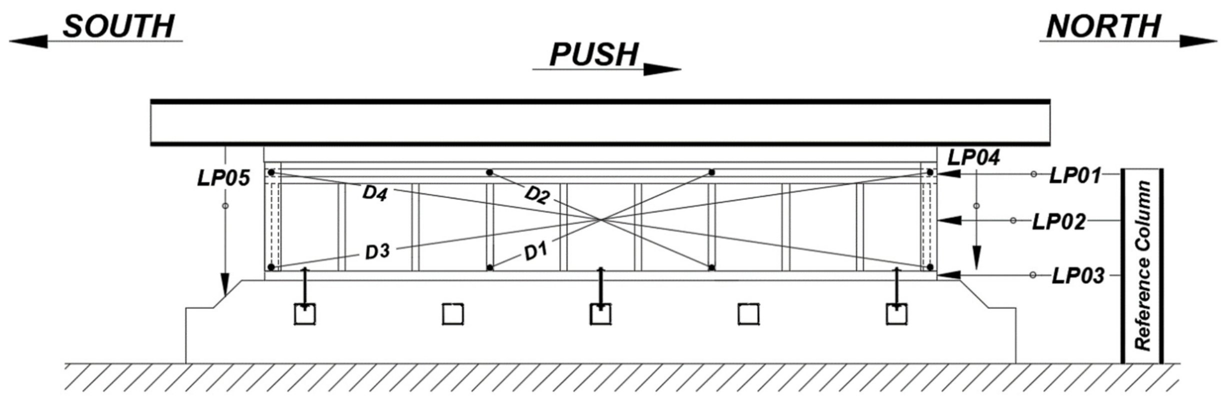


Figure 3.72 **Diagonal, end uplift, and lateral displacement potentiometer schematic.**

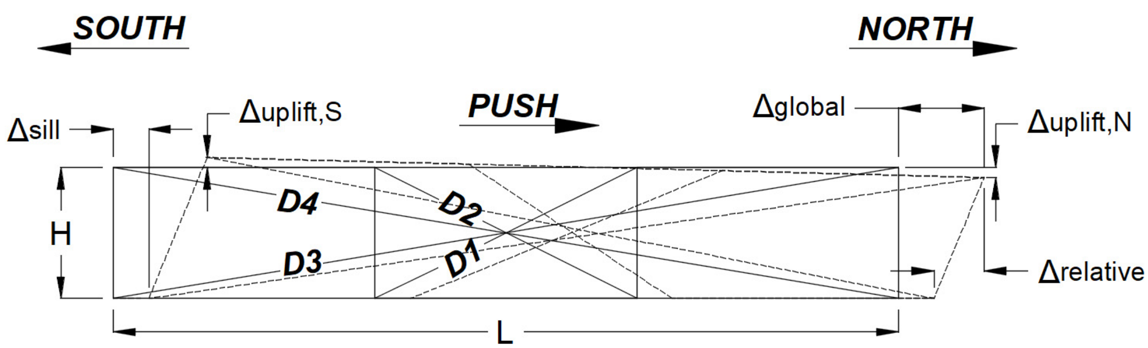


Figure 3.73 **Deformed cripple wall with measurements used for resolving lateral displacement from diagonal and uplift measurements.**

Figures 3.74 through 3.77 show the relative drift versus the relative drift resolved from the diagonal and uplift measurements for Specimen A-13 and A-14. Figures 3.74 and 3.76 overlay the resolved lateral drifts from the inside diagonals on the left and the resolved lateral drifts from the outside diagonals on the right for Specimen A-13 and A-14, respectively. Figures 3.75 and 3.77, overlay the resolved lateral drifts from the diagonals running from the bottom north end of the wall to the top south end of the wall on the left, and the resolved lateral drifts running from the top north end of the wall to the bottom south end of the wall on the left for Specimen A-13 and A-14, respectively. All these figures include a green line, indicating the measured relative drift plotted against itself, as a reference. These cripple walls were chosen because they only differ in their retrofit condition. Specimen A-13, the unretrofitted cripple wall, had resolved relative drift values within 1.5% drift of the measured relative drift. The difference tended to be less than that for most of the test, only diverging at later drift amplitudes. On average between push and pull loading, the relative drift resolved from the inside diagonals differed by 0.8% relative drift and the relative drift resolved from the outside diagonals differed by 0.7% relative drift. These values increased for the differences between the measured relative drift and resolved relative measurements from the inside diagonals for the retrofitted cripple wall with an average difference of 5.0%, while the difference for the measured relative drift and resolved relative drift of the outside diagonals was 2.1% relative drift. This shows that the addition of the plywood panels for the retrofit reduced the shear distortion through the interior of the cripple wall where the panels are attached. Overall, the pattern was the same for all existing and retrofitted cripple walls. All of the resolved relative drift figures can be found in Appendix C.2.

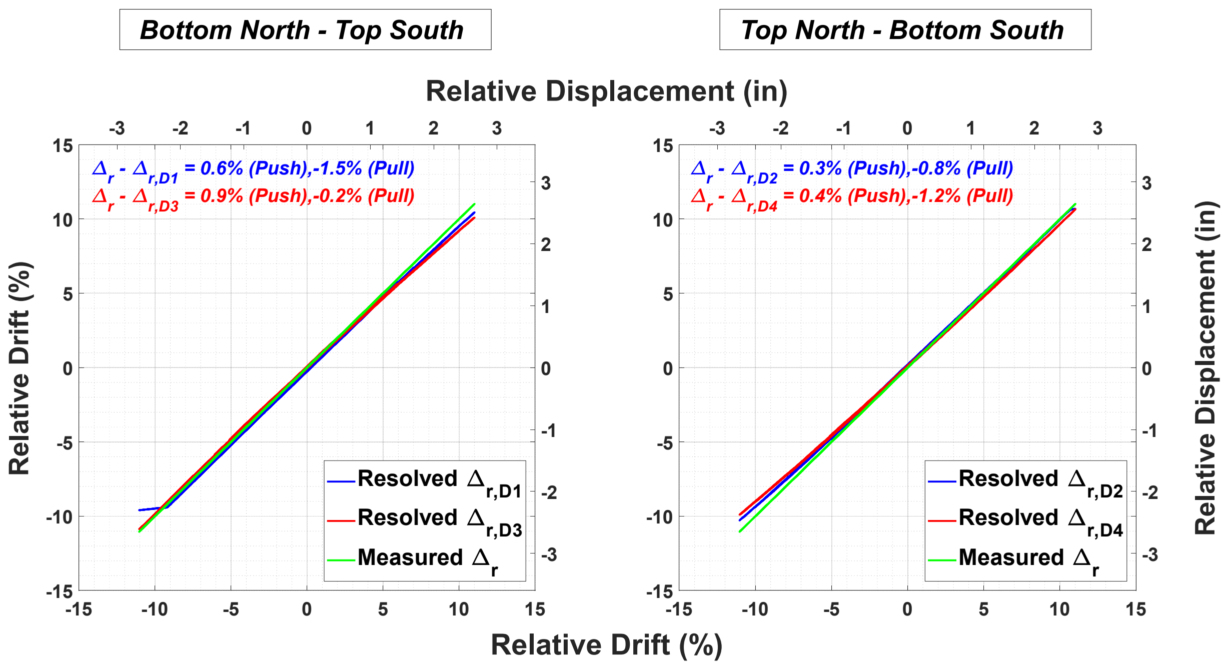


Figure 3.74 Specimen A-13: resolved relative drift from diagonal measurements in one direction versus measured relative drift for the existing 6-ft-tall cripple wall with horizontal siding.

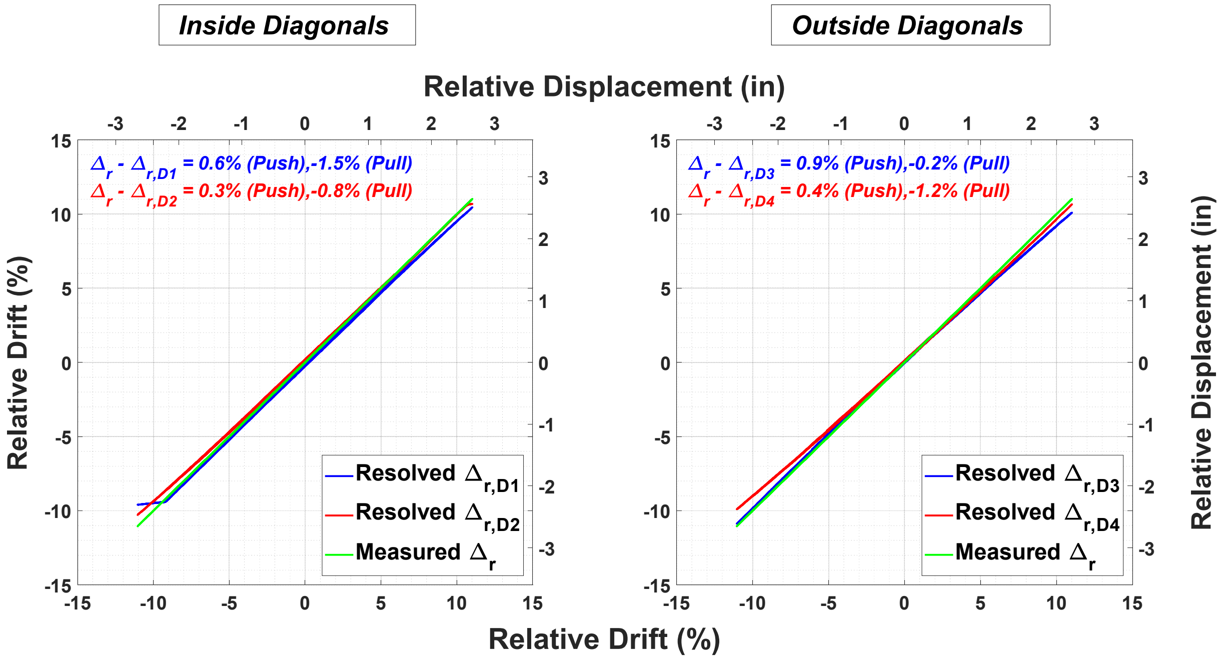


Figure 3.75 Specimen A-13: resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for the existing 6-ft-tall cripple wall with horizontal siding.

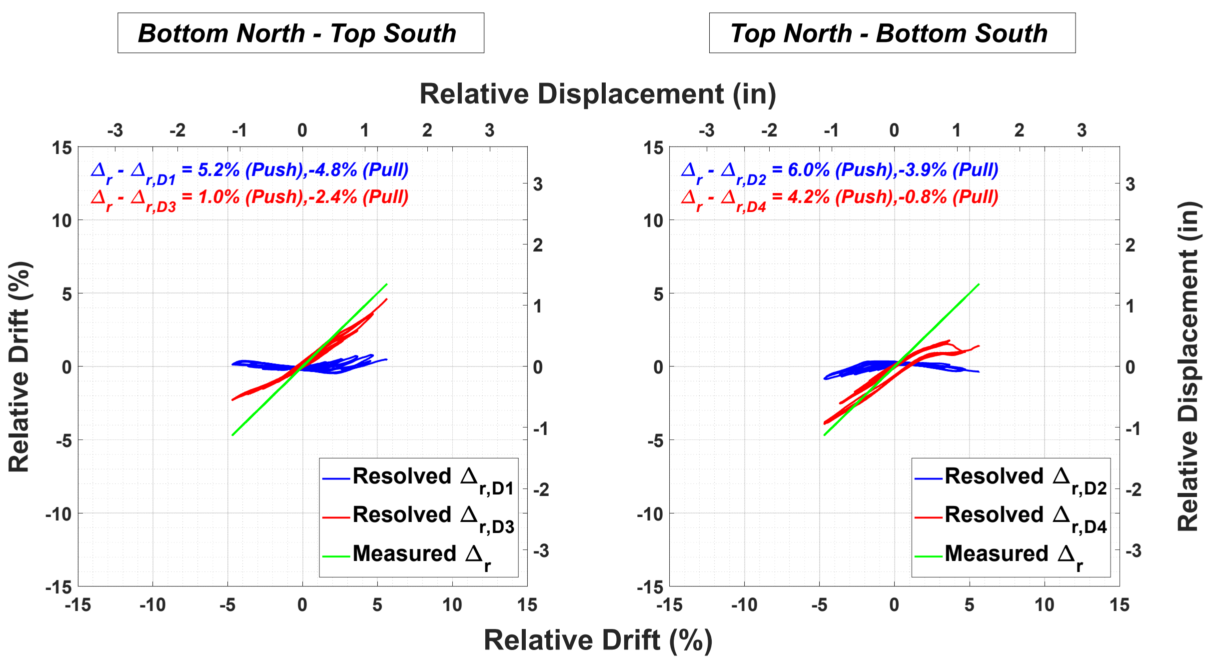


Figure 3.76 Specimen A-14: resolved relative drift from diagonal measurements in one direction versus measured relative drift for retrofitted 6-ft-tall cripple wall with horizontal siding.

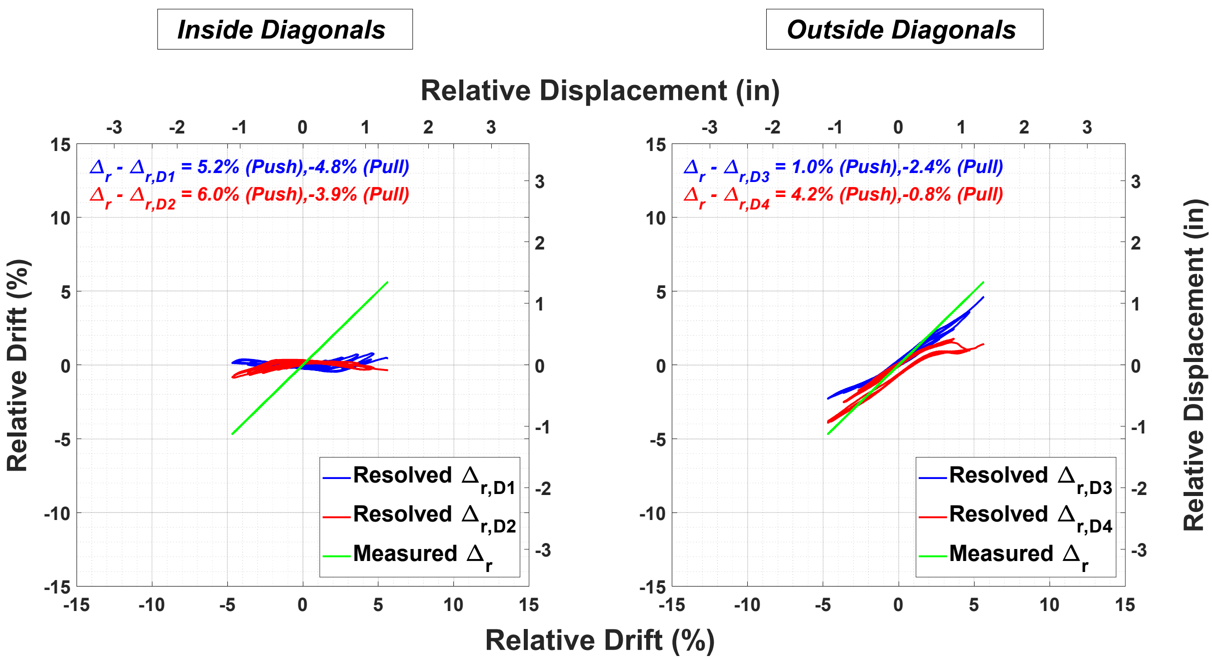


Figure 3.77 **Specimen A-14 resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for the retrofitted 6-ft-tall cripple wall with horizontal siding.**

3.6 UPLIFT MEASUREMENTS

Two linear potentiometers were used to measure the uplift at both ends of the cripple wall. These potentiometers were attached to the foundation and the steel load transfer beam. The calculations for determining the uplift of the cripple walls is shown in the previous section as the uplift measurements were factored into calculating the resolved relative displacement from the diagonal measurements. Table 3.9 summarizes the maximum uplift measurement at each end of the wall for all specimens. All cripple walls experienced uplift at the ends when being displaced with the exception of the existing cripple walls finished with horizontal sheathing.

Figures 3.78 and 3.79 show the end uplift versus relative drift response for the existing and retrofitted 6-ft-tall specimens with horizontal siding finishes, respectively. At each drift ratio level, the existing specimen's height was reduced due to the deformation of the wall and insufficient forces to overcome the uplift resistance. With the added retrofit, deformation still occurred, but the increased strength of the cripple wall caused uplift to occur as it is greater than the uplift resistance provided by the vertical load and weight of the specimen. The addition of the retrofit increased the end of wall uplift for all specimens. For the existing cripple walls with horizontal siding over diagonal sheathing, there was a drastic difference in the uplift at the south end versus the north end of the walls, which is attributed to the orientation of the diagonal sheathing boards. When loaded in the pull direction, the sheathing board moved both laterally and vertically upward, whereas the sheathing boards moved laterally and vertically downward in the opposite direction of loading. This caused the uplift to be strongly governed by the direction of loading. The largest end uplifts were measured for the horizontal siding over diagonal sheathing finished cripple wall with a light vertical load. The peak south end uplift was 550% greater (0.26 in. to 1.69 in.) and the north end uplift was 275% greater (0.68 in.–2.55 in.) when compared to the cripple wall's counterpart with a heavy vertical load. A comparison of all end uplift versus relative drift responses can be seen in Appendix C.3.

Table 3.9 End uplift measurements.

Specimen no.	South-end uplift (in.)	North-end uplift (in.)
A-7	0	0.03
A-8	0.20	0.46
A-9	0.26	0.68
A-10	0.79	0.77
A-11	0.04	0.17
A-12	0.12	0.10
A-13	0	0
A-14	0.80	1.30
A-23	0.23	0.10
A-24	0.72	0.86
A-28	1.69	2.55

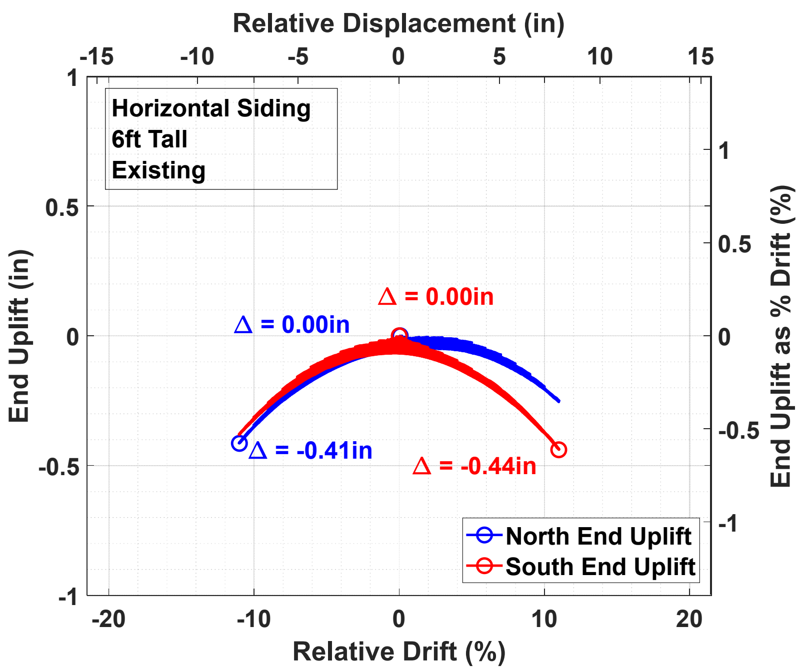


Figure 3.78 Specimen A-13 end uplift versus *relative* drift for the existing 6-ft-tall cripple wall with horizontal siding.

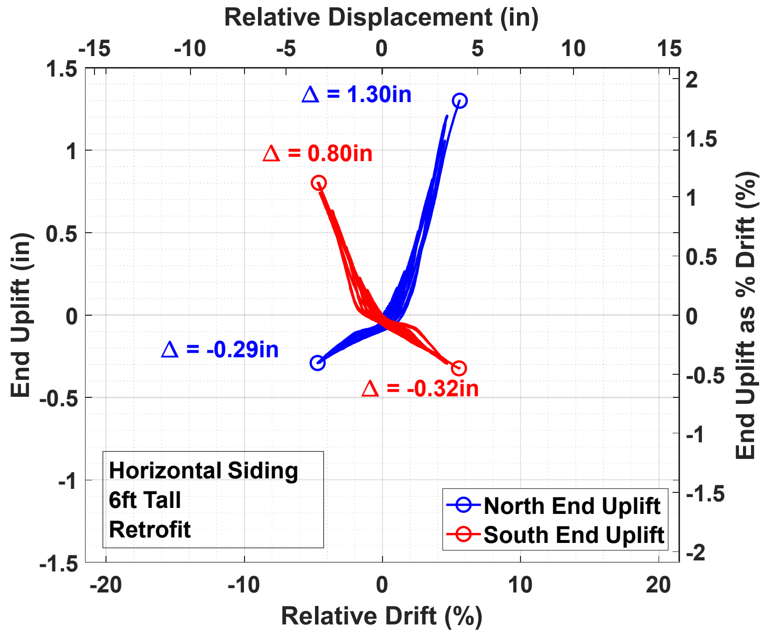


Figure 3.79 **Specimen A-14 end uplift versus *relative* drift for the retrofitted 6-ft-tall cripple wall with horizontal siding.**

3.7 COMPARISON OF RETROFITTED CRIPPLE WALLS

One of the major goals of this Project is to understand and quantify the effectiveness of retrofitting cripple walls. Five pairs of cripple walls with retrofits and dry exterior finishes were tested. Retrofitting cripple walls works to address vulnerabilities in the connection of the cripple wall to the framing above, the cripple wall framing, and the foundation sill plate anchorage to the foundation. This involves adding connectors to improve the connection from the cripple wall to the framing above, adding wood structural panels to the interior framing of the cripple wall to strengthen the cripple wall, and installing additional anchor bolts to increase the sliding resistance of the dwelling. In addition, for taller cripple walls (typically 4 ft or greater), tie-downs, more commonly called hold-downs in California, are installed to increase the uplift capacity of the dwelling. These tie-downs are intended to preserve the attachment of the cripple wall with the foundation and reduce the tendency of walls with higher aspect ratios from rotating and tearing the plywood sheathing loose from the sill plate. For the purposes of this testing program, connectors used to improve connection from the cripple wall to the framing above were not implemented as only cripple wall components were tested.

The design guidelines for retrofitting in this project come from the *FEMA P-1100* prescriptive design provisions, which were published in December 2018 [FEMA 2018]. The *FEMA P-1100* prescriptive design provisions were the basis of the retrofit design used in this Project with the exception of the retrofitted specimen in Phase 1 and Specimen A-8, which were both tested before *FEMA P-1100* was published. For these two cripple walls, the same retrofitting strategy was used as those that followed the *FEMA P-1100* guidelines, but there are differences in the fastening of the wood structural panels and the amount of anchor bolts added. The retrofit details for these cripple walls were derived from engineering calculations that followed ATC-110 [ATC 2014]. Details on the *FEMA P-1100* retrofit design can be found in Section 2.4.

Overall, the retrofit dramatically increased the lateral strength of the cripple wall, and in some cases, it also increased the drift capacity of the cripple wall. Recall that cripple wall retrofit pairs are identical to each other besides the addition of the retrofit. Figures 3.80 through 3.89 show overlays of the global and relative lateral displacement versus lateral load hysteretic response for specimen pairs. Figure 3.90 shows the lateral strength per linear foot of the five retrofitted cripple walls. Figure 3.91 shows the relative drift (total drift minus the displacement between the sill plate and foundation) at strength for the five retrofitted cripple walls. Lastly, Figure 3.92 shows the percent increase in strength of the retrofitted specimens compared to their existing counterpart. Note that all boundary conditions and vertical load were the same for the specimens.

The most significant improvement to performance due to the retrofit was for both the 2-ft-tall and the 6-ft-tall cripple walls with horizontal siding exterior finish. For the 2-ft-tall specimen, the peak strength per linear foot increased from 186 plf to 1867 plf in the push direction and from 162 plf to 1794 plf in the pull direction, a 925% and 1007% increase in capacity, respectively. For the 6-ft-tall specimen, the peak strength per linear foot increased from 89 plf to 1786 plf in the push direction and from 97 plf to 1754 plf in the pull direction, a 1907% and 1708% increase in capacity, respectively. The strengths of Specimen A-8 and Specimen A-14 are much closer than those of Specimen A-7 and Specimen A-13 due to the relatively small contribution the framing and horizontal siding have in resisting lateral loads. The drift capacity did not increase, however, due to the flexibility of the existing horizontal siding cripple walls. The average push and pull secant stiffnesses associated with the relative drift at 80% strength increased 929% for the 2-ft-tall cripple walls and 3215% for the 6-ft-tall cripple walls.

Both specimens with horizontal siding over diagonal sheathing and T1-11 plywood panels showed significant increases in peak strength. For the horizontal siding over diagonal sheathing specimens, the peak strength per linear foot increased from 1156 plf to 2473 plf in the push direction and from 1713 plf to 2626 plf in the pull direction, a 114% and 53% increase in capacity, respectively. In addition, the secant stiffnesses associated with the relative drift at 80% strength increased 95% in the push direction and 161% in the pull direction. It would be expected that given additional anchorage, the strengths and drift capacity of both walls would continue to increase, especially with Specimen A-10, which did not exhibit the asymmetric response of Specimen A-9. It should be noted that large cracks formed in sill plates of both cripple walls; therefore, even with additional anchorage, the sill plates might not have been able to handle such large loads.

As stated in Section 3.5, the retrofit design for a T1-11 wood structural panel finished cripple wall was somewhat unique from the *FEMA P-1100* prestandard retrofit design. Instead of adding plywood to the interior framing, additional nails were fastened to the T1-11 panels on the exterior because when sufficiently fastened, T1-11 panels provide adequate resistance to seismic demands as they are plywood panels. No additional anchor bolts were added to the T1-11 retrofitted 2-ft-tall cripple wall for the following reasons: accessing the interior to install anchor bolts would increase the cost of the retrofit, and houses built in the 1960s or later often have more anchor bolts and anchor bolts in better condition compared to older houses. This strategy was only implemented for the 2-ft-tall cripple wall.

The *FEMA P-1100* guidelines were used for the 6-ft-tall cripple wall retrofit, which called for four additional anchor bolts to be added. For the T1-11 2-ft-tall wood structural panel specimens, the strength per linear foot increased from 1156 plf to 2473 plf in the push direction and from 1713 plf to 2626 plf in the pull direction, a 114% and 53% increase in capacity, respectively. The secant stiffness associated with the relative drift at 80% strength increased 108% in the push direction and 63% in the pull direction. The retrofit to the T1-11 cripple wall also exhibited the largest increase in drift capacity. This is primarily due to the additional nailing on the previously un-nailed edge of the panels. For the 6-ft-tall specimens, the strength increased from 375 plf to 851 plf in the push direction and from 382 plf to 848 plf in the pull direction, a 127% and 122% increase in capacity, respectively. The secant stiffness associated with the relative drift at 80% strength increased 36% in the push direction and 71% in the pull direction. The T1-11 retrofit had the lowest strength of any of the retrofitted specimens, which was due to the wider edge nail spacing (4 in. on center compared with 3 in. on center) as well as the lack of an interior and exterior finish (only contained an exterior finish).

Once strength occurred, the retrofitted cripple walls experienced fairly consistent incremental drops in load at subsequent displacement cycles as the nails fastening the plywood panels to the interior framing either pulled out of the framing or tore through the plywood. One exception to this trend was for Specimen A-10, which failed due to fractures in the anchor bolts, leading to a dramatic loss of lateral load capacity in subsequent displacement cycles. The other exception was for Specimen A-14, which reached peak strength at 4% drift, dropped 8.6% in the push and 2.7% in the pull direction from 4% to 5% drift, then dropped 13.9% in the push and 39.9% in the pull direction from 5% to 6% drift, and then dropped 69.1% in the push direction and 40.6% in the pull direction from 6% to 8% drift. These dramatic drops in load were due to the flexible nature of a taller cripple wall, which causes significant shear and flexural forces on the plywood panels instead of the mostly shear forces that the plywood panels on shorter cripple walls experience. Overall, loss of capacity of the cripple walls occurred when multiple edges of the plywood panels had detached from the framing. A more in-depth look at the damage characteristics of the retrofitted cripple walls is provided in Chapter 4.

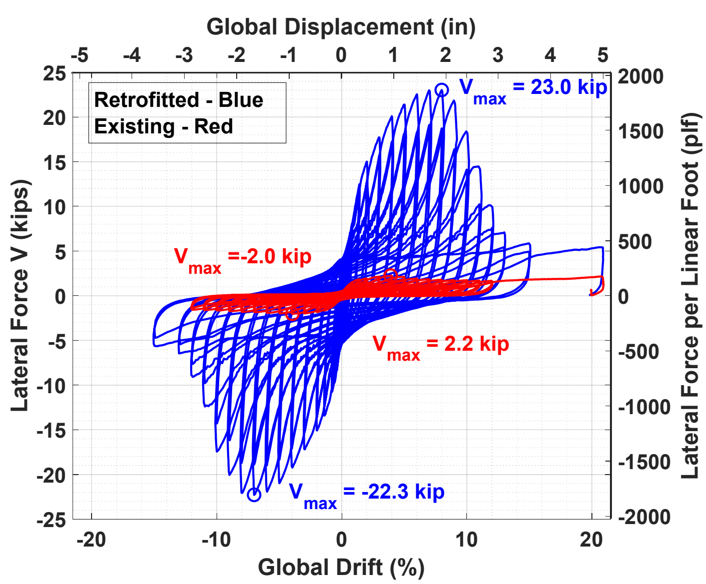


Figure 3.80 **Specimens A-7 and A-8: comparison of *global* drift versus lateral load hysteretic response for retrofitted and existing 2-ft-tall cripple walls with horizontal siding.**

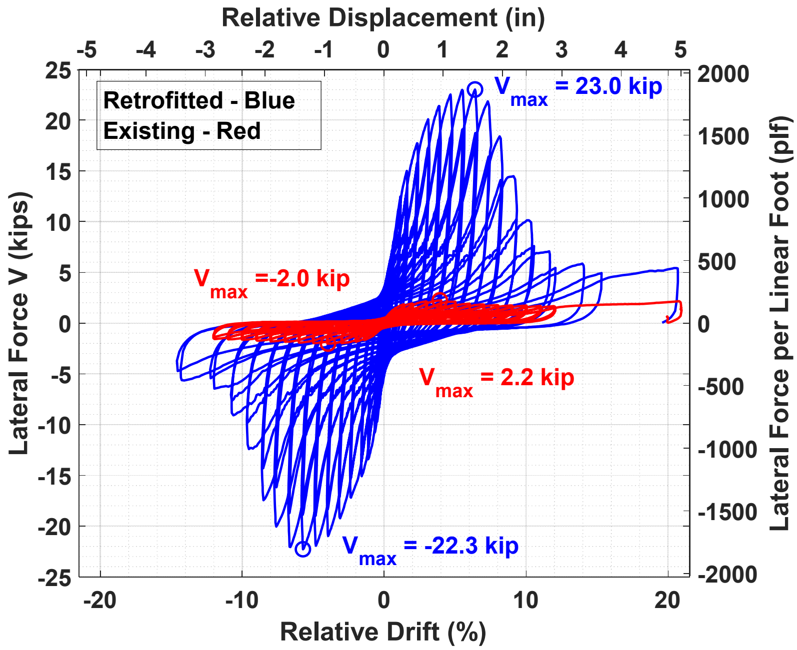


Figure 3.81 Specimens A-7 and A-8: comparison of *relative* drift versus lateral load hysteretic response for retrofitted and existing 2-ft-tall cripple walls with horizontal siding.

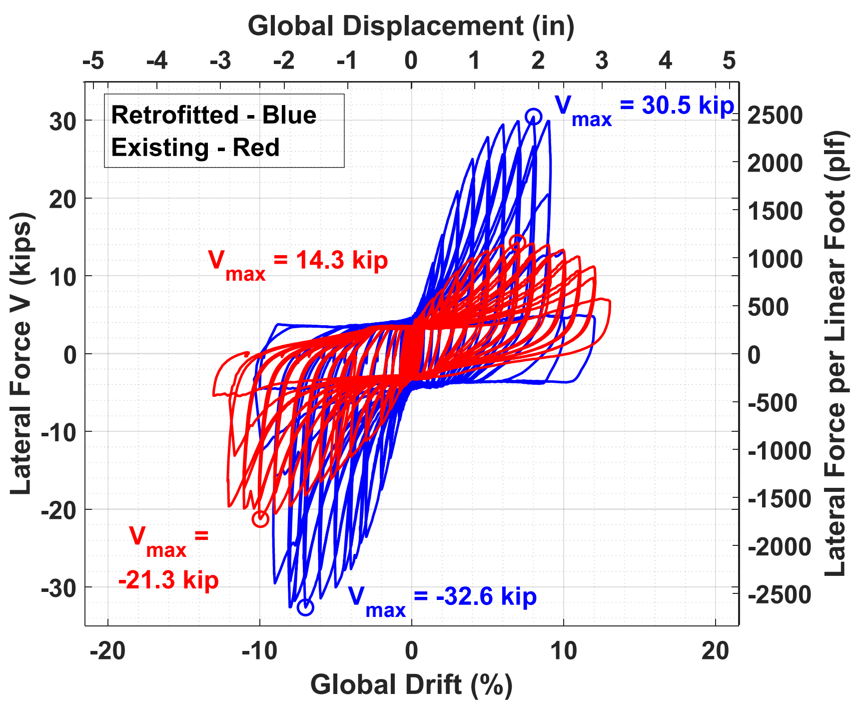


Figure 3.82 Specimens A-9 and A-10: comparison of *global* drift versus lateral load hysteretic response for retrofitted and existing, cripple walls with horizontal siding over diagonal sheathing.

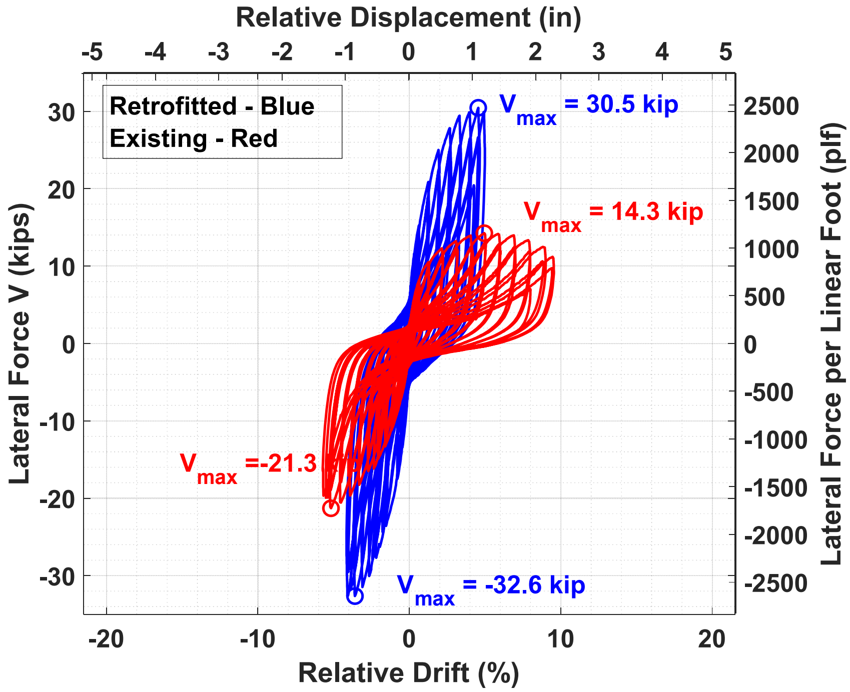


Figure 3.83 Specimens A-9 and A-10: comparison of *relative* drift versus lateral load hysteretic response for retrofitted and existing cripple walls with horizontal siding over diagonal sheathing

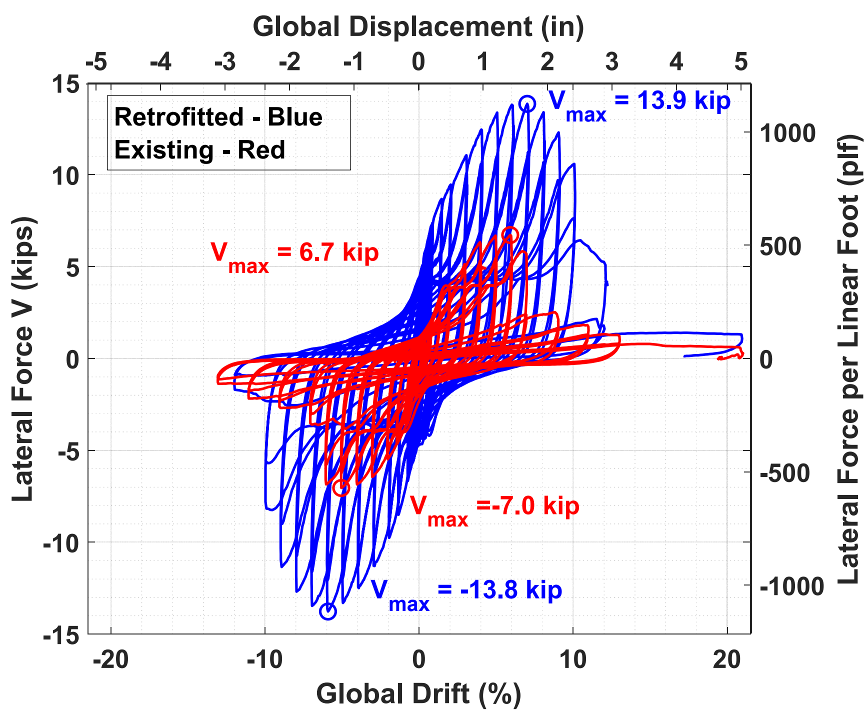


Figure 3.84 Specimens A-11 and A-12: comparison of *global* drift versus lateral load hysteretic response for retrofitted and existing cripple walls with T1-11 plywood.

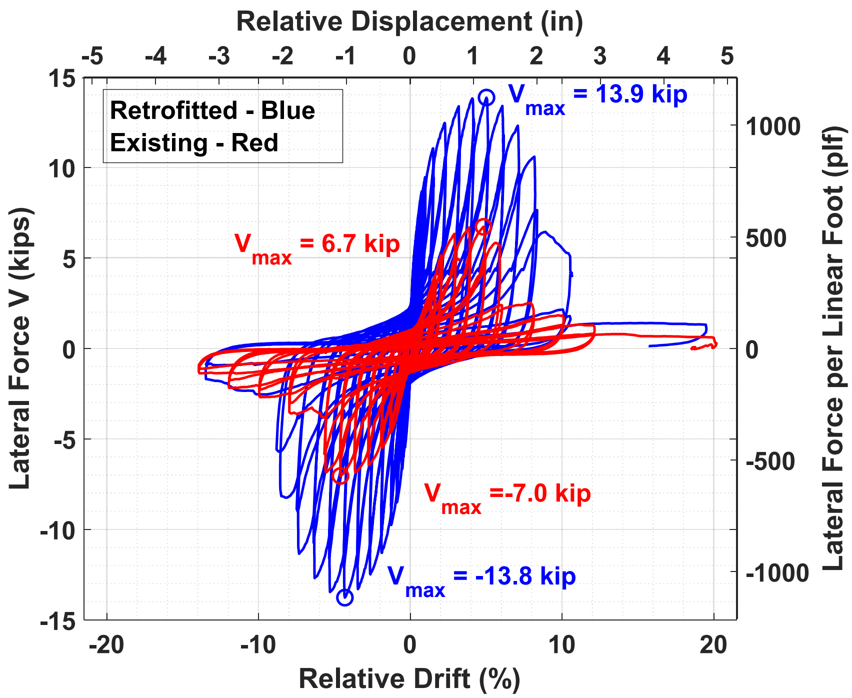


Figure 3.85 Specimens A-11 and A-12: comparison of *relative* drift versus lateral load hysteretic response for retrofitted and existing cripple walls with T1-11 plywood.

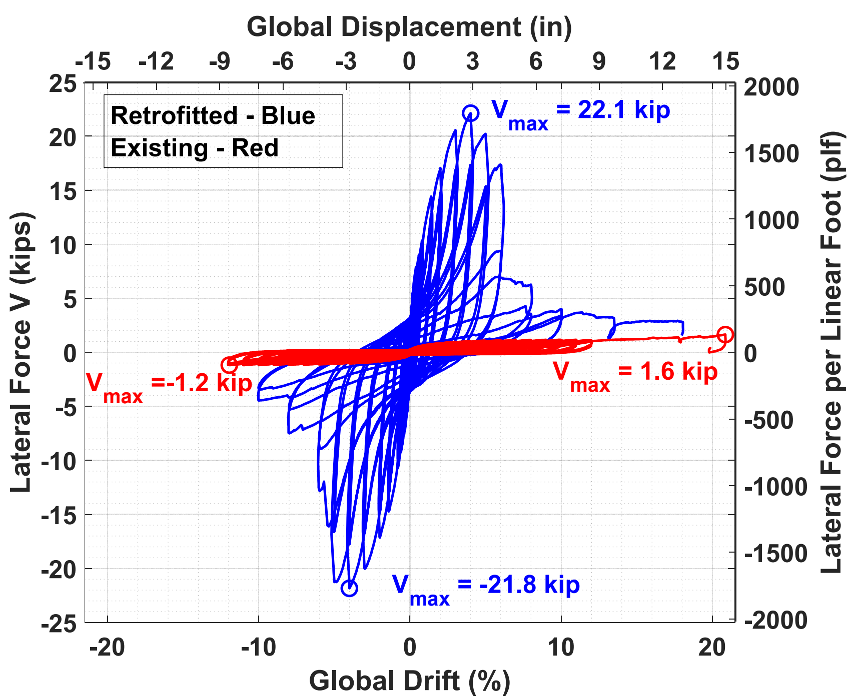


Figure 3.86 Specimens A-13 and A-14: comparison of *global* drift versus lateral load hysteretic response for retrofitted and existing 6-ft-tall cripple walls with horizontal siding.

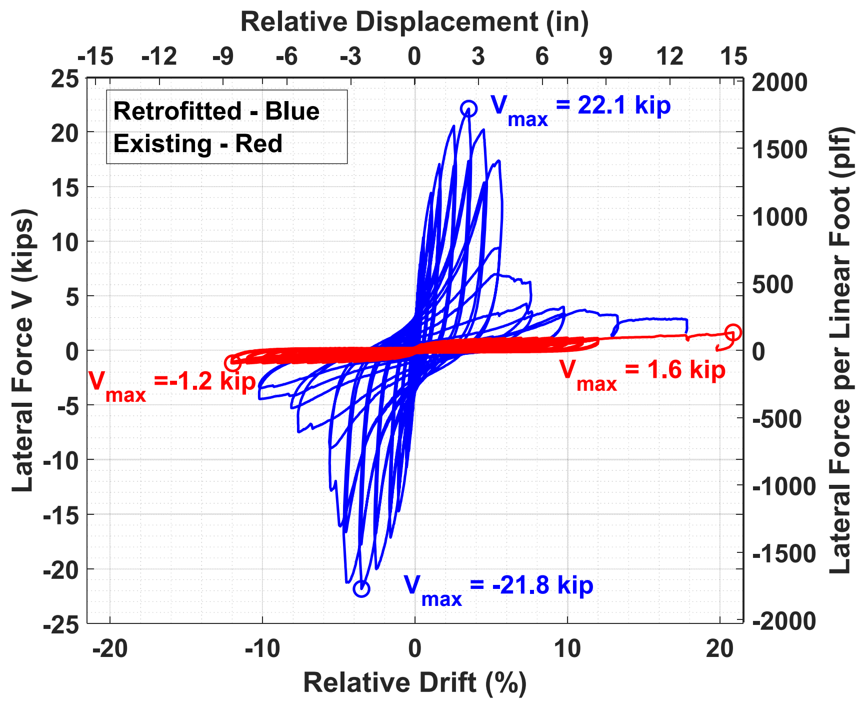


Figure 3.87 **Specimens A-13 and A-14: comparison of *relative* drift versus lateral load hysteretic response for retrofitted and existing 6-ft-tall cripple walls with horizontal siding.**

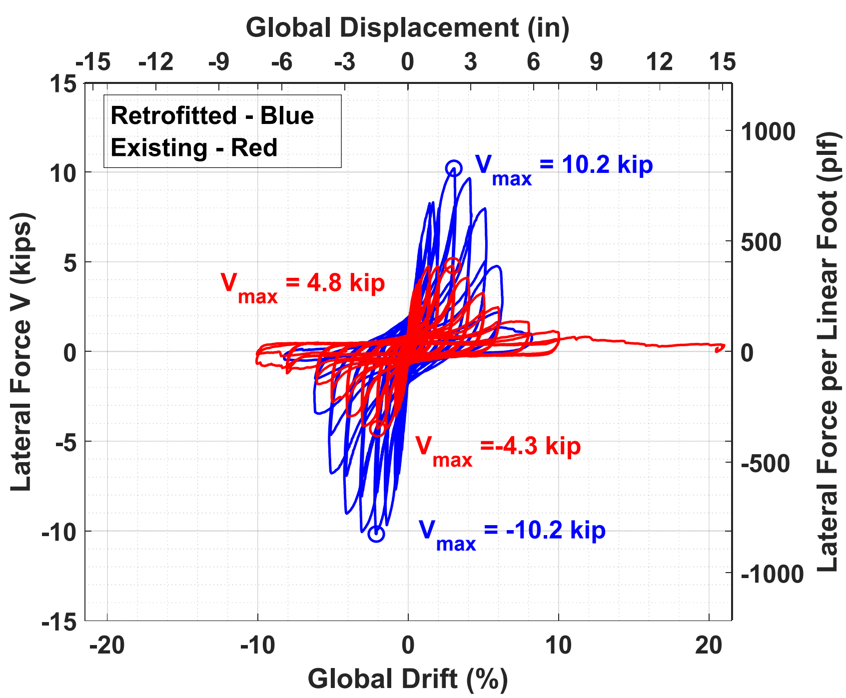


Figure 3.88 **Specimens A-23 and A-24: comparison of *global* drift versus lateral load hysteretic response for retrofitted and existing 6-ft-tall cripple walls with T1-11 plywood.**

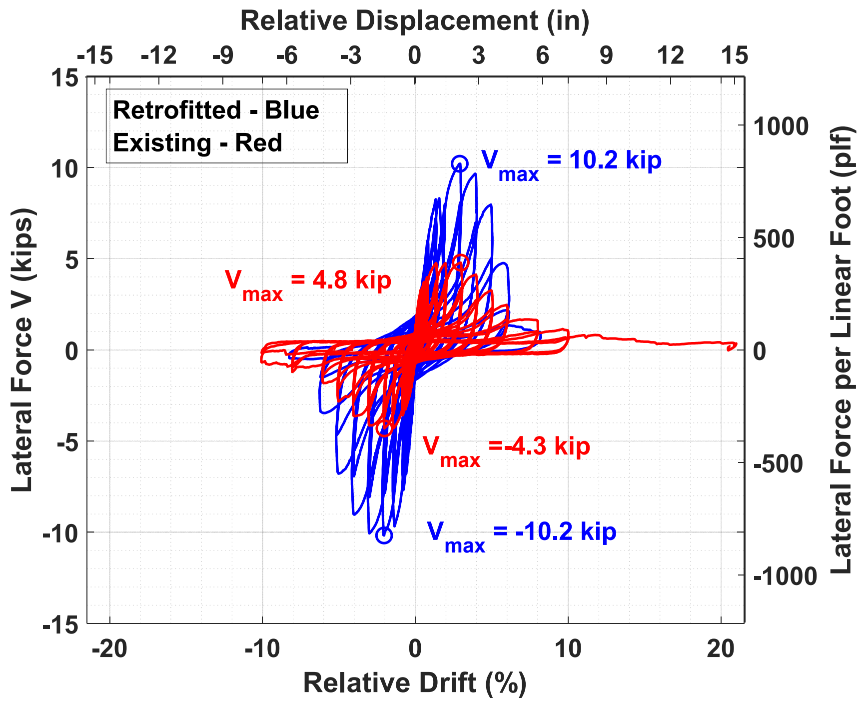


Figure 3.89 Specimens A-23 and A-24: comparison of *relative* drift versus lateral load hysteretic response for retrofitted and existing 6-ft-tall cripple walls with T1-11 plywood.

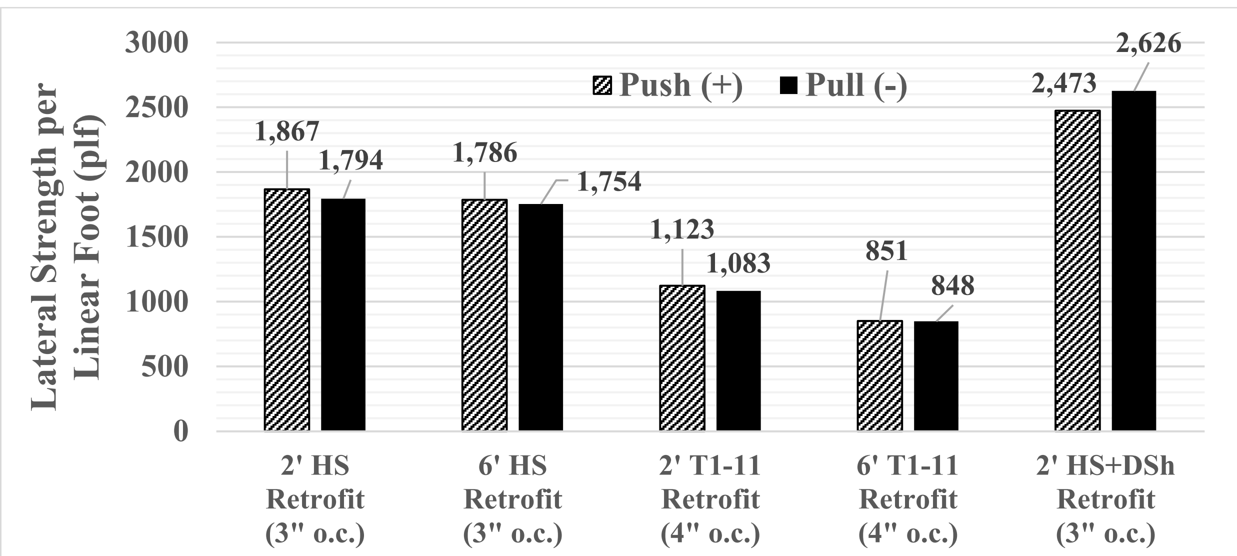


Figure 3.90 Lateral strength per linear foot for retrofitted specimens.

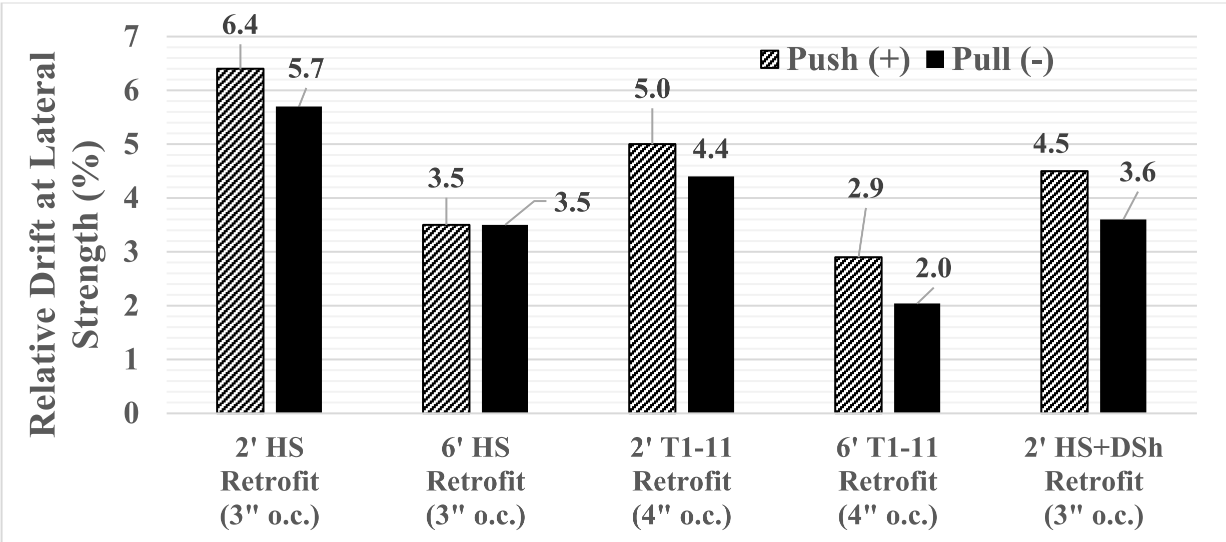


Figure 3.91 ***Relative*** **drift at lateral strength for retrofitted specimens.**

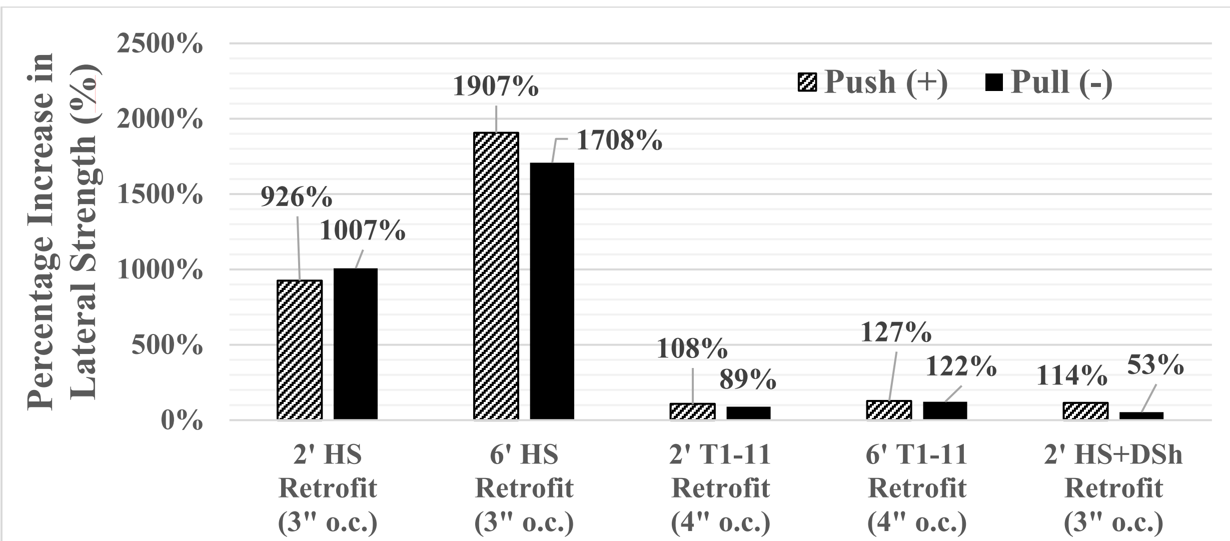


Figure 3.92 **Contribution of retrofit to lateral strength for retrofitted specimens.**

3.8 ENVELOPES OF HYSTERETIC RESPONSE

It is useful to compare the response of the cripple walls using overlays of the envelopes extracted from the lateral force–lateral displacement hysteresis. These curves are obtained by extracting the strength at each drift amplitude throughout the loading protocol. It is noted that only the leading cycles of each cycle group were considered in the development of these envelopes. Figures 3.93 through 3.99 show key comparisons of the cripple walls using the envelopes of each specimen's hysteresis. Both the push and pull loading direction is displayed in the same quadrant for ease of comparison. Figure 3.93 compares existing 2-ft-tall cripple walls with the three dry exterior finishes. The response for the horizontal siding cripple wall was nearly symmetric in the push and pull directions of loading, which is not the case for the horizontal siding over diagonal sheathing.

The orientation of the diagonal sheathing boards largely affected the response of the wall. Initially, the response was nearly symmetric, but at larger displacements in the pull loading direction, the gaps between the sheathing boards closed and the boards began to bear on one another. Once this occurred, the cripple wall continued to gain strength in this loading direction while losing strength in the opposite loading direction. It is evident that the diagonal sheathing boards provided drastic increases in both capacity (734% average increase) and stiffness (132% average increase for secant stiffness associated with relative drift at 80% strength).

In Figure 3.94, the same comparison is shown for the retrofitted counterparts. With the added retrofit, the differences in strength and stiffness were much less pronounced between the horizontal siding and horizontal siding over diagonal sheathing finished cripple walls. This horizontal siding specimen achieved higher strength than the T1-11 wood structural panel cripple wall, which is largely due to the increased number of nails attaching the plywood to the horizontal siding finished wall versus the T1-11 finished wall (3 in. o.c. and 4 in. o.c.). The added retrofit produced a much more symmetric response for the horizontal siding over diagonal sheathing cripple wall, but if the tests had not been terminated by the anchor bolts fracturing, the sheathing boards bearing on each other likely would have produced an asymmetric response at larger displacements.

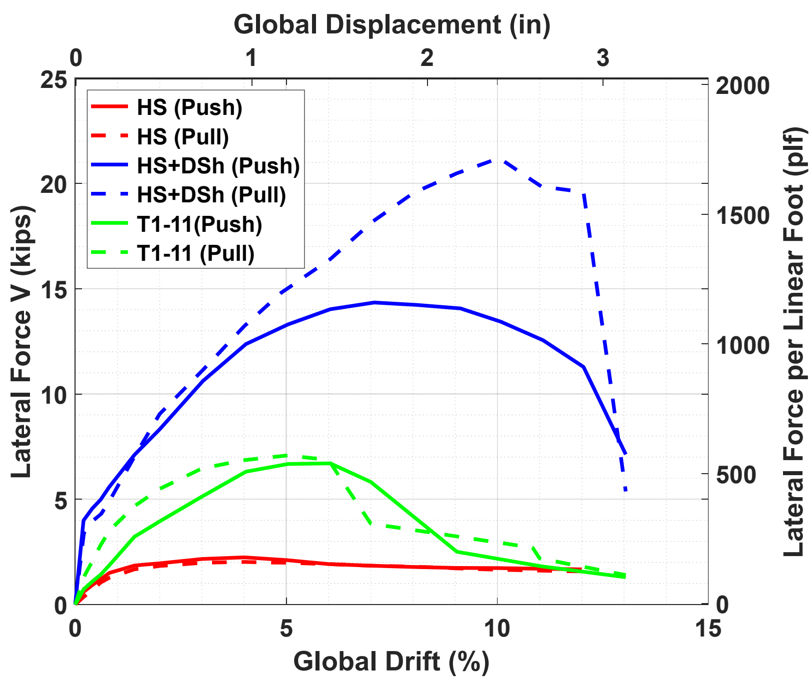


Figure 3.93 **Comparison of existing Specimens A-8, A-10, and A-12 finish materials: envelopes of *global* drift versus lateral strength.**

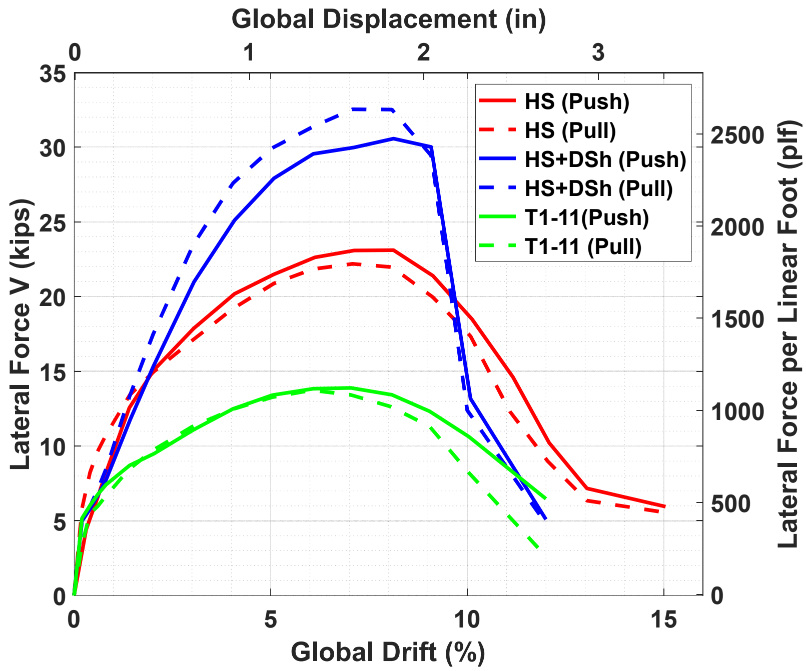


Figure 3.94 Comparison of retrofitted Specimens A-8, A-10, and A-12 finish materials: envelopes of *global* drift versus lateral strength.

Figure 3.95 compares the envelopes for the existing cripple walls with horizontal siding over diagonal sheathing finishes. These were the only pair of specimens to have a difference in applied vertical load. Recall that the low vertical load is 150 plf and representative of the gravity load of a single-story dwelling with light construction materials, while the heavy vertical load is 450 plf and mimics the weight of a two-story dwelling with heavy construction materials. The increase in strength regardless of loading directions was similar. Increase of vertical load produced a 47% increase in strength in the push loading direction and 53% increase in the pull loading direction. The secant stiffness associated with relative drift at 80% strength was nearly identical for both specimens, showing that the increased vertical load had little influence on the stiffness of the cripple walls.

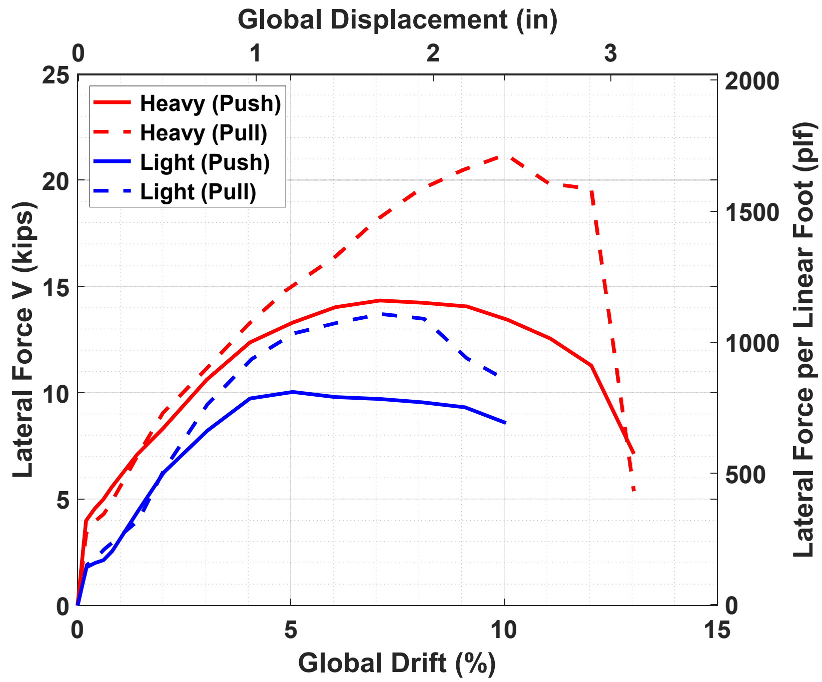


Figure 3.95 **Specimen A-9 and A-28 comparison of envelopes of *global* drift versus lateral strength: varying vertical load.**

Comparisons of the envelopes of global drift versus lateral strength for 2-ft-tall and 6-ft-tall cripple walls are given in Figures 3.96 to 3.99. Figure 3.96 shows the existing cripple walls finished with horizontal siding. The strength was twice as much for the 2-ft-tall specimen compared to the 6-ft-tall specimen. The 6-ft-tall cripple wall continued to gain strength throughout the entire test, whereas the 2-ft-tall specimen peaked a 4% drift. There was only a 25% drop in capacity for the 2-ft-tall specimen from 4% to 12% drift. The response of both specimens demonstrates how flexible and weak cripple walls are when finished with horizontal siding. Since the orientation of the sheathing is aligned with the direction of loading, the lateral resistance is primarily achieved through the moment couple between the nails fastening the boards to the studs. The secant stiffness associated with relative drift at 80% strength was 22 times higher for the 2-ft-tall cripple wall than the 6-ft-tall cripple wall. With the added retrofit, the strength for the horizontal siding finished cripple walls was much more similar, as shown in Figure 3.97. The drift at peak for the 2-ft-tall specimen occurred at 8% whereas it was achieved at 4% for the 6-ft-tall specimen. This is largely due to the anchor of the specimens. The 6-ft-tall wall was constructed with two additional anchor bolts as well as tie-downs at both ends, which reduced the amount of relative displacement between the sill plate and the foundation. The secant stiffness associated with relative drift at 80% strength was 94% higher for the 2-ft-tall cripple wall than the 6-ft-tall cripple wall. Because taller walls are expected to be more flexible than shorter walls, it is important to note that the addition of the retrofit drastically reduced gap between the secant stiffnesses of the 2-ft-tall and 6-ft-tall cripple walls. In addition, the relative drift at strength was higher for the 2-ft-tall specimen than the 6-ft-tall specimen due to the added retrofit.

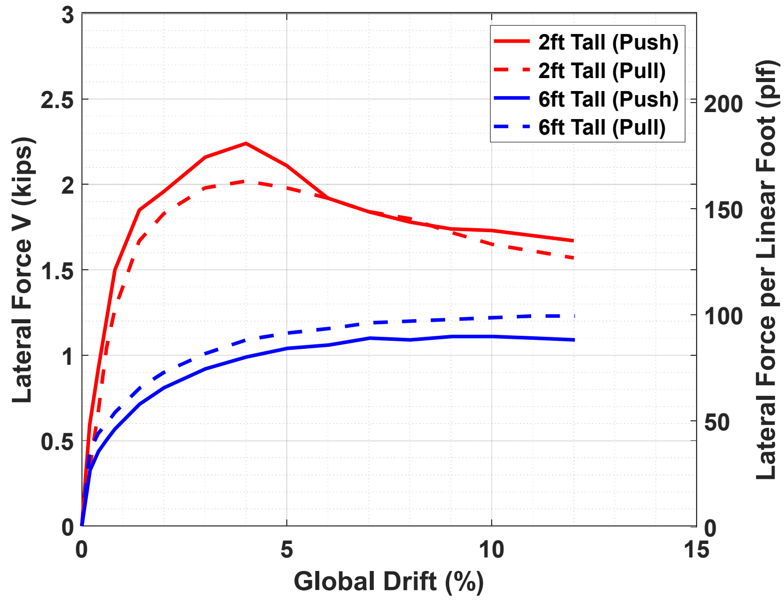


Figure 3.96 Specimen A-7 and A-13 comparison of envelopes of *global* drift versus lateral strength: response for the existing 2-ft-tall and 6-ft-tall cripple walls with horizontal siding.

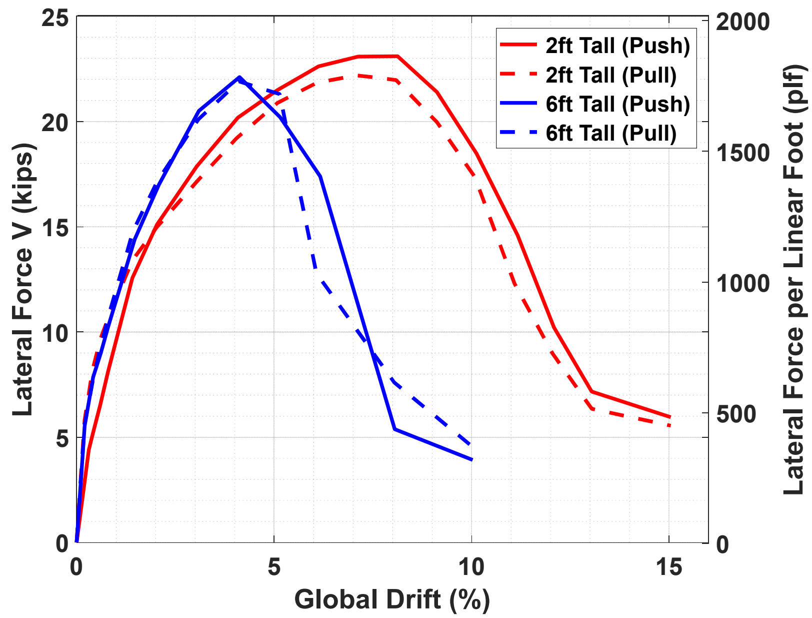


Figure 3.97 Specimen A-8 and A-14 comparison of envelopes of *global* drift versus lateral strength: response for the retrofitted 2-ft-tall and 6-ft-tall horizontal siding finished cripple walls.

Figure 3.98 shows the envelopes of global drift versus lateral strength hysteretic response for the existing 2-ft- and 6-ft-tall T1-11 wood structural panel finished cripple walls; Figure 3.99 shows the same for the retrofitted specimens. The relative responses regardless of the retrofit condition are strikingly similar. The main difference between the existing and retrofit specimen is the decrease of nail spacing from 8 in. on center to 4 in. on center. By doubling the amount of edge, the strength is around doubled for both cripple walls, regardless of height. The increase in height reduced the strength by 32% for the existing specimens and 23% for retrofit specimens. The response was more symmetric for the retrofitted specimens due to the row of nails added to the panel overlaps, which was present for the existing specimens.

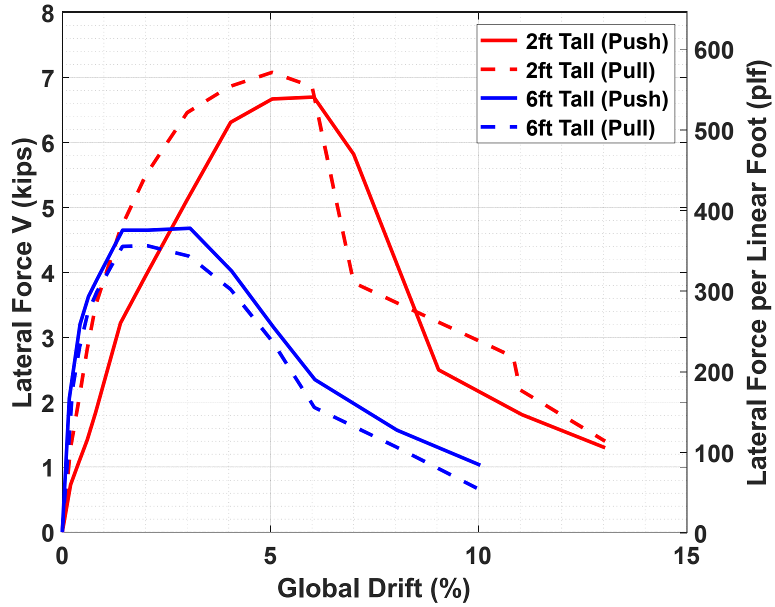


Figure 3.98 Specimen A-11 and A-23 comparison of envelopes of *global* drift versus lateral strength: response for the existing 2-ft- and 6-ft-tall cripple walls finished with T1-11 wood structural panels.

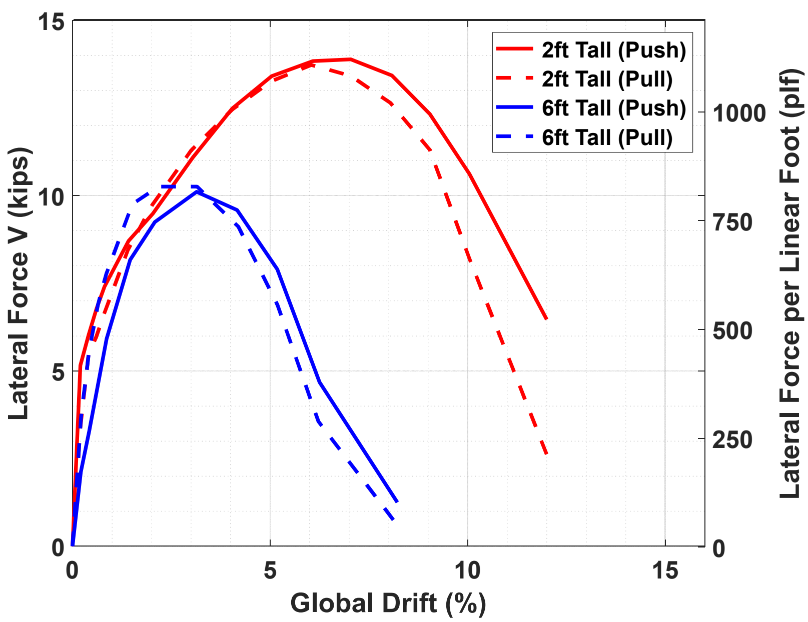


Figure 3.99 Specimen A-12 and A-24 comparison of envelopes of *global* drift versus lateral strength: response for the retrofitted 2-ft- and 6-ft-tall cripple walls finished with T1-11 wood structural panels.

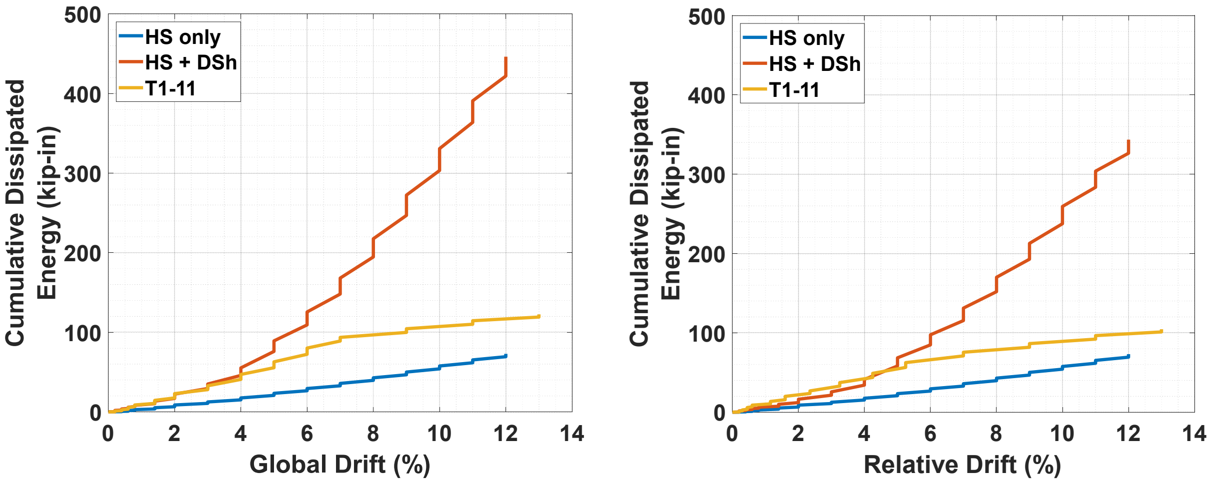
3.9 HYSTERETIC ENERGY DISSIPATION

An important characteristic to describe the seismic resiliency of a cripple wall is the energy dissipated by the cripple wall during loading. Figures 3.100 through 3.104 show various comparisons of the cumulative energy dissipated versus drift. The cumulative energy dissipated was calculated as the sum of area of the hysteretic loops in both push and pull loading direction for each cycle level group. The energy dissipated was calculated for both the leading and the trailing cycles in both the push and pull directions of loading. Both the relative and global responses are presented. These responses differed largely if the cripple wall slid on the foundation, as the friction between the sill plate and the foundation dissipates a significant amount of energy.

Figure 3.100 compares the cumulative energy dissipated for existing, 2-ft-tall cripple walls tested. The three walls varied in their exterior finishes: shiplap horizontal lumber siding, shiplap horizontal lumber siding over diagonal lumber sheathing, and T1-11 wood structural panels. The cripple wall with horizontal siding over diagonal sheathing dissipated almost 250% more energy than the T1-11 cripple wall and over 500% more energy than the horizontal siding cripple wall. Considering the relative drift response, the horizontal siding over diagonal sheathing cripple wall dissipated over 230% more energy than the T1-11 cripple wall and almost 350% more energy than the horizontal siding cripple wall. It should be noted that the horizontal siding over diagonal sheathing cripple wall failed due to fracturing of all the anchor bolts; therefore, the amount of energy dissipated would have been expected to continue to increase in later displacement cycles.

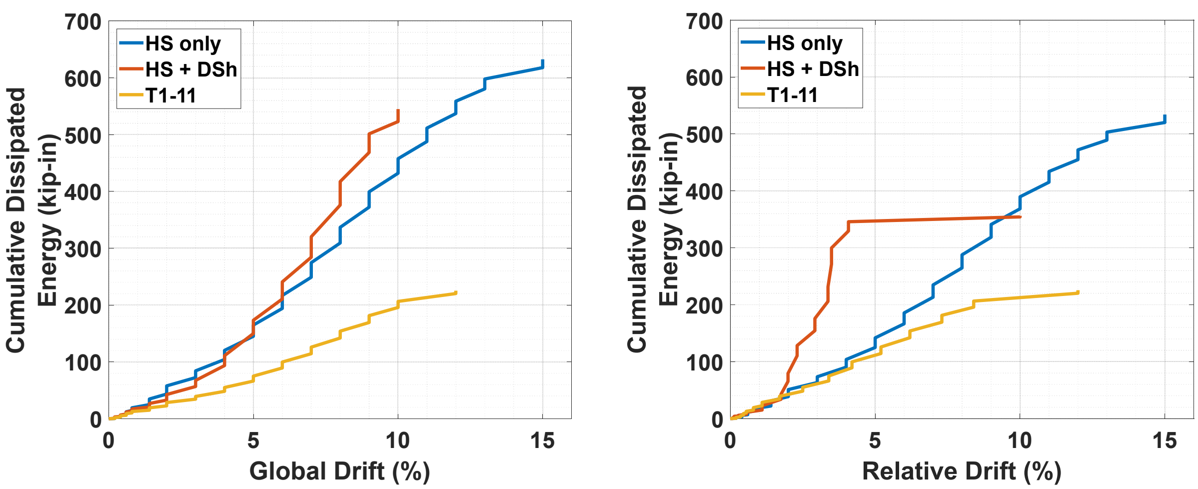
Figure 3.101 compared the retrofitted, 2-ft-tall cripple walls with the same exterior finishes discussed in the previous figure. By the end of the test, the horizontal siding cripple wall dissipated the most energy, but the horizontal siding over diagonal sheathing cripple wall had dissipated over 250% more energy than either of the other cripple walls by 4% relative drift. The horizontal siding over diagonal sheathing had the largest displacement of the sill plate relative to the foundation of any of the retrofitted 2-ft-tall cripple walls. If considering the global response by 10% global drift, the horizontal siding over diagonal sheathing cripple wall had dissipated around 20% more energy than the horizontal siding cripple wall and 170% more energy than the T1-11 cripple wall. Like the existing cripple wall, the retrofitted horizontal siding over diagonal sheathing cripple wall failed due to fracturing of the anchor bolts; therefore, it is expected that the energy dissipation would have significantly increased if the anchor bolts had stayed intact.

Figure 3.102 compares the energy dissipated by the existing 2-ft-tall cripple walls with horizontal siding over diagonal sheathing. The two cripple walls differed in the vertical load applied on the wall. The heavy vertical load was 450 plf, and the light vertical load was 150 plf. From the global response, the heavy vertical load wall dissipated around 35% more energy by 10% drift. The light vertical wall was trending to dissipate more energy than the light vertical load cripple wall through a comparison of the relative drift response. Because the light vertical load cripple wall failed prematurely due to the sill plate splitting across its entire span, it is difficult to tell how much energy both cripple walls would have dissipated had it not been for failures of the sill plate or the anchor bolts.



(a) (b)

Figure 3.100 The existing 2-ft-tall cripple wall hysteretic energy dissipation comparison: (a) *global* response; (b) *relative* response.



(a) (b)

Figure 3.101 The retrofitted 2-ft-tall cripple wall hysteretic energy dissipation comparison: (a) *global* response; and (b) *relative* response.

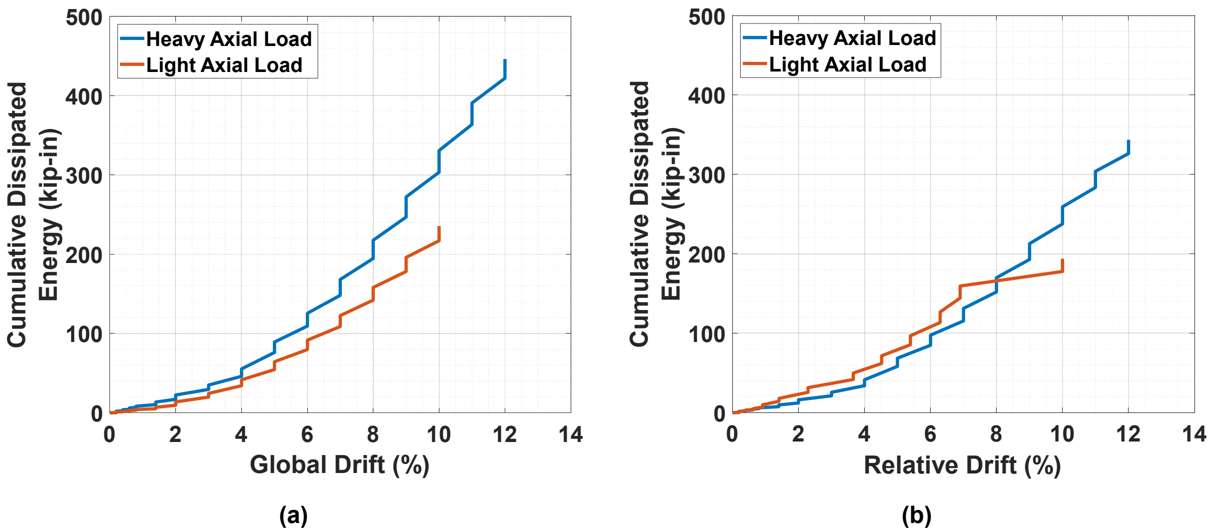


Figure 3.102 Horizontal siding over diagonal sheathing vertical load hysteretic energy dissipation comparison: (a) *global* response; and (b) *relative* response.

Figure 3.103 compares the energy dissipated of the existing and retrofitted, 2-ft-tall and 6-ft-tall cripple walls with horizontal siding. For both the existing and retrofitted cases, the 6-ft-tall cripple walls dissipated more energy than their 2-ft-tall counterparts even though both walls had lower peak strengths. In terms of both the global and the relative response, the existing 2-ft-tall specimen dissipated nearly 50% energy more than its counterpart. For the 6-ft-tall specimens, the loading protocols diverged at 6% global drift (4.9% relative drift); at this point the retrofitted specimen dissipated nearly 200% more energy in the global response (240% more energy in the relative response) than existing specimen. The increased energy dissipation is largely due to the amount of displacement imposed on the 6-ft-tall specimens being three times as much as the 2-ft-tall specimens, as well as the increased number of fasteners attaching the horizontal siding and plywood doing work. The added retrofit accounted for a nearly a 700% increase in the energy dissipated by 10% global drift for the 2-ft-tall cripple walls and over an 850% increase for the 6-ft-tall cripple walls. Horizontal siding had by far the lowest capacity of any of the dry exterior finishes; therefore, the added retrofit would naturally provide large increases in the amount of energy dissipated.

Figure 3.104 shows the hysteretic energy dissipation of the global and relative response for the 2-ft- and 6-ft-tall specimens finished with T1-11 wood structural panels for both the existing and retrofitted condition. The loading protocols for each of the specimens were the same up to the 6% global drift ratio level. At this point, the retrofitted, 2-ft-tall cripple wall dissipated nearly 50% more than its existing counterpart, and the retrofitted, 6-ft-tall specimen dissipated over 110% more energy than the existing specimen. There was a 70% increase in the energy dissipation at this point from the existing to retrofitted 2-ft-tall specimens, and a 150% increase from the existing to retrofitted 6-ft-tall specimens. As with the horizontal siding finished specimens, the increased dissipation was due to the increased number of fasteners in the taller walls as well as the imposed displacements being three times as large.

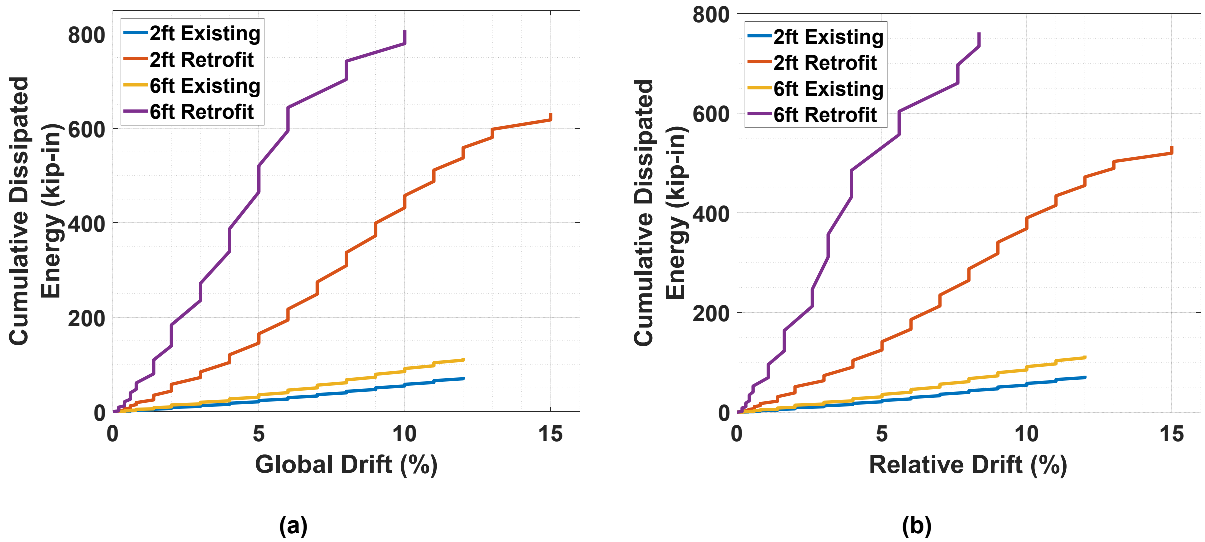


Figure 3.103 Horizontal siding hysteretic energy dissipation comparison: (a) ***global*** **response; and (b)** ***relative*** **response.**

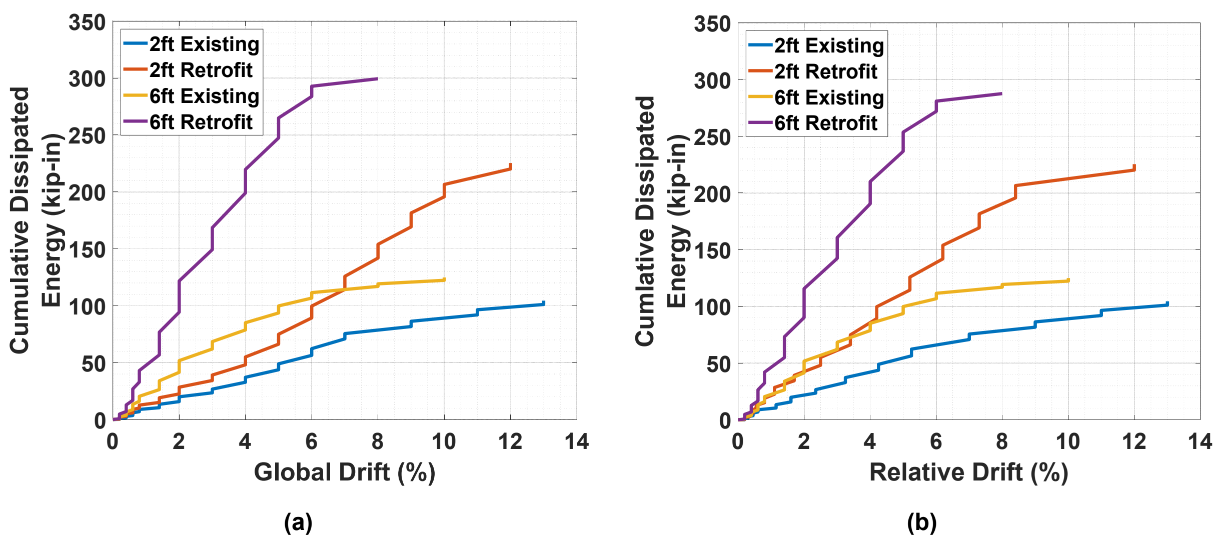


Figure 3.104 T1-11 wood structural panel hysteretic energy dissipation comparison: (a) ***global*** **response; and (b)** ***relative*** **response.**

3.10 RESIDUAL DRIFT

As the cripple walls were cyclically loaded, they accumulated residual deformation. Residual deformation is an effective parameter to evaluate the structural performance of a cripple wall under seismic excitation. In addition, residual deformation represents the final state of a structure after an earthquake, thus making it a concern for homeowners as the aesthetic and structural performance of the dwelling are both affected. The residual displacement of the cripple walls was measured at the end of each displacement cycle level and can be defined as the amount of displacement in the cripple wall measured when there is no lateral force being imposed on the cripple wall. As the amplitude of the displacement increased, the residual displacement increased to the point where it became visible, even prior to the cripple walls' achieving full strength. Figure 3.105 shows the global residual displacement of the cripple walls after the 1.4% drift cycle group. Global residual displacement refers to not only the residual displacement of the cripple wall itself but also to the residual displacement of the sill plate relative to the foundation. Figure 3.109 shows the relative residual displacement of the cripple walls after the 1.4% drift cycle group. The relative residual displacement accounts for only the deformation sustained in the cripple wall, excluding any deformation of the sill plate relative the foundation. For convenience, the relative residual displacement will be referred to as residual displacement. This measurement is a better indicator of the structural performance of the cripple wall as it only accounts for the cripple wall's residual deformation. There were variations in the alignment of the sill plate connection to the foundation as the anchor bolt holes were oversized ¼ in. It should be noted that the residual displacements are not normalized by any height metric within Figures 3.105 through 3.109. Naturally, the 6 ft-tall-cripple walls would have more residual displacement than their 2-ft-tall counterparts due to the imposed displacement being three times as much for the 6-ft-tall cripple walls than the 2-ft-tall cripple walls.

For 2-ft-tall cripple walls, the global residual displacement was 0.18 to 0.34 in. or 0.8%–1.4% drift; see Figure 3.105. The largest global residual displacement was for Specimen A-28, which had a horizontal siding over diagonal sheathing finish and a light vertical load. There was little difference between the global residual displacement for the existing and retrofitted cripple walls. For the 6-ft-tall cripple walls, the ranges in global residual displacement were from 0.51 to 0.59 in. or 0.7% to 0.8% drift. In terms of global residual displacement as a percentage drift, the height of the cripple wall had little effect on the residual displacement. When looking at the residual displacement at 1.4% global drift (Figure 3.106), the cripple walls with horizontal siding remained fairly consistent, except for the retrofitted 6-ft-tall specimen. The cripple walls with horizontal siding over diagonal sheathing had dramatic differences between the global residual displacement and the residual displacement, decreasing from 0.22–0.34 in. to 0.03–0.06 in. This was caused by the large amounts of displacement that accrued between the sill plate and the foundation as the cripple wall slid along the foundation instead of the wall itself deforming. This figure also indicates the relative drift of the cripple walls at 1.4% global drift.

It is more useful to compare the residual displacements in the cripple walls at the same relative drift amplitude. If a linear interpolation is made to determine the residual displacement at 1.4% relative drift, the residual displacements are much more consistent, as shown in Figure 3.107 For the cripple walls with horizontal siding, the residual displacement is fairly consistent, regardless of retrofit condition and height, e.g., between 0.18 and 0.19 in. (0.7–0.8% drift) for the 2-ft-tall specimens and between 0.52 and 0.58 in. (0.7–0.8% drift) for the 6-ft-tall specimens. The same trend is exhibited for the T1-11 finished cripple walls. The residual displacement is between 0.19 and 0.21 in. (~0.8% drift) for the 2-ft-tall specimens and between 0.58 and 0.59 in. (~0.8% drift) for the 6-ft-tall specimens. These results are fairly consistent with the horizontal siding cripple walls. The cripple walls with horizontal siding over diagonal sheathing have significantly less residual displacement at 1.4% relative drift, in the range of 0.08 to 0.13 in. (0.3–0.5% drift). The cripple wall with the light vertical load is on the lower end of that range, which is due to the decreased amount of weight on top of the cripple wall keeping it from returning to its undeformed position. Overall, the horizontal siding over diagonal sheathing finish combination is more elastic than the other finish material combinations, which is likely attributed to the orientation of the diagonal sheathing providing increased lateral resistance.

Figure 3.109 shows the global residual displacement at strength. Since the strengths occurred over a wide range of drifts and the amount of sill plate to foundation displacement varies drastically between specimens, there are not as many decipherable trends between the walls. Figure 3.108 shows the residual displacement at lateral strength; it is evident that the T1-11 and horizontal siding finishes have comparable residual displacement between the existing cripple walls and the retrofitted cripple walls for the 2-ft-tall specimens. This is as much the same for the case of the 6-ft-tall walls, where the existing cripple wall with horizontal siding has a residual displacement of 3.7 in. This is by far the most flexible cripple wall tested. The retrofitted cripple wall with horizontal siding over diagonal sheathing had the lowest residual displacement, 0.26 in. This is less than half the residual displacement at peak strength of any of the other cripple walls.

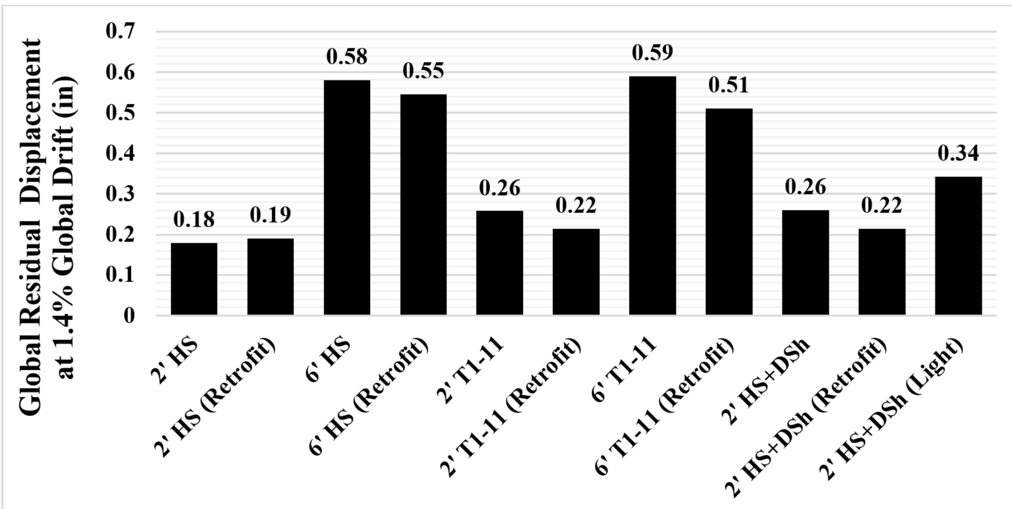


Figure 3.105 ***Global*** **residual displacement of cripple walls at the end of the 1.4% global drift cycle group.**

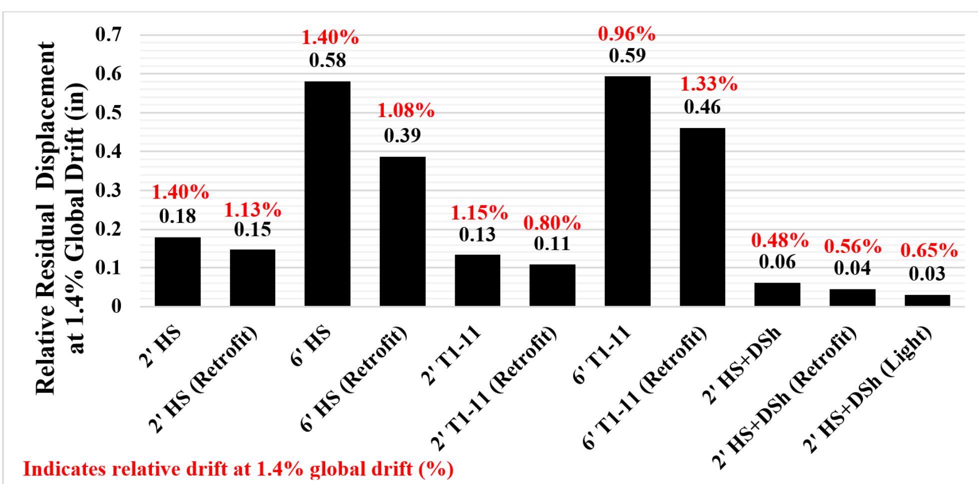


Figure 3.106 ***Relative*** **residual displacement of cripple walls at the end of the 1.4% global drift cycle group.**

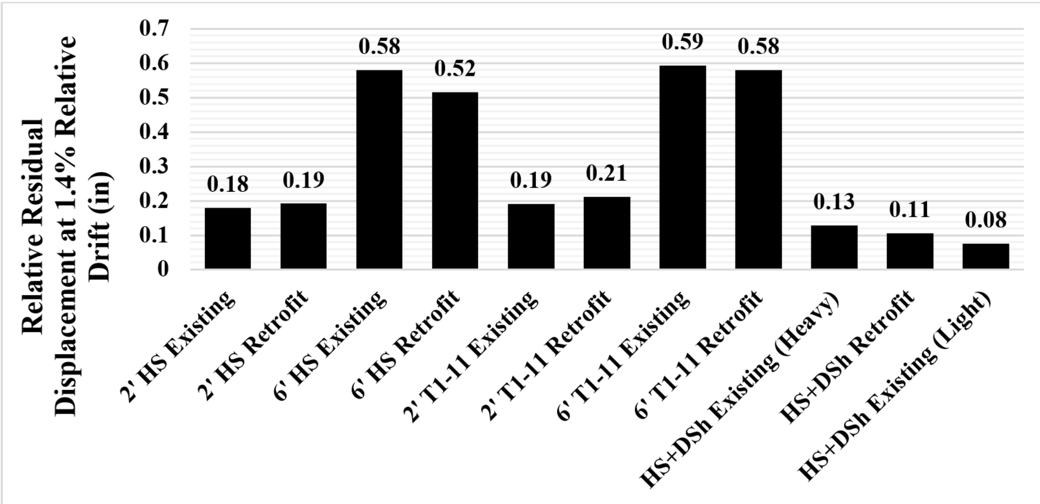


Figure 3.107 *Relative* residual displacement of cripple walls at the end of the 1.4% relative drift cycle, linearly interpolated.

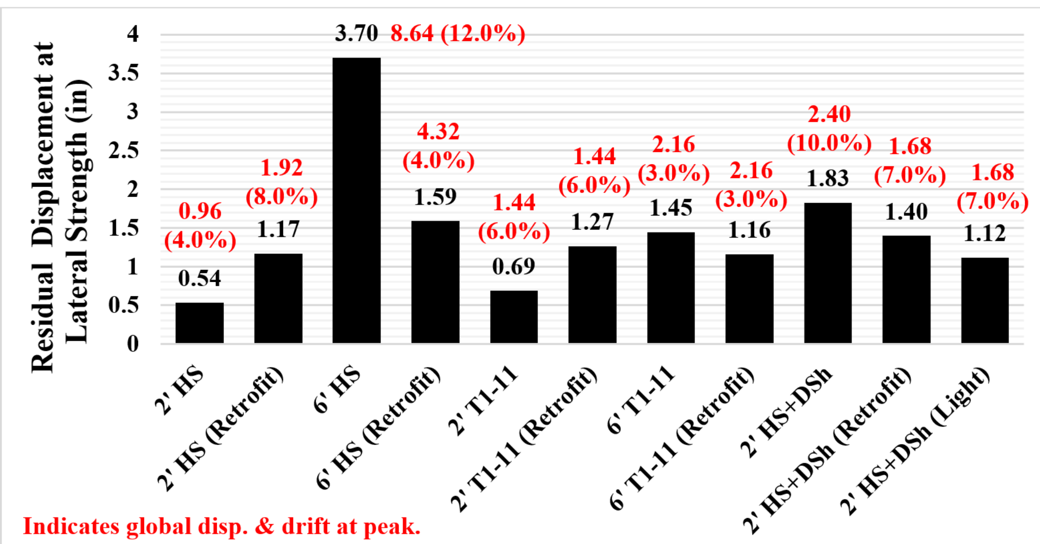


Figure 3.108 *Global* residual displacement of cripple walls at the end of the peak strength drift cycle group.

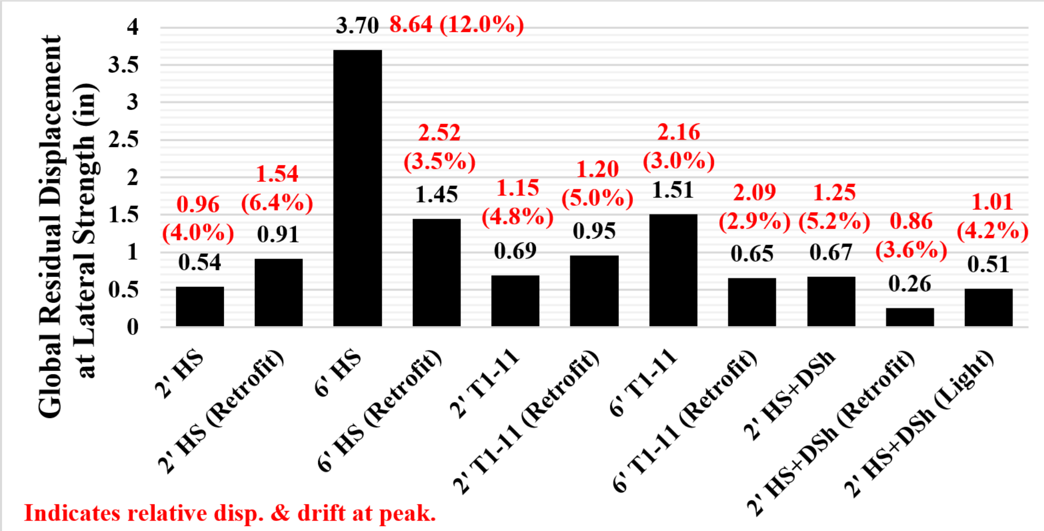


Figure 3.109 ***Relative*** **residual displacement of cripple walls at the end of the peak strength drift cycle group.**

3.11 VERTICAL LOAD

The vertical load was applied vertically with two 4 in. × 4 in. × 3/8 in. HSS members acting as point loads, using four hydraulic jacks connected to four rods. The hydraulic jacks use the ceiling of the strong floor as a reaction point. The load is measured with four vertical load cells, one for each rod. The connection of the rods to the hydraulic jacks are only able to rotate, creating a pinned connection at the ceiling of the strong floor. As the cripple walls displace, the applied load starts to develop a horizontal component that needs to be included in the actual horizontal force being applied to the cripple wall. Since the horizontal component opposed the measured lateral force, the corrected lateral force would be a reduced measured lateral force. The vertical load experienced by the cripple wall is also reduced due to the displacement of the cripple wall, but to a negligible degree.

Figure 3.110 shows the set up for the application of the vertical load, and Figure 3.111 shows the geometry of the vertical load and lateral load as the cripple wall displaces. Overall, the correction for the lateral load was a reduction in the range of 0–3% for all cycles for the 2-ft-tall cripple walls and 0–6% for 6-ft-tall cripple walls. During the monotonic push, the correction would have a maximum reduction of around 5% for 2-ft-tall cripple walls and 10% for 6-ft-tall cripple walls. Note that all results presented have accounted for these corrections. The only exception to the aforementioned vertical load application was for Specimen A-28, which had a lighter vertical load. For this specimen, the vertical load came from steel plates spanning the top of the horizontal steel transfer beam. The same system of applying the load was not used due to the lack of accuracy and precision in the hydraulic system at such low loads. The equation for the corrected lateral load and corrected vertical load are as follows:

$$V_{actual} = V_{measured} - P_{rod}\sin(\theta_{rod}),$$

$$P_{actual} = P_{rod}\cos(\theta_{rod}), \text{ where } \theta_{rod} = \sin^{-1}\frac{\Delta}{L_{rod}}$$

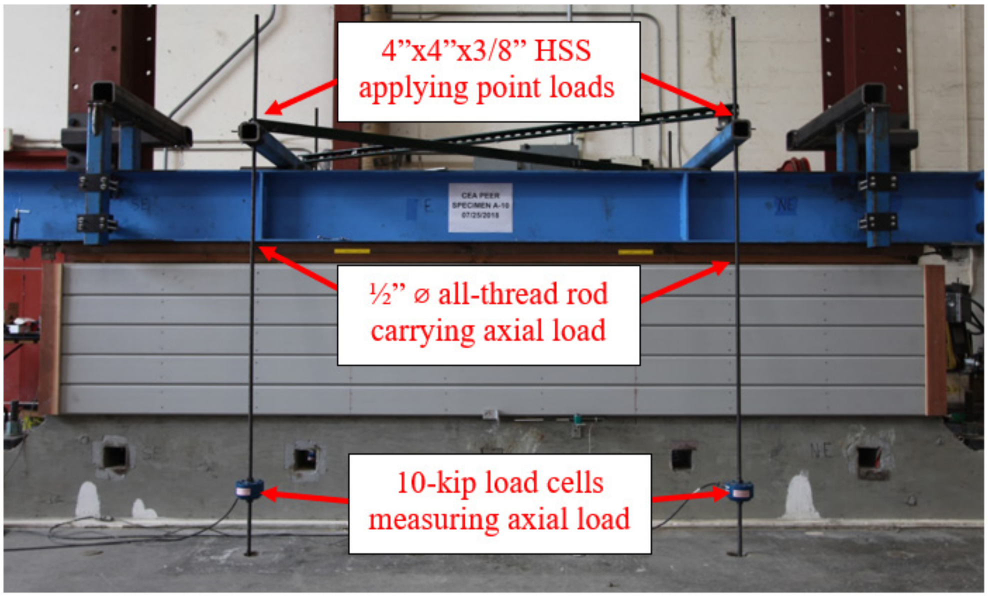


Figure 3.110 Axial load set up.

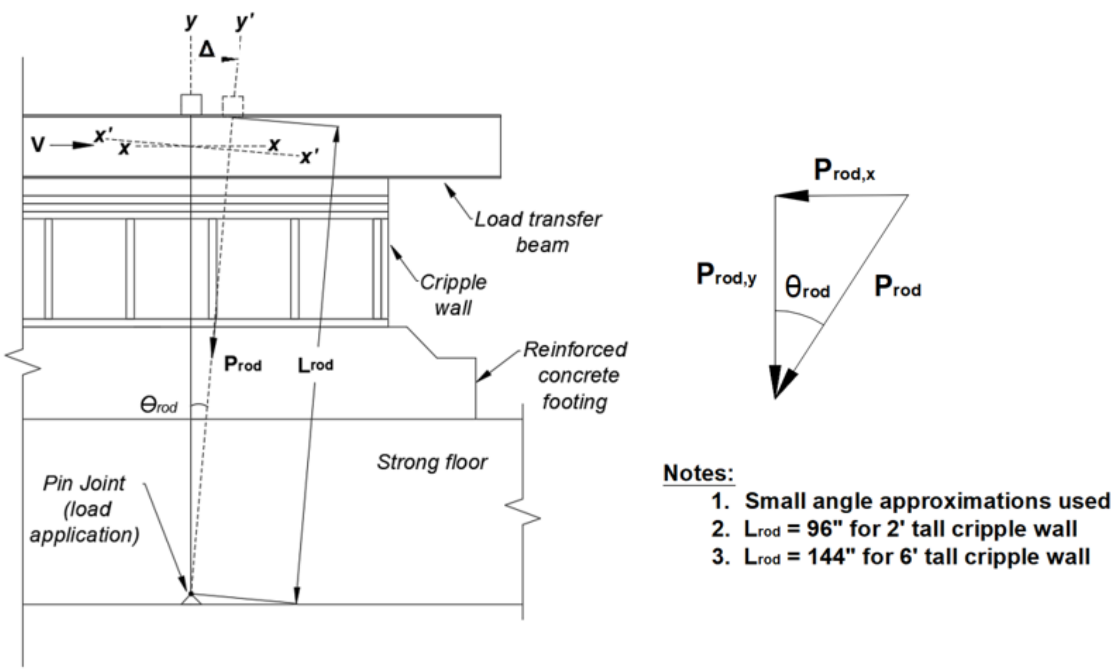


Figure 3.111 Schematic of displaced geometry for lateral load correction.

The vertical load for all specimens but Specimen A-28 was 450 plf. Specimen A-28 had a vertical load of 150 plf. The 450 plf load is representative of the weight of a two-story dwelling with heavy building materials. The 150 plf vertical load is representative of a one-story dwelling with light building materials. In order to achieve 450 plf of vertical load, 5 kips were applied between the four hydraulic jacks after the weight of the horizontal load transfer beam, laminated wood beam, and HSS sections had been accounted for. Throughout the displacement cycles, the vertical load applied by the jacks would oscillate. These oscillations are shown in Figure 3.112. For all cripple walls, the vertical loads fluctuated from by a range of 1.3 to 3.7 kips over their entire loading protocol. The maximum vertical load experienced was 7.7 kips by Specimen A-8, and the lowest vertical load was 3.2 kips by Specimen A-14.

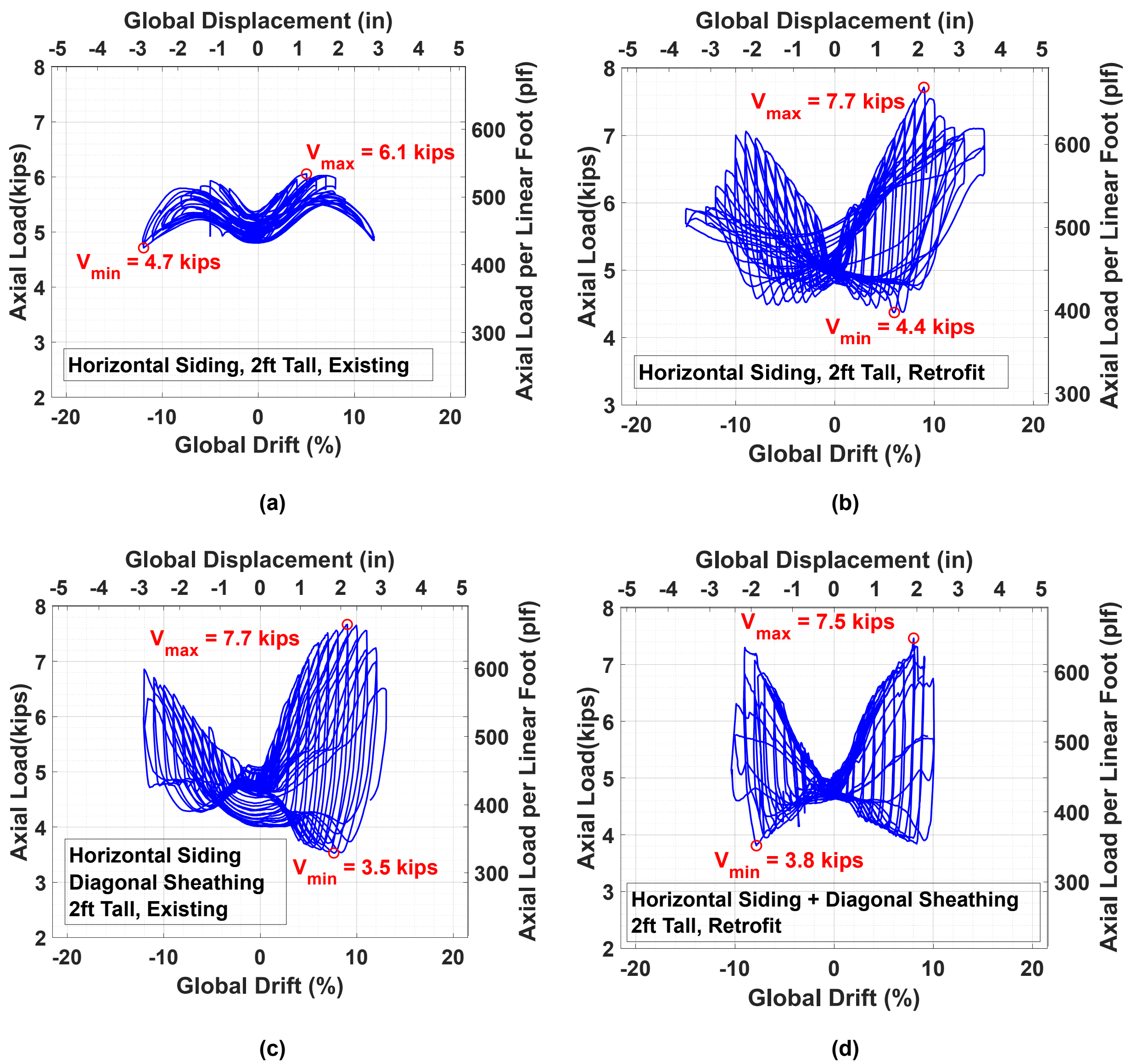


Figure 3.112 Axial (vertical) load versus global drift for specimens: (a) Specimen A-7; (b) Specimen A-8; (c) Specimen A-9; (d) Specimen A-10; (e) Specimen A-11; (f) Specimen A-12; (g) Specimen A-13; (h) Specimen A-14; (i) Specimen A-23; and (j) Specimen A-24.

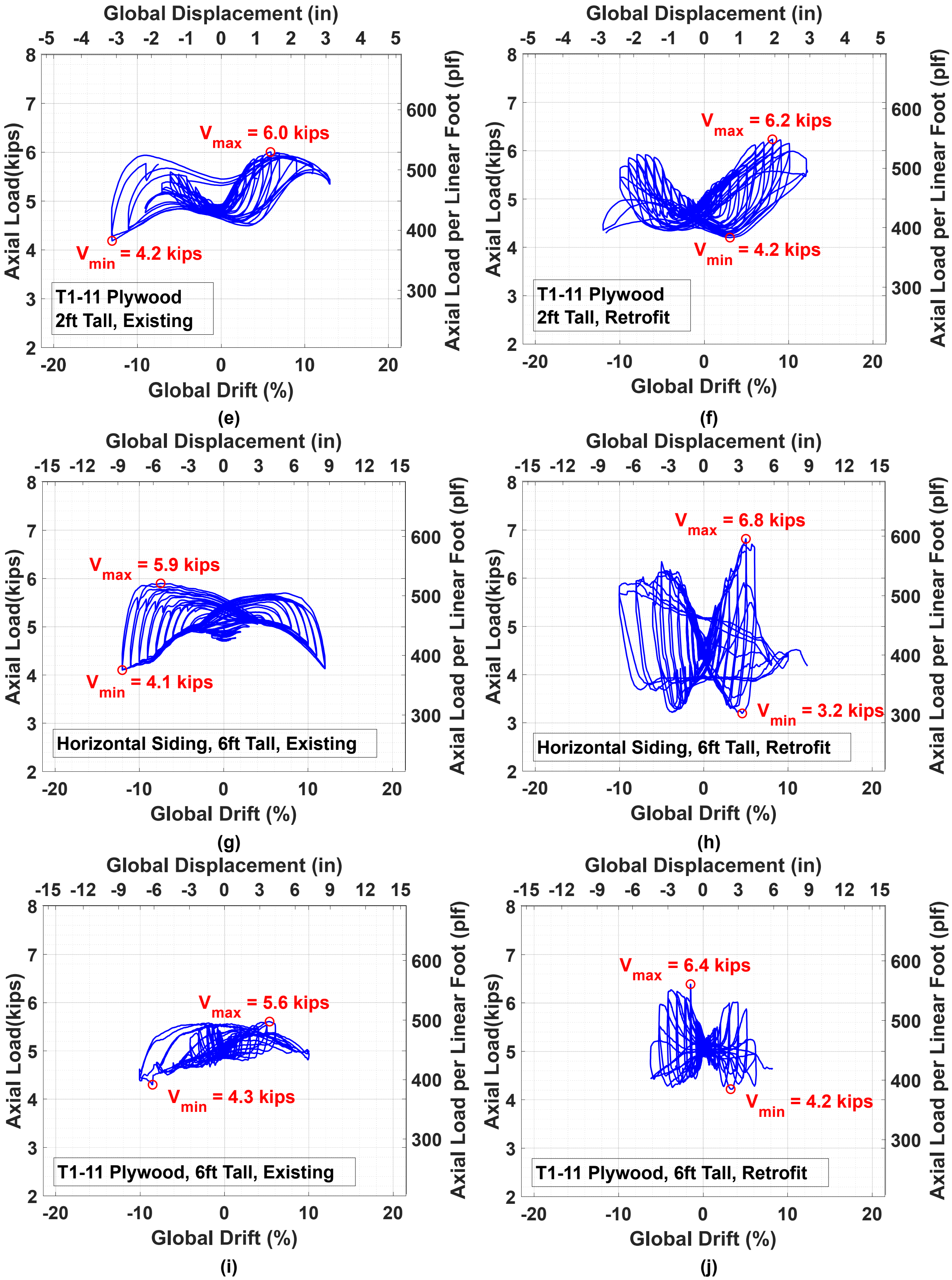


Figure 3.112 (continued).

3.12 HORIZONTAL SIDING STATIC ANALYSIS

As stated in Section 3.2, horizontal siding is the weakest exterior finish of those tested. The lateral strength per linear foot for the existing 2-ft-tall cripple wall with horizontal siding had an average of 174 plf between the push and pull directions, whereas the 6-ft-tall counterpart had an average of 97 plf between both directions of loading. Figure 3.113 shows an overlay of the hysteresis for these two cripple walls, while Figure 3.114 shows an overlay of their envelopes of their hysteretic response.

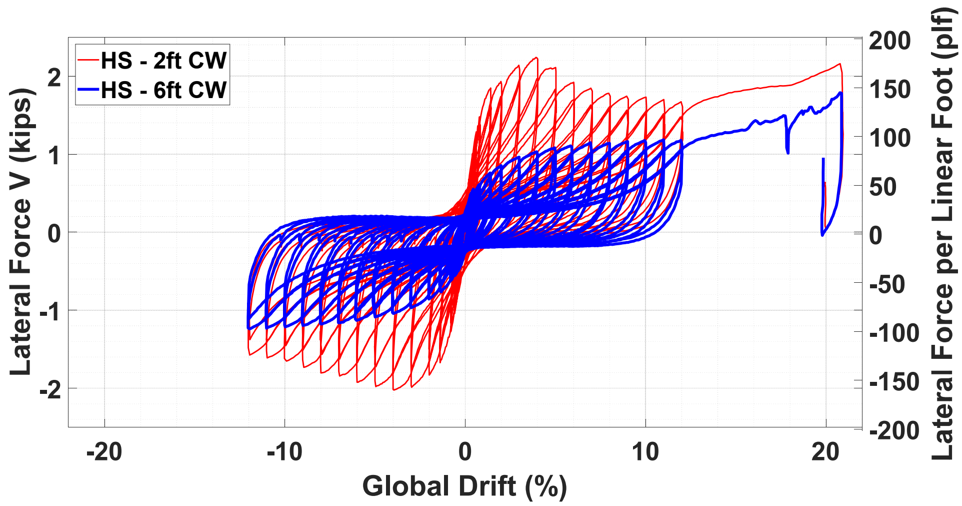


Figure 3.113 Specimens A-7 and A-13 lateral force versus *global* lateral drift and displacement hysteresis overlay.

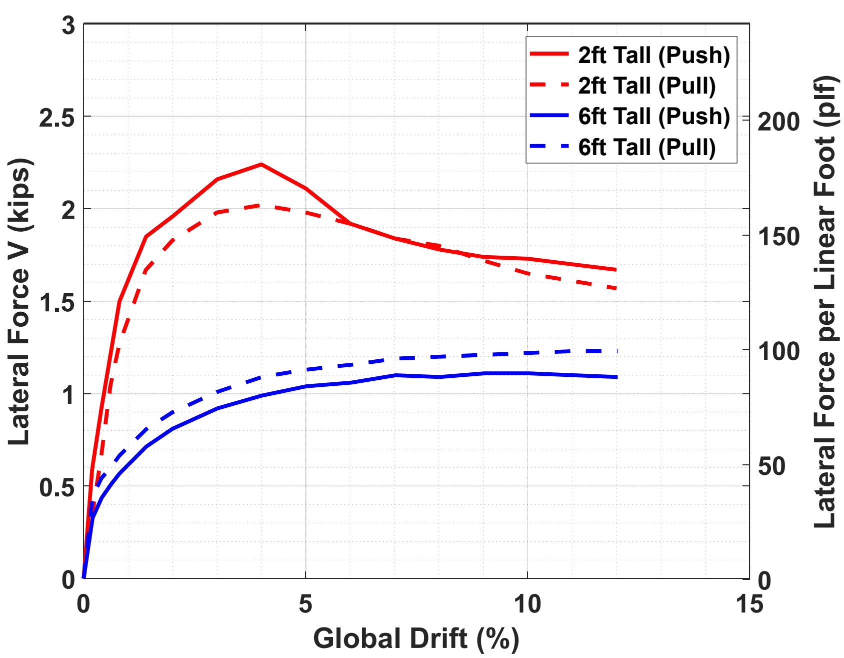


Figure 3.114 Specimens A-7 and A-13 lateral force versus *global* lateral drift and displacement envelope of hysteretic response overlay.

Horizontal siding gains most its strength from the resistance of the nail couples attaching the siding to the framing. Some resistance is also provided by the friction between the overlap of the shiplap boards. When the 6-ft-tall cripple wall is loaded, there is three times the moment imposed on the cripple wall compared with the 2-ft-tall cripple wall. There are three times as many siding boards on the taller wall leading to three times as many nail couples and nearly three times as many contact interfaces between the siding boards. Because of this, the lateral load capacity of the horizontal siding should be comparable for the 6-ft-tall cripple wall and the 2-ft-tall cripple wall. In the CUREE-Caltech Woodframe Project cripple wall tests at UC Davis [Chai et al. 2002], the peak strengths between the 2-ft-tall cripple walls and 4-ft-tall cripple walls were nearly identical. Table 3.10 shows the testing matrix for the UC Davis tests, and Table 3.11 shows the peak strength ratio for 2-ft-tall and 4-ft-tall cripple walls, which only varied in height.

Table 3.10 CUREE-Caltech Woodframe Project UC Davis level cripple wall testing matrix [Chai et al. 2002].

Specimen No.	Wall Height	Stucco	Bracing %	Gravity lbs/ft	Loading History
M	2’	No	100	450	Monotonic
1	2’	No	100	450	Normal
2	2’	Yes	100	450	Normal
3	2’	Yes	100	450	Near-fault
4	2’	No	66	100	Normal
5	2’	Yes	66	100	Normal
6	2’	Yes	66	100	Near-fault
7	4’	No	100	450	Normal
8	4’	Yes	100	450	Normal
9	4’	Yes	100	450	Near-fault
10	4’	No	66	100	Normal
11	4’	Yes	66	100	Normal
12	4’	Yes	66	100	Near-fault

Table 3.11 Ratio of 4-ft-tall to 2-ft-tall peak strengths of the CUREE UC Davis level cripple wall tests.

2-ft-tall specimen no.	$V_{\text{max, avg, 2 ft tall}}$ (kips)	2-ft-tall specimen no.	$V_{\text{max, avg, 4 ft tall}}$ (kips)	($V_{\text{max, avg, 4 ft tall}}$)/ ($V_{\text{max, avg, 2 ft tall}}$)
1	10.10	7	10.75	1.06
2	14.35	8	15.95	1.11
3	15.15	9	15.70	1.04
4	6.20	10	5.30	0.85
5	10.50	11	11.25	1.07
6	10.20	12	10.75	1.05
Average Ratio = 1.03				

A comparison of the 2-ft-tall and 6-ft-tall cripple wall show a 79% increase in lateral load capacity of the 2-ft-tall cripple wall compared with the 6-ft-tall cripple wall or a ratio of 1.79. Because the capacity of the cripple walls is so low for these two specimens (next lowest is the existing T1-11 cripple wall with 558 plf on average), much of the strength comes from the framing itself. 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. Through a static analysis of the moment capacities of the two cripple walls, it can be seen that the moment capacity ratios are close to the peak moment ratios.

Moment capacity of nail couples in siding

There are 40 nail couples for the 2-ft-tall cripple wall and 130 nail couples for the 6-ft-tall cripple wall. The average distance between the two nails in each couple is 2.75 in. Work done by Fonseca et al. [2002] as part of the CUREE-Caltech Woodframe Project have shown that the peak load-slip of an 8d common hot-dipped galvanized nail fastening a Douglas Fir – Larch stud and 3/8 in.-OSB panel is 185 lbf for a perpendicular wood grain connection [Fonseca et al. 2002]. This is used as a baseline for the capacity of each nail in the couple. A free body diagram of this can be seen in Figure 3.118.

$$2'CW: \quad 40\ couples * (185\ lbf * 2.75'') * \frac{1\ ft}{12\ in} * \frac{1\ kip}{1000\ lbf} = 1.696\ kip - ft$$

$$6'CW: \quad 130\ couples * (185\ lbf * 2.75'') * \frac{1\ ft}{12\ in} * \frac{1\ kip}{1000\ lbf} = 5.511\ kip - ft$$

$$M_{Nail\ Couples,2ft} = 1.696\ kip - ft$$

$$M_{Nail\ Couples,6ft} = 5.511\ kip - ft$$

Moment capacity of framing nails withdrawal strength

Nail withdrawal is a primarily a function of the fastener penetration, fastener diameter, moisture content of wood, and specific gravity of wood. From the USDA Review of End Grain Nail Withdrawal Research [Rammer and Zelinka 2004], an analysis of available literature was used to determine the withdrawal strength of the nails connecting the studs to the top plates and sill plate. The Marquardt-Levenberg [Marquardt 1963; Levenberg 1944] nonlinear curve fitting procedure was used to determine the withdrawal strength of nails in end-grain. The formula relies on a fitting parameter (*a*), penetration depth (*d*), and specific gravity of the wood (*g*) to give the withdrawal strength per depth of penetration (*W*). For Douglas Fir with a specific gravity of 0.5, a 16d common nail with a diameter of 0.161 in., and penetration depth of 2 in., the withdrawal strength is as follows:

$$W = adg^{3/2} \quad where\ a = 2531\ lbf/in^2$$

$$W = 2531 \frac{lbf}{in^2} * 0.161 in * 0.5^{3/2} * \frac{1\ kip}{1000\ lbf} = 0.145\ {}^{kip}/_{in}$$

$$Withdrawal\ strength\ per\ nail = 0.145\ {}^{kip}/_{in} * 2\ in. penetration = 0.290\ kip\ per\ nail$$

$$Withdrawal\ strength\ per\ stud = 0.290\ kip * 4\ nail\ per\ stud = 1.16\ kips$$

By taking the moment about the pivot points on the framing that are at each of the 8 interior studs and the two end-flat stud (shown in Figure 3.118), the moment capacity of the framing from the nail withdrawal strength is found as:

$$M_{Framing\ Nails} = 1.352\ kip - ft$$

This is the same value for both the 2-ft-tall and 6-ft-tall cripple wall as the framing details are identical except for the length of the studs.

Moment capacity of overturning resistance due to gravity

Finally, the applied vertical load resists overturning of the cripple wall. This resistance is the same for both the 2-ft-tall cripple wall and the 6-ft-tall cripple wall as the framing details were identical except for length of the stud; the applied vertical load was the same for the two cripple walls. By calculating the tributary area of the applied load, the amount of compressive load each interior stud and corner can be determined. The overturning moment resistance is taken by multiplying the compressive load by the distance from the center-of-mass of the studs/corner to the pivot points on which the interior studs and corners will rotate on when laterally displaced. Note that both the top and bottom of the studs and corners resist overturning. The compressive loads and distance to the pivot points can be seen in Figure 3.115.

$$M_{Overturning\ Resistance} = 0.824\ kip - ft$$

Peak moment applied to cripple walls

The peak moment applied to the cripple walls was taken as the peak strength of experienced during each test multiplied by the length of the studs. Since there are no relative displacements between where the load is applied (horizontal steel transfer beam), the lever arm is taken as the distance between the sill plate and the lower top plate, i.e., the length of the stud.

$$M_{Peak,2ft} = 19.5\ in * 2.24\ kips * \frac{1\ ft}{12\ in} = 4.656\ kip - ft$$

$$M_{Peak,6ft} = 67.5\ in * 1.23\ kips * \frac{1\ ft}{12\ in} = 6.92\ kip - ft$$

Ratio of peak moment to moment capacity

Table 3.12 shows a summation of the moment capacity due to the nail coupling on the horizontal siding boards, moment capacity due to the nail withdrawal strength of the framing, and the moment capacity of overturning resistance due to the gravity load for both of the cripple walls. From the static analysis, the ratio of moment capacities for the 6-ft-tall cripple wall and 2-ft-tall cripple wall is 2.01. From the peak moment experienced by the two cripple walls, the ratio is 1.89. This amounts to a 6.2% difference instead of a 79% difference where only the peak strengths were analyzed. Due to the variability intrinsic to wood construction, the results from this analysis show little difference between the static analysis and the test results.

Table 3.12 Analysis results for 2-ft-tall and 6-ft-tall, existing cripple walls with horizontal siding.

Static Analysis				
Height (ft)	$M_{\text{overturning resistance}}$ **(kip-ft)**	$M_{\text{framing nails}}$ **(kip-ft)**	$M_{\text{nail couples}}$ **(kip-ft)**	M_{capacity} **(kip-ft)**
2	1.256	0.824	1.696	3.776
6	1.256	0.824	5.511	7.591
$M_{\text{capacity,6 ft tall}}/ M_{\text{capacity,2 ft tall}} = 2.01$				

Test Data	
Height (ft)	M_{peak} **(kip-ft)**
2	3.656
6	6.920
$M_{\text{peak,6 ft tall}}/ M_{\text{peak,2 ft tall}} = 1.89$	
Percent difference = 6.2%	

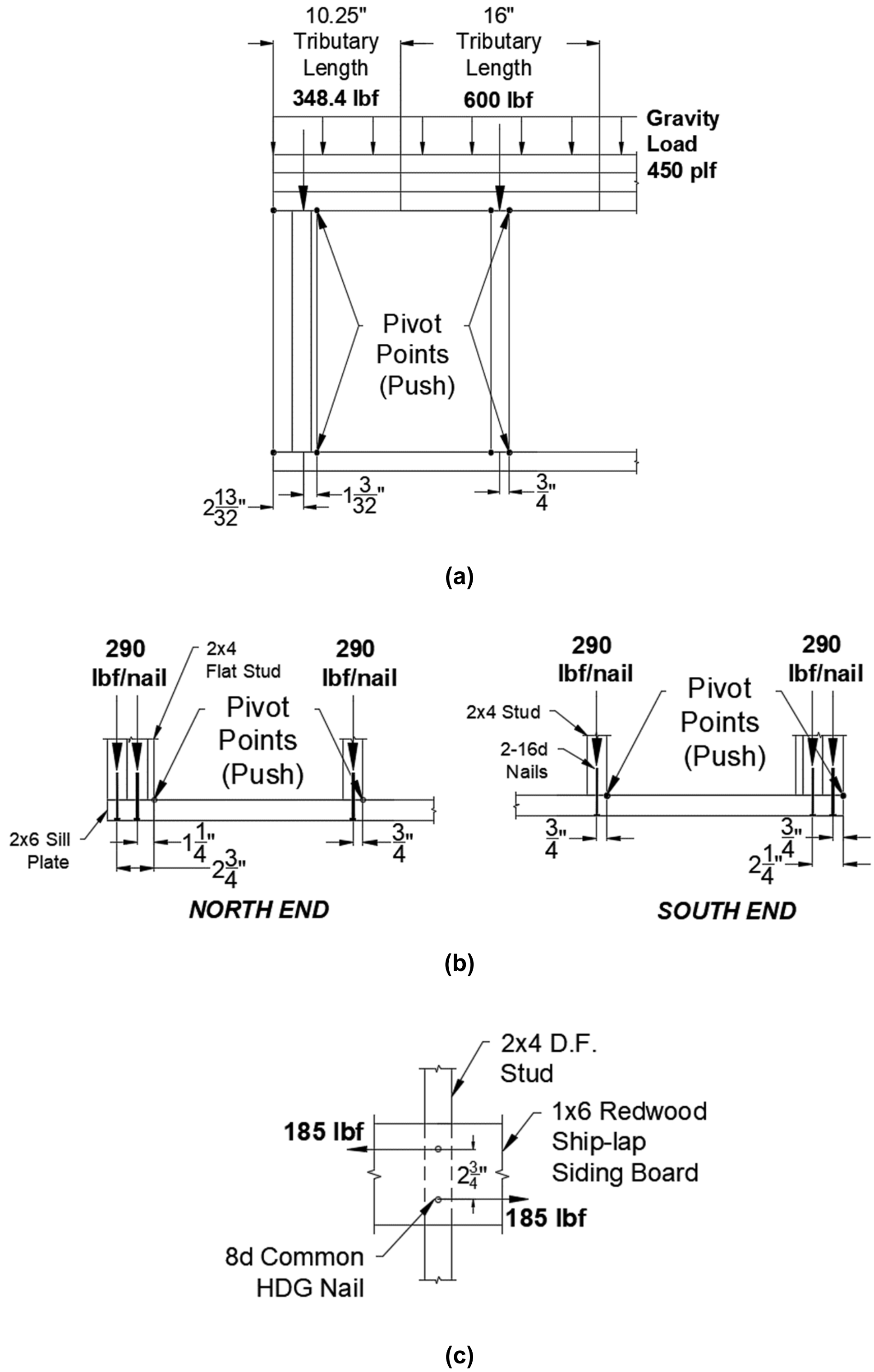


Figure 3.115 Force diagram for moment capacities for cripple walls with horizontal siding exterior finish: (a) overturning; (b) framing nailing; and (c) siding nailing.

4 Damage Characteristics of Cripple Walls

4.1 GENERAL

This chapter presents the damage characteristics and evolution of damage of the cripple walls at the different imposed lateral displacements. Tracking the physical damage of cripple walls is key to be able to make determinations about the structural integrity of a cripple wall after a seismic event. Of particular interest will be the finish and plywood nail withdrawal/rotation, plywood panel tearing/buckling, and rotation as well as uplift and splitting of framing members. In addition, a look at the fracture to anchor bolts causing failures to the cripple walls will be provided. For all drift ratio levels, photographs of damage were taken at the initial push and initial pull of each drift amplitude. In addition, from the 0.2% to the 1.4% drift ratio levels, photographs were taken at the end of the cycle grouping with the purpose of recording the state of damage at zero imposed lateral load as well as the residual displacement that accrued in the cripple walls. The ability to relate the physical damage of a cripple wall to the strength capacity of a cripple wall is key to determine what repairs are required to fix the structural and non-structural components of a cripple wall and the superstructure. This chapter will be broken into sections based on the damage to each of the six cripple walls.

4.2 DAMAGE CHARACTERISTICS FROM 0.0% TO 1.4% DRIFT RATIO LEVEL (SERVICE-LEVEL DRIFT)

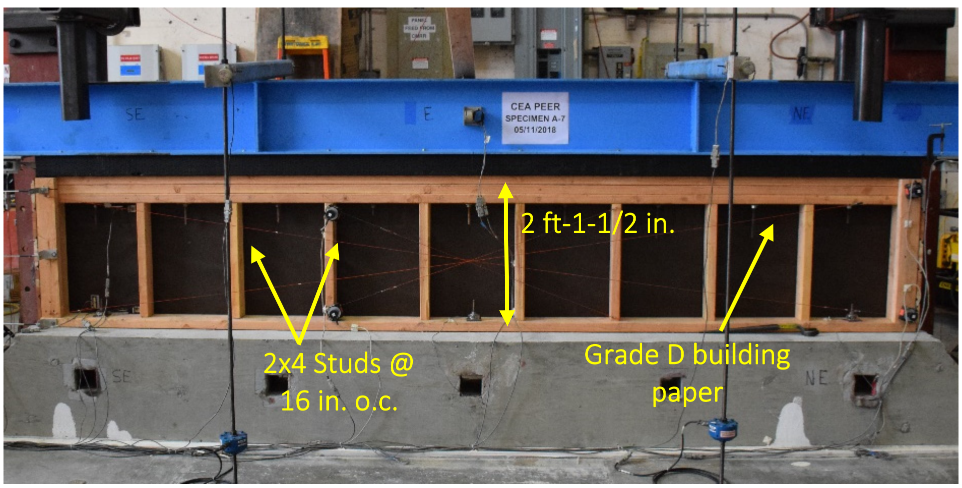
Understanding the physical damage characteristics of cripple walls at low-level drift amplitudes is important to be able to see how damages accrue when a dwelling undergoes small deformations either caused by a small-medium seismic event or a further away large seismic event. The state of each of cripple wall specimens at the start of the test are shown. Recall that the imposed drift ratio levels from the loading protocol are 0.2%, 0.4%, 0.6%, 0.8%, and 1.4%. During these drift ratio levels, photographs were taken at the initial push, the initial pull, and at the end of the displacement cycle. At the end of the displacement cycle, the cripple wall is in its residual state with no lateral forces acting upon it. Descriptions and images are provided for the 1.4% drift ratio level whether there was observable damage to the cripple wall or not.

4.2.1 Specimen A-7: 0.0% to 1.4% Drift Ratio Level

The photographs in Figure 4.1 show the initial state of Specimen A-7. Specimen A-7 is a 2-ft-tall cripple wall with horizontal siding exterior finish in an existing condition. The siding is 1 × 6 redwood fastened with two 8d common hot-dipped galvanized nails per stud. There was no notable pre-existing damage on the cripple wall prior to testing. The actual height of the cripple wall is 2 ft-1-1/2 in. due to the addition of a third top plate as part of top boundary condition B; the actual length of the cripple wall is 12 ft-4-1/2 in. due to the addition of built-up corners as part of bottom boundary condition c. Details of the siding nailing and corner are given in Figure 4.1(c) and (d), respectively.

From the 0.0% drift ratio level to the 0.6% drift ratio level, no visible damage was present on the cripple wall. The only notable change was the formation of wrinkles on the building paper. From 0.6% to 1.4% drift, there was again no noticeable damage to the specimen. Figure 4.2 shows the state of the cripple wall at -1.4% drift. The only significant change to the wall was more profound wrinkling in the building paper. The wrinkles originated from the studs where the building paper was stapled. Figures 4.2 (e) and (f) show the displacement of the siding relative to the foundation and the top plate, respectively. At this displacement level, there were no significant displacements at these two interfaces. Lastly, as shown in Figure 4.2(a), a stepping pattern is beginning to show between the siding boards. This shows that the imposed displacement on the cripple wall was being carried by the cripple wall itself, i.e., sliding of the sill plate on the foundation did not occur.

(a)



(b)

Figure 4.1 **Specimen A-7 pre-test photographs of the existing 2-ft-tall cripple wall finished with horizontal siding: (a) exterior elevation; (b) interior elevation; (c) top of north exterior of wall corner view; and (d) north-end of exterior of wall view.**

(c) (d)

Figure 4.1 (continued).

(a) (b)

Figure 4.2 Cripple wall details at -1.4% drift ratio level @ Δ = -0.336 in.: (a) middle exterior of wall view; (b) north interior of wall corner view; (c) south interior of wall corner view; (d) south exterior of wall corner view; (e) bottom of wall exterior view; and (f) top exterior of wall view.

(c)

(d)

(e)

(f)

Figure 4.2 (continued).

4.2.2 Specimen A-8: 0.0% to 1.4% Drift Ratio Level

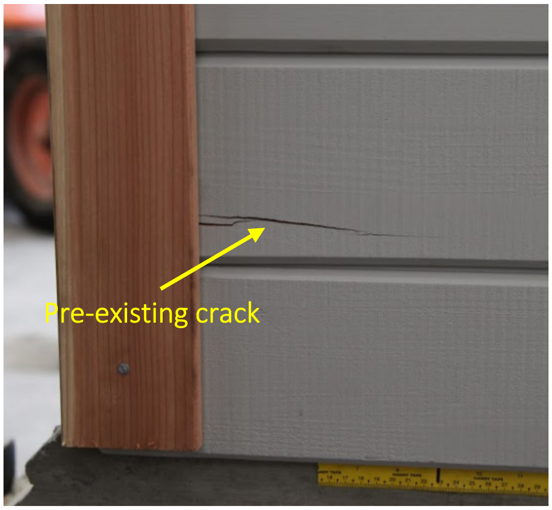
The photographs in Figure 4.3 show the initial state of Specimen A-8 prior to testing. Specimen A-8 is a retrofitted 2-ft-tall cripple wall with horizontal siding exterior finish. The siding is 1 × 6 redwood fastened with two 8d common hot-dipped galvanized nails per stud. The retrofit included the addition of three sections of 15/32-in.-thick plywood fastened with 8d common hot-dipped galvanized nails spaced at 3 in. on center around the edge of the panels and 12 in. on center in the field of the panels. A 1/8-in. gap is present between the plywood panels. In addition, two extra anchor bolts were added to reduce the anchor bolt spacing from 64 in. on center to 32 in. on center. The dimensions of the cripple wall are identical to Specimen A-8. In Figures 4.3 (c) and (d), pre-existing cracks in the siding can be seen. These cracks originated from over-driven nails at the two locations.

Details of the damage characteristics at -1.4% drift are shown in Figure 4.4. At the top of the wall the top siding board has displaced 1/8 in. relative to the framing [Figure 4.4(a)], and at bottom of the wall the displacement of the bottom siding board is 1/8 in. relative to the foundation. Panel rotation and nail withdrawal has become visible; see Figure 4.4 (c) and (d). At this point, the displacements between the siding boards have begun to increase. A red line was added down a row of nails on the siding to better track these displacements as shown in Figure 4.4(e).

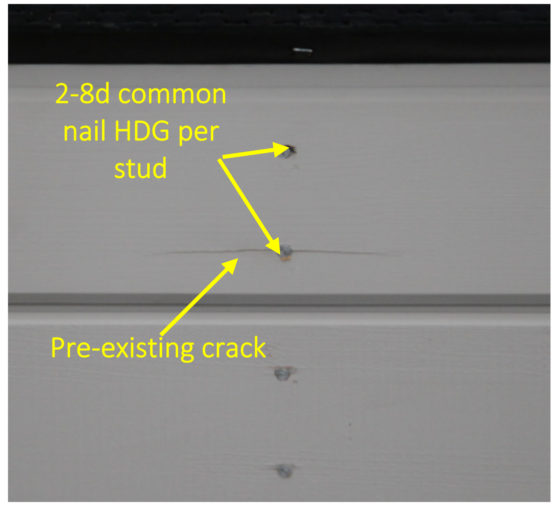
(a)

(b)

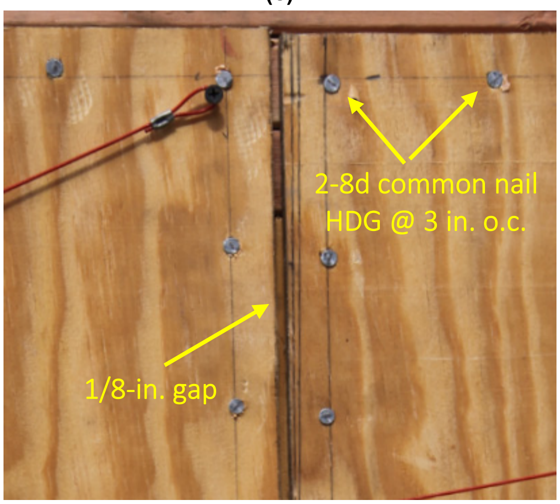
Figure 4.3 **Specimen A-8 pre-test photographs of the retrofitted 2-ft-tall cripple wall finished with horizontal siding: (a) exterior elevation; (b) interior elevation; (c) bottom of north exterior of wall corner view; (d) top middle exterior of wall view; (e) top middle interior of wall view; and (f) top of interior of wall south corner view.**



(c)



(d)



(e)

(f)

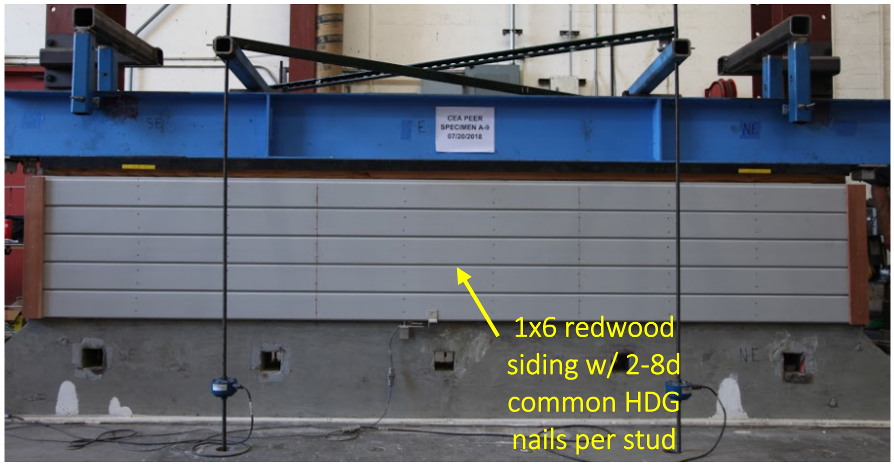
Figure 4.3 (continued).

Figure 4.4 **Specimen A-8 damage state details at -1.4% drift ratio level @ Δ = -0.336 in.: (a) top middle of exterior wall view; (b) bottom middle of exterior wall view; (c) bottom of north interior corner view; (d) bottom interior of wall (middle and south panels); (e) middle exterior of wall view; and (f) north-end interior of wall view.**

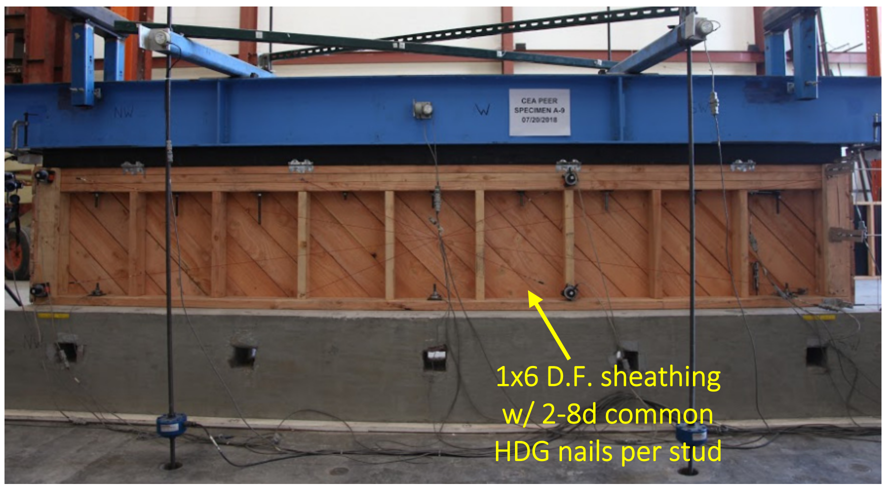
4.2.3 Specimen A-9: 0.0% to 1.4% Drift Ratio Level

The photographs in Figure 4.5 show the initial state of Specimen A-9 prior to testing. Specimen A-9 is an existing 2-ft-tall cripple wall with horizontal siding over diagonal sheathing exterior finish. The siding is 1 × 6 redwood fastened with two 8d common hot-dipped galvanized nails per stud. The diagonal sheathing boards 1 × 6 construction grade Douglas Fir were oriented at a 45° angle from southwest to northeast; see Figure 4.5(c). The dimensions of the cripple wall were the same as Specimen A-7 and Specimen A-8. Figure 4.5 (d), (e), and (f) show pre-existing cracks in the corner siding as well as the sheathing boards. These cracks occurred during construction. The cracks in the sheathing boards originated when the siding boards were being installed. The cracks in the corner siding board originated from when the corner trim was being installed.

No visible changes or damages occurred to the specimen from 0.0% to 1.4% drift. Figure 4.6 shows details of the cripple wall at -1.4% drift. No visible damage was apparent on the specimen besides a small rotation of the studs [Figure 4.12(a)], and slight nail withdrawal on the siding [Figure 4.6(e)] and the corner trim [Figure 4.6(d)].

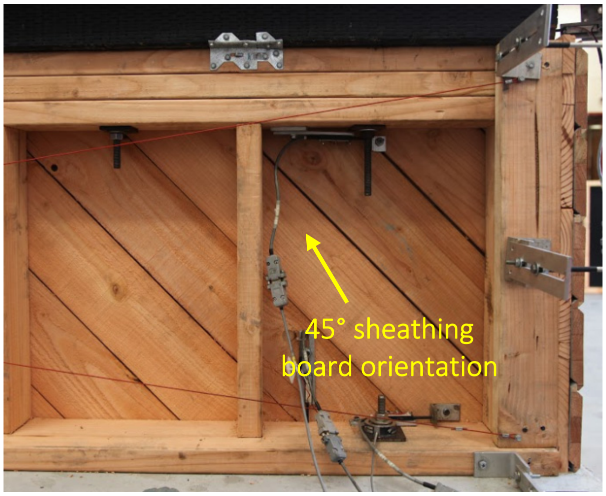


(a)

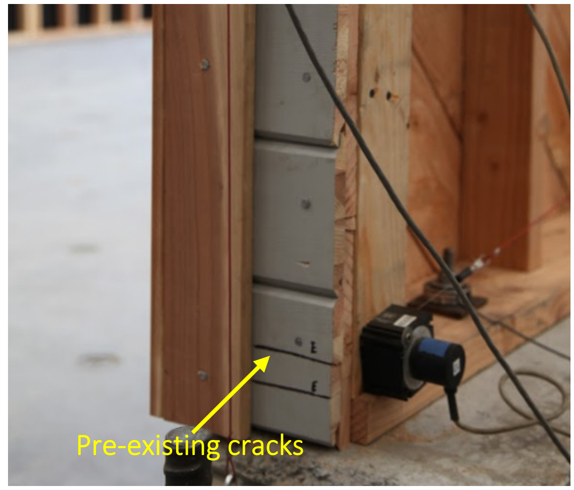


(b)

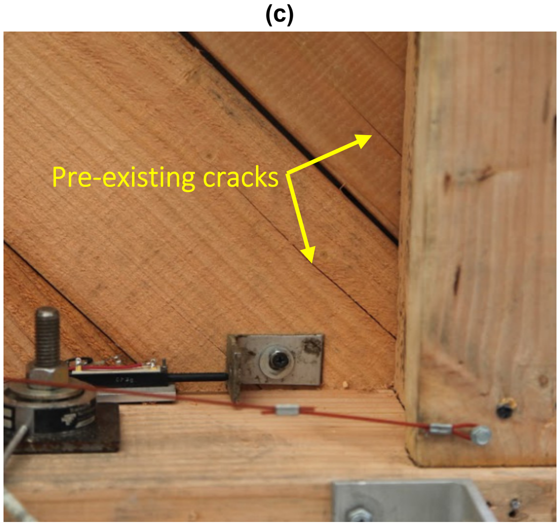
Figure 4.5 **Specimen A-9 pre-test photographs of the existing 2-ft-tall cripple wall finished with horizontal siding over diagonal sheathing, heavy vertical load: (a) exterior elevation; (b) interior elevation; (c) south-end interior of wall corner view; (d) bottom of north-end interior of wall view; (e) bottom of south-end interior of wall corner view; and (f) bottom of middle interior of wall view.**



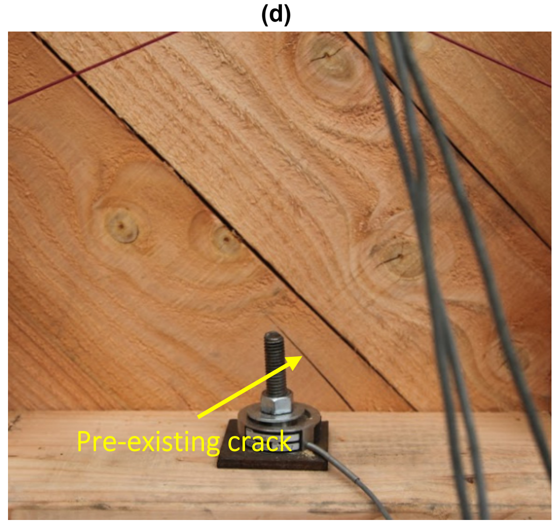
(c)



(d)



(e)



(f)

Figure 4.5 (continued).

Figure 4.6 **Specimen A-9 damage state details at -1.4% drift ratio level @ Δ = -0.336 in.: (a) bottom interior of wall view; (b) south-end interior of wall view; (c) middle of wall corner view; (d) top of north-end exterior of wall corner view; (e) top middle exterior of wall view; and (f) south-end exterior of wall view.**

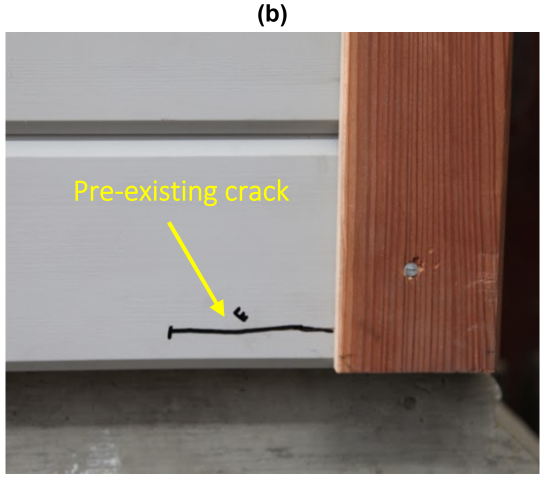
4.2.4 Specimen A-10: 0.0% to 1.4% Drift Ratio Level

The photographs in Figure 4.7 show the initial state of Specimen A-9 prior to testing. Specimen A-9 is a retrofitted 2-ft-tall cripple wall with horizontal siding over diagonal sheathing exterior finish. The siding is 1 × 6 redwood fastened with two 8d common hot-dipped galvanized nails per stud. The diagonal sheathing boards 1 × 6 construction grade Douglas Fir were oriented at a 45° angle from southwest to northeast. The retrofit included the addition of three sections of 15/32-in.-thick plywood fastened with 8d common hot-dipped galvanized nails spaced at 3 in. on center around the edge of the panels and 12 in. on center in the field of the panels. A 1/8 in. gap is present between the plywood panels. In addition, four extra anchor bolts were added. Two anchor bolts were placed in the existing slots of the foundation, reducing the anchor bolt spacing from 64 in. on center to 32 in. on center, and two additional anchor bolts were epoxied 12 in. in from the outermost anchor bolts. The dimensions of the cripple wall are identical to Specimen A-9. Figure 4.7(c) shows a pre-existing crack on the siding board that was present before the installation of the siding.

Figure 4.8 shows details of the damage at -1.4% drift. shows the interior and exterior faces of the cripple wall at the 1.4% drift ratio level. As with Specimen A-9, there were not many changes to state of the cripple wall from the start of the test. The only noticeable differences were a slight uplift of the corners and the start of the plywood panel crushing against the corner flat stud [Figure 4.8(d)] as well as a 1/4-in. displacement of the bottom siding board relative to the foundation; see Figure 4.8(a). This shows that the cripple wall had begun to slide on the foundation. Rotations in the plywood panels [Figure 4.8(c)] were not as significant in Specimen A-10 as they were in Specimen A-8 at -1.4% drift.

(a)

(b)

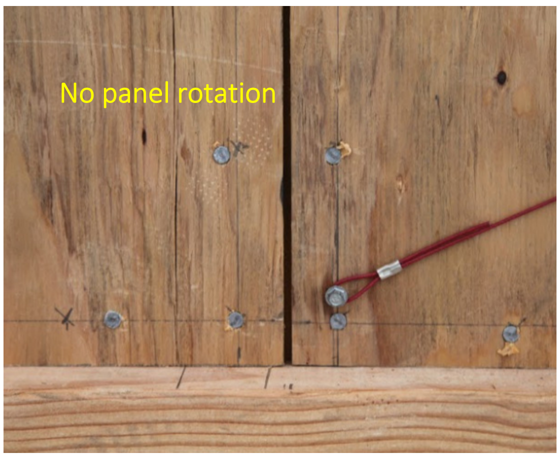


(c)

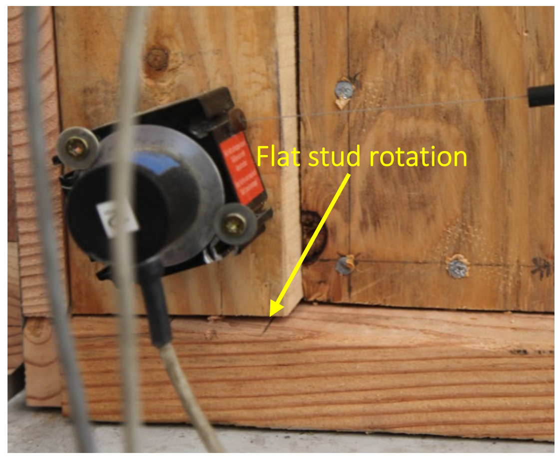
Figure 4.7 **Specimen A-10 pre-test photographs of the retrofitted 2-ft-tall cripple wall finished with horizontal siding over diagonal sheathing: (a) exterior elevation; (b) interior elevation; and (c) bottom of south-end exterior of wall corner view.**

(a)

(b)

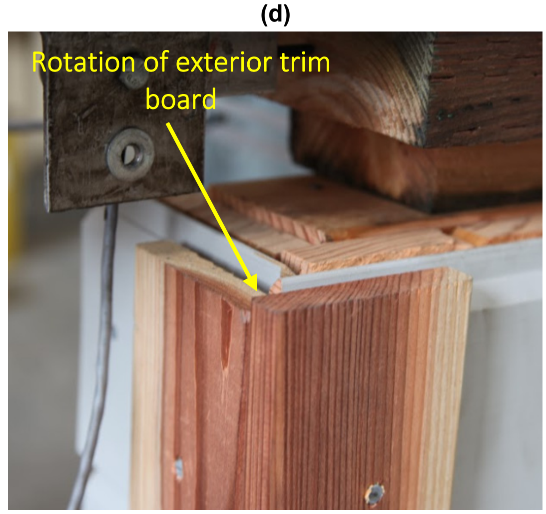


(c)



(d)

(e)



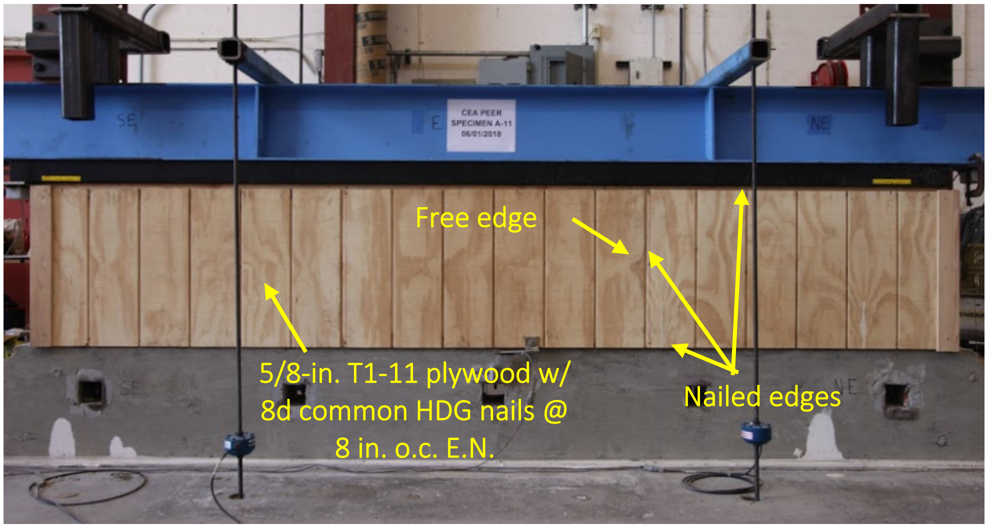
(f)

Figure 4.8 **Specimen A-10 damage state details at -1.4% drift ratio level @ Δ = -0.336 in.: (a) bottom middle exterior of wall view; (b) top middle exterior of wall view; (c) bottom middle interior of wall view (north and middle panels); (d) bottom of north-end interior of wall corner view; (e) middle exterior of wall view; and (f) top of south-end exterior of wall corner view.**

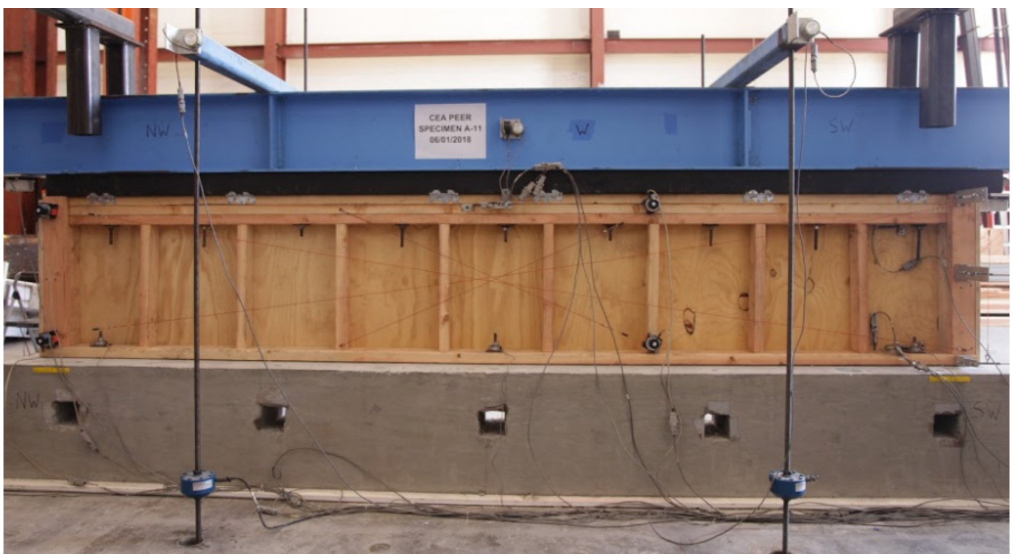
4.2.5 Specimen A-11: 0.0% to 1.4% Drift Ratio Level

The photographs in Figure 4.9 show the initial state of Specimen A-11 prior to testing. Specimen A-11 is an existing 2-ft-tall cripple wall with a 5/8-in.-thick T1-11 plywood exterior finish. Recall that the T1-11 panels were fastened on three edges and relied on the overlapping -panel to hold the underlying T1-11 panel in place. An 1/8 in. gap is left between panels. The height of the cripple wall is the same as the previous specimens, but the length of the cripple wall is 12 ft instead of 12 ft-4-1/2 in. The length was shortened in order to accommodate three 4-ft sections of T1-11 plywood. The T1-11 panels came in 4-ft × 8-ft sections, so no partial widths of the panels were used. The panels were fastened with 8d common hot-dipped galvanized nails at 8 in. on center around the edges of the panels and 12 in. on center throughout the field of the panels. Figure 4.9(d) shows pre-existing damage to the corner of one the T1-11 panels where a nail was overdriven. Figure 4.9(e) shows a pre-existing crack in the sill plate at the south end of the wall.

Details of the damage at -1.4% drift are shown in Figure 4.10. Panel rotation at the top of the wall is visible in Figure 4.10(a). Figure 4.10(a) and (b) show the slip of the underlying T1-11 panels at the top of both overlap locations. Since the underlying panel was not fastened, it began to slip from the overlying panel at low drift amplitudes. In Figure 4.10(c), the corner trim on the top of the exterior face had displaced around a 1/4 in. from the corner trim on the corner face. This same displacement is not evident at the bottom of the corners; see Figure 4.10(d).

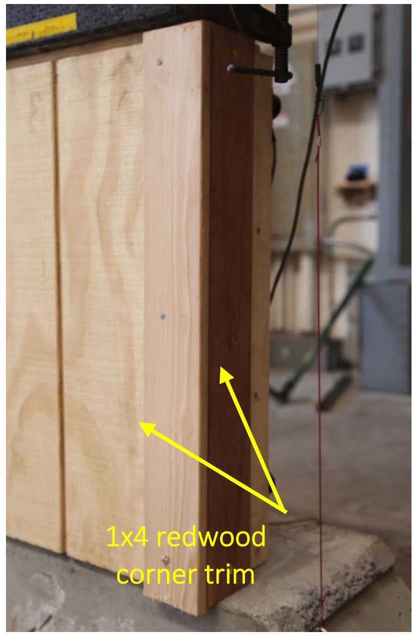


(a)



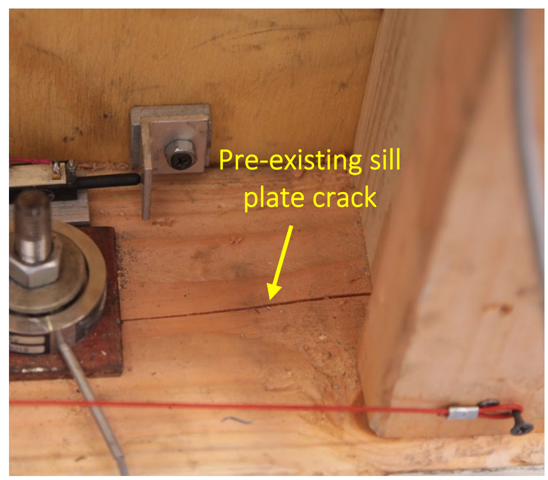
(b)

Figure 4.9 **Specimen A-11 pre-test photographs of the existing 2-ft-tall cripple wall finished with T1-11 wood structural panels): (a) exterior elevation; (b) interior elevation; (c) north-end exterior of wall corner view; (d) top middle exterior of wall view (middle and north panels); and (e) bottom of south-end interior of wall view.**



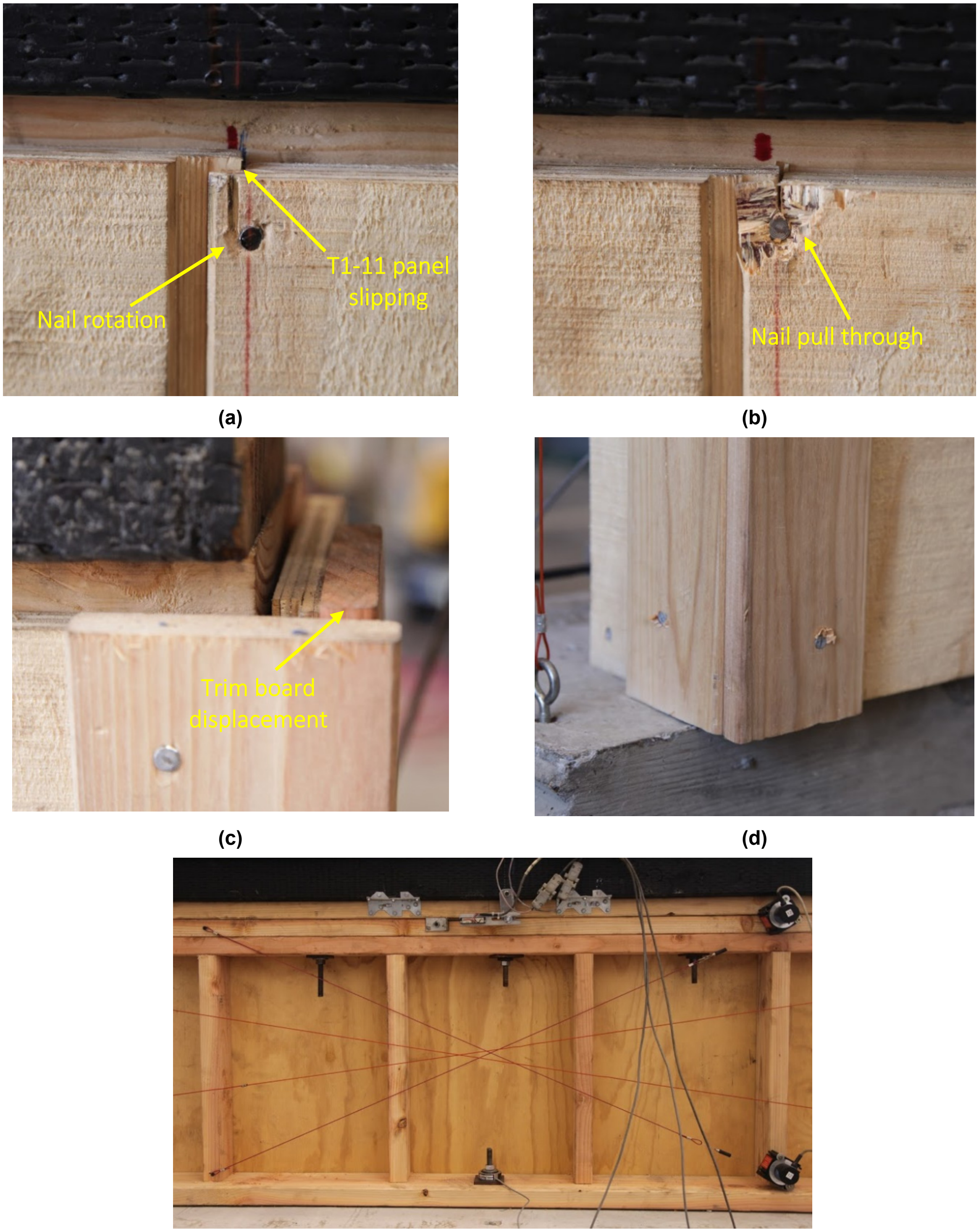
(c)

(d)



(e)

Figure 4.9 (continued).



(a) (b) (c) (d) (e)

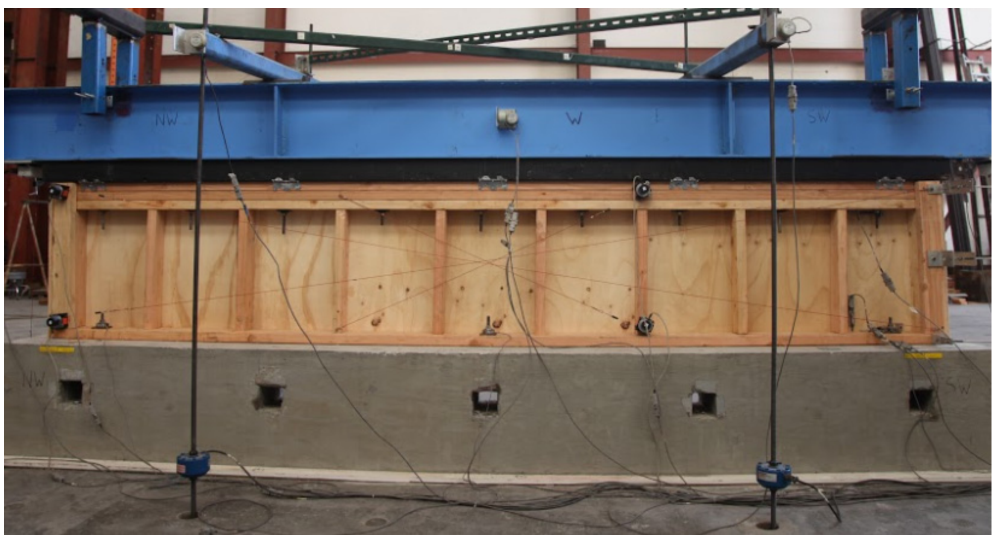
Figure 4.10 **Specimen A-11 damage state details at +1.4% drift ratio level @ Δ = +0.336 in. unless otherwise noted: (a) top middle interior of wall view (south and middle panels); (b) top middle interior of wall view (north and middle panels); (c) top of north exterior of wall corner view at -1.4% drift; (d) bottom of south-end exterior of wall corner view; and (e) middle interior of wall view.**

4.2.6 Specimen A-12: 0.0% to 1.4% Drift Ratio Level

The photographs in Figure 4.11 show the initial state of Specimen A-12 prior to testing. Specimen A-12 is a retrofitted 2-ft-tall cripple wall with 5/8-in.-thick T1-11 plywood exterior finish. Recall that the retrofit for T1-11 cripple walls requires that the T1-11 panels be fastened on all edges. This requires an extra row of nails as shown in Figure 4.11(c). In addition, nail spacing was reduced from 8 in. on center around the edges to 4 in. on center around the edges. The panels were fastened with 8d common hot-dipped galvanized nails at 4 in. on center around the edges of the panels and 12 in. on center throughout the field of the panels. An 1/8-in. gap was left between panels. Unlike all other retrofitted specimens, no additional anchor bolts were added to this cripple wall. The reasoning for this is discussed in Section 3.5. The dimensions of the cripple wall are the same as the existing counterpart, Specimen A-11. Figure 4.11(d) shows a pre-existing crack in the sill plate at the south end of the cripple wall.

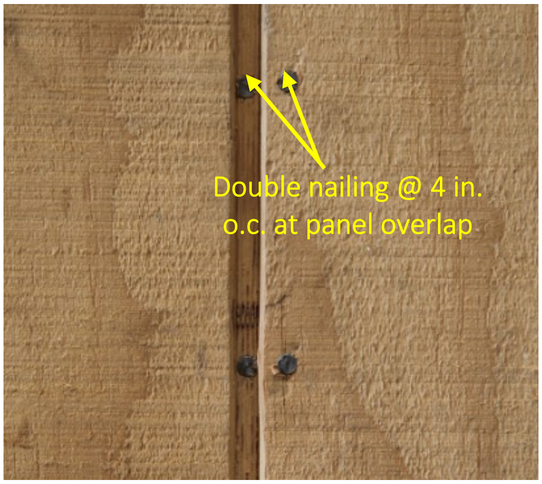
Details of damage at -1.4% drift are shown in Figure 4.12. At this point, rotation of the T1-11 panels had begun to accrue, but it was much less significant than the existing specimen; see Figure 4.12(c). This can be attributed to the addition of the added nailing at the panel joints. In the groves of the T1-11 panels, the nails have begun to pull through and outside of the grooves, the nails have begun to rotate; see Figure 4.12(a). Since the thickness of the panel is decreased in the grooves, the nails are prone to pull through the panel and stay attached to the framing. Lastly, a small crack had formed at the bottom of the south end, flat stud; see Figure 4.12(e).

(a)

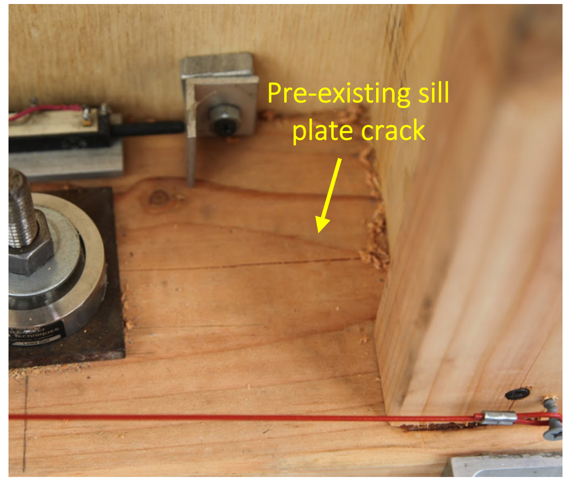


(b)

Figure 4.11 Specimen A-12 pre-test photographs of the retrofitted 2-ft-tall cripple wall finished with T1-11 wood structural panels: (a) exterior elevation; (b) interior elevation; (c) top middle exterior of wall view (middle and north panels); and (d) bottom of south-end interior of wall view.

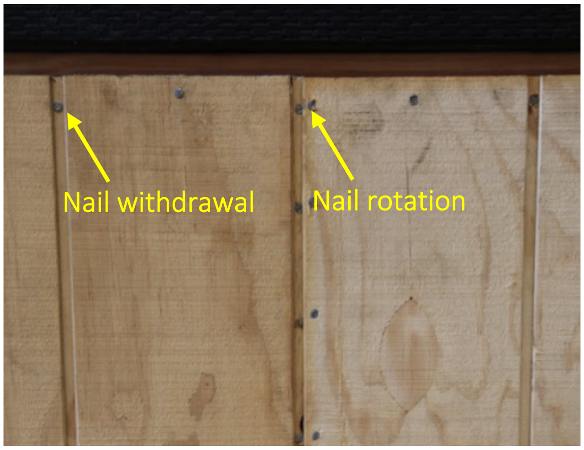


(c)



(d)

Figure 4.11 (continued).



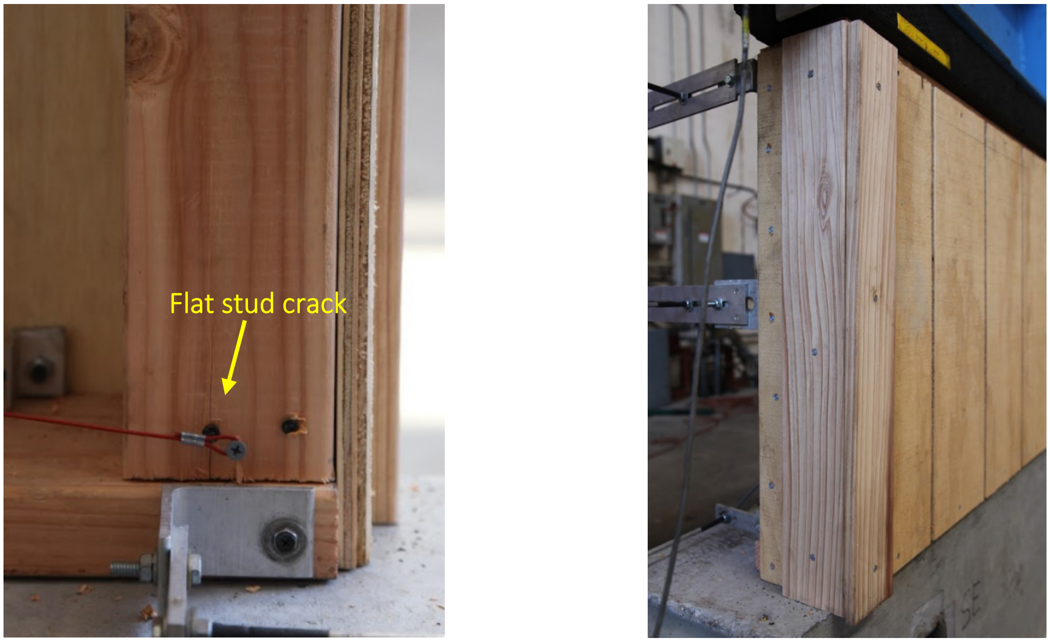
(a)

(b)

(c)

(d)

Figure 4.12 **Specimen A-12 damage state detail at +1.4% drift ratio level @ Δ = +0.336 in.: (a) top middle interior of wall view (north and middle panels); (b) top south-end exterior of wall view; (c) top middle interior of wall view (south and middle panels); (d) bottom of north-end exterior of wall corner view; (e) bottom of south-end interior of wall corner view; and (f) south-end exterior of wall corner view.**



(e) (f)

Figure 4.12 (continued).

4.2.7 Specimen A-13: 0.0% to 1.4% Drift Ratio Level

The photographs in Figure 4.13 show the initial state of Specimen A-13. Specimen A-13 is an existing 6-ft-tall cripple wall with horizontal siding exterior finish. The siding is 1 × 6 redwood fastened with two 8d common hot-dipped galvanized nails per stud. There was no notable pre-existing damage on the cripple wall prior to testing. The actual height of the cripple wall is 6 ft-1-1/2 in., and the length of the cripple wall is the same as Specimen A-7, the 2-ft-tall counterpart.

From the start of the test to the 0.8% drift ratio level, a cracked formed in a corner siding board at the top of the north corner of the cripple wall. In addition, uplifting of the corner studs at the north corner of the specimen became apparent at 0.8% drift. Figure 4.14 shows close-up images at some key locations on the specimen at-1.4% drift. In Figure 4.14(a), the rotations of the studs are shown. At this point, displacements between siding boards have started to become visible; see Figure 4.14(c).

(a)

(b)

Figure 4.13 Specimen A-13 pre-test photographs of the existing 6-ft-tall cripple wall finished with horizontal siding: (a) exterior elevation; and (b) interior elevation.

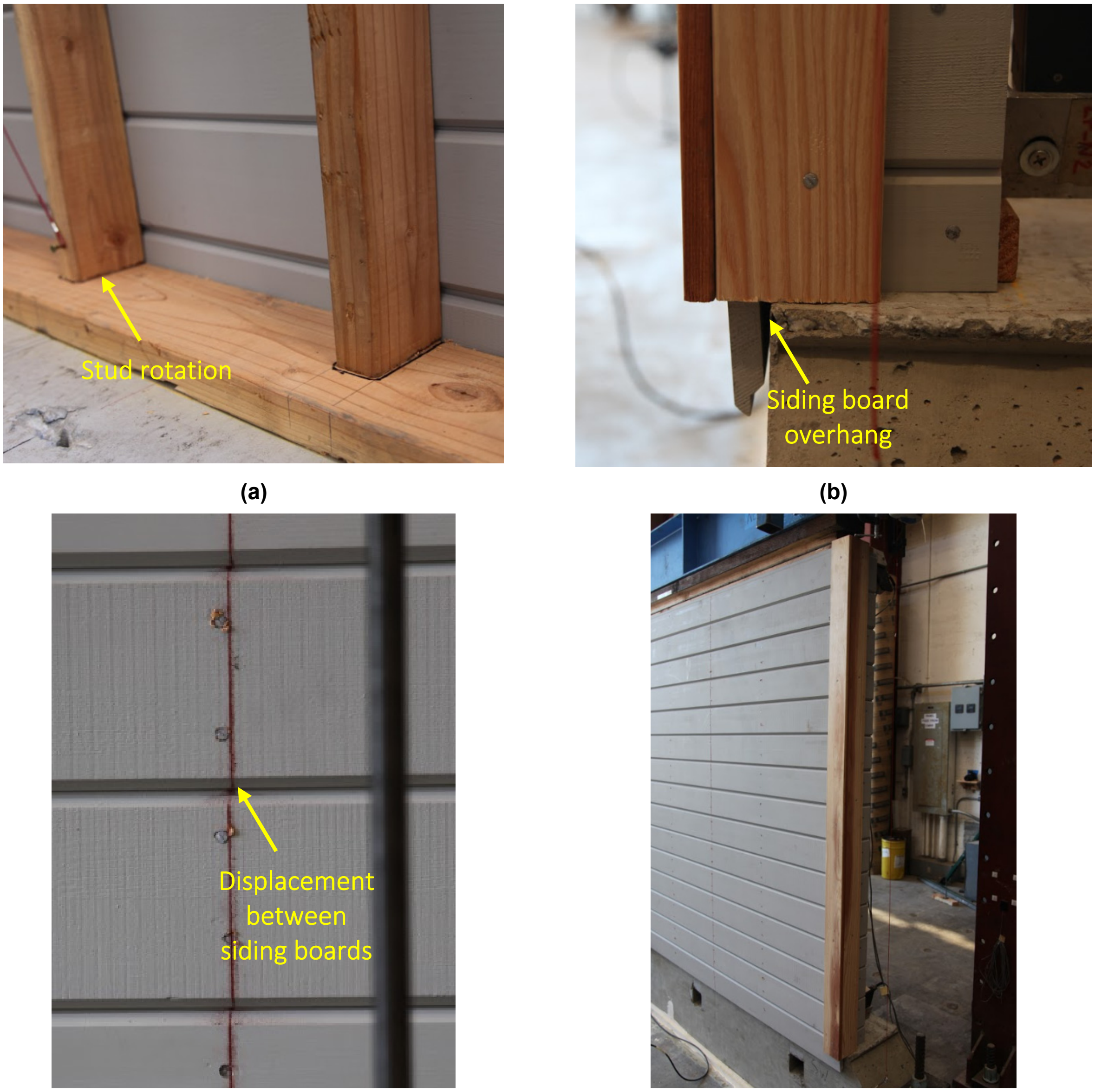


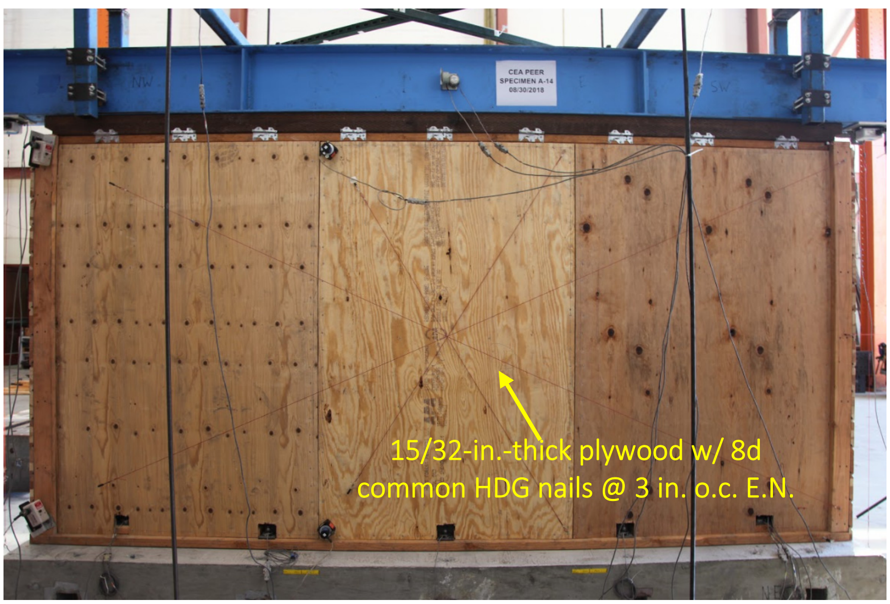
Figure 4.14 **Details of damage for Specimen A-13 at +1.4% drift ratio level @ Δ = +1.008 in.: (a) bottom middle interior of wall view; (b) bottom north-end of wall corner view; (c) middle exterior of wall view; and (d) north-end exterior of wall corner view.**

4.2.8 Specimen A-14: 0.0% to 1.4% Drift Ratio Level

The photographs in Figure 4.15 show the initial state of Specimen A-14 prior to testing. Specimen A-14 is a retrofitted 6-ft-tall cripple wall with horizontal siding exterior finish. The siding is 1 × 6 redwood fastened with two 8d common hot-dipped galvanized nails per stud. The retrofit included the addition of three sections of 15/32 in.-thick plywood fastened with 8d common hot-dipped galvanized nails spaced at 3 in. on center around the edge of the panels and 12 in. on center in the field of the panels. A 1/8-in. gap was present between the plywood panels. In addition, four extra anchor bolts were added. Two anchor bolts were placed in at the corners of the cripple wall. These anchor bolts were epoxied into place and were part of the hold-downs used to resist uplift of the cripple wall. Two additional anchor bolts were epoxied 12 in. in from the location of the hold-downs. The dimensions of the cripple wall were identical to Specimen A-13. Figure 4.15(c), (d), and (e) shows pre-existing cracks in the bottom corner siding board, bottom corner trim board, and top corner trim board, respectively. These cracks formed during the installation of the siding and trim boards.

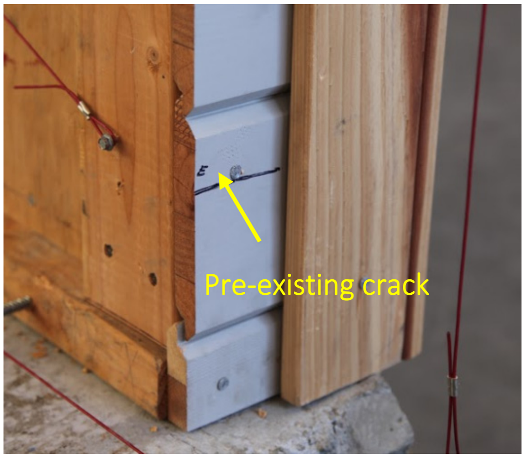
Figure 4.16 shows close-up images of the state of the specimen at -1.4% drift. The most notable damage is a split in the blocking just north of the middle anchor bolt; see Figure 4.16(b). From this photograph and Figure 4.16(a), uplift and rotation of the plywood panels is apparent. Sliding of the sill plate over the foundation had also begun, shown by the 1/4-in. displacement between the bottom siding plate and foundation in Figure 4.16(c). The nails had begun to withdrawal at some locations and show incipient pull through at other locations.

(a)



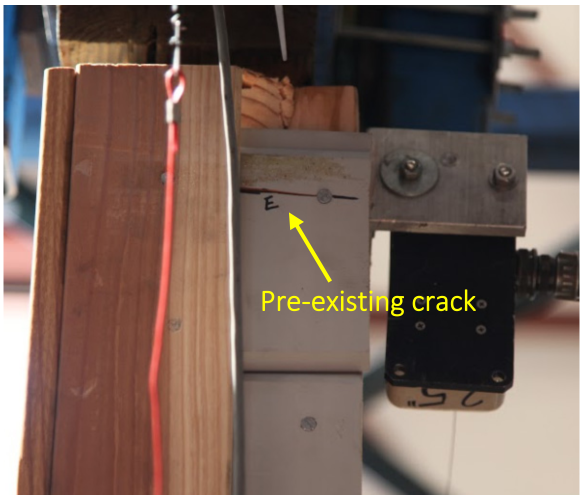
(b)

Figure 4.15 Specimen A-14 pre-test photographs for the retrofitted 6-ft-tall cripple wall finished with horizontal siding: (a) exterior elevation; (b) interior elevation; (c) bottom of south-end interior of wall corner view; (d) bottom of north-end of wall corner view; and (e) top of north-end of wall corner view.



(c)

(d)



(e)

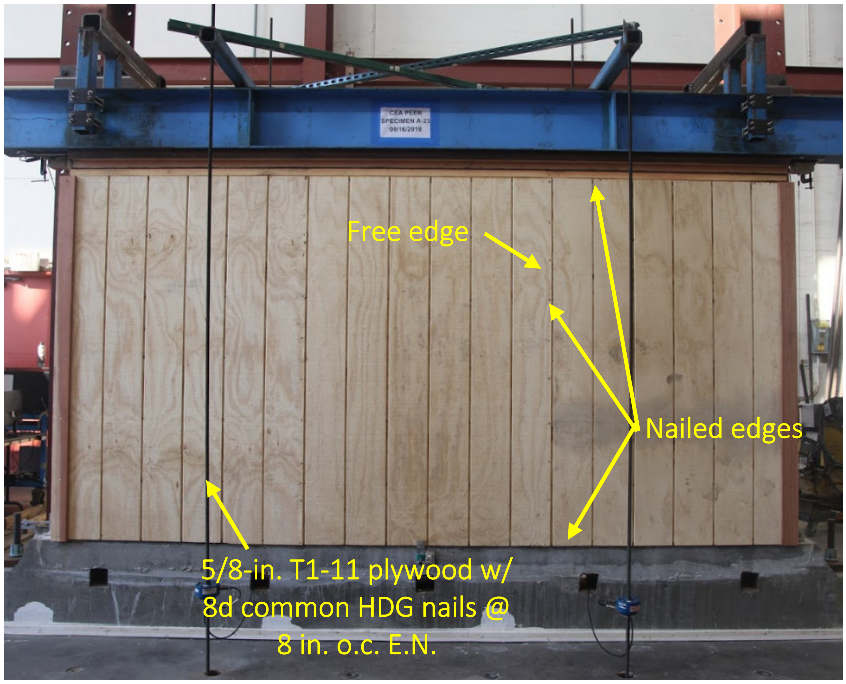
Figure 4.15 (continued).

Figure 4.16 Specimen A-14 damage state details of damage at -1.4% drift ratio level @ Δ = -1.008 in.: (a) bottom middle interior of wall view (north and middle panels); (b) bottom of north-end interior of wall view; (c) bottom middle of exterior of wall view; (d) bottom of south-end exterior of wall corner view; (e) top of south-end interior of wall corner view; and (f) middle exterior of wall view.

4.2.9 Specimen A-23: 0.0% to 1.4% Drift Ratio Level

The photographs in Figure 4.17 show the initial state of Specimen A-23 prior to testing. Specimen A-23 is an existing 6-ft-tall cripple wall with 5/8-in.-thick T1-11 wood structural panels. Recall that the T1-11 panels were fastened on three edges and relied on the overlapping T1-11 panel to hold the underlying T1-11 panel in place. A 1/8-in. gap was left between panels. The height of the cripple wall was the same as the previous 6-ft-tall specimens (6 ft-1-1/2 in.), but the length of the cripple wall was 12 ft instead of 12 ft-4-1/2 in. The length was shortened in order to accommodate three 4-ft sections of T1-11 plywood. The T1-11 panels came in 4-ft × 8-ft sections so no partial widths of the panels were used. The panels were fastened with 8d common hot-dipped galvanized nails at 8 in. on center around the edges of the panels and 12 in. on center throughout the field of the panels. Figure 4.17(d) shows pre-existing cracks at the top of the south end flat stud. These cracks occurred during framing of the cripple wall. The largest crack does not contain a fastener (removed), instead a fastener was driven 1/2 in. from the crack location.

At the 0.8% drift ratio level, the T1-11 panels show visible rotation, as the underlying panel had little resistance to rotate due to the lack of nailing on the panel edge. Figure 4.18 show details of the damage at -1.4% drift. Along with panel rotation, the nails connecting the panels show visible rotation; see Figure 4.18(b). There is even tearing of nails through the edges of the panels at the corners; see Figure 4.18(d). The cripple wall also began to exhibit uplift of the corner trim and the corner studs. The sill plate remained in contact with the foundation. As with the 2-ft-tall counterpart Specimen A-11, the underlying panel had begun to slip away from the overlying panel due to the lack of fasteners keeping it in place; see Figure 4.18(a).

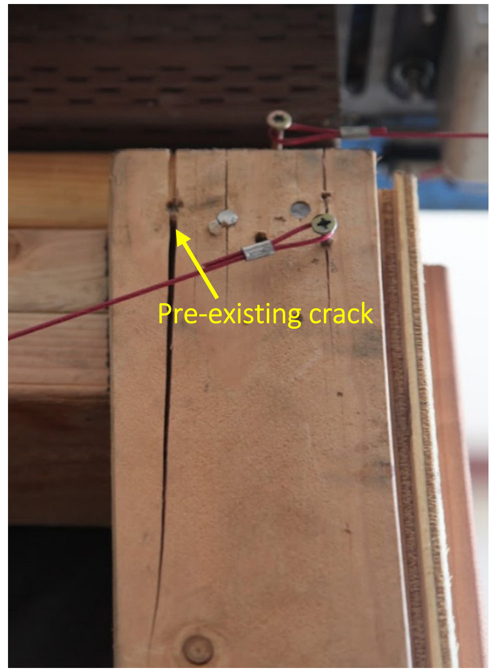


(a)

(b)

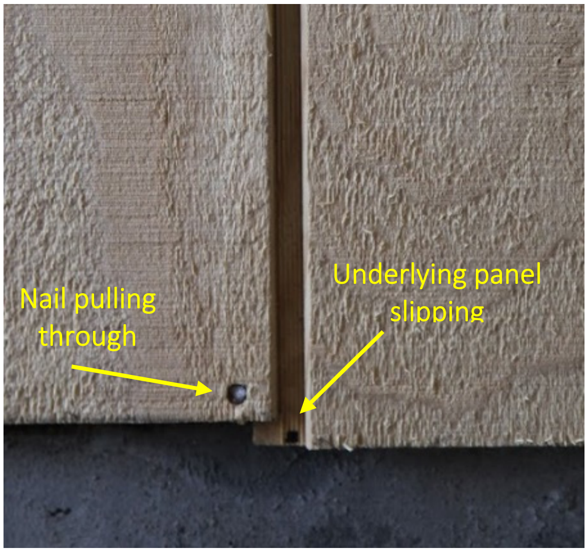
Figure 4.17 Specimen A-23 pre-test photographs of the existing 6-ft-tall cripple wall finished with T1-11 wood structural panels): (a) exterior elevation; (b) interior elevation; (c) south-end exterior of wall corner view; and (d) top of south-end of wall corner view.

(c)

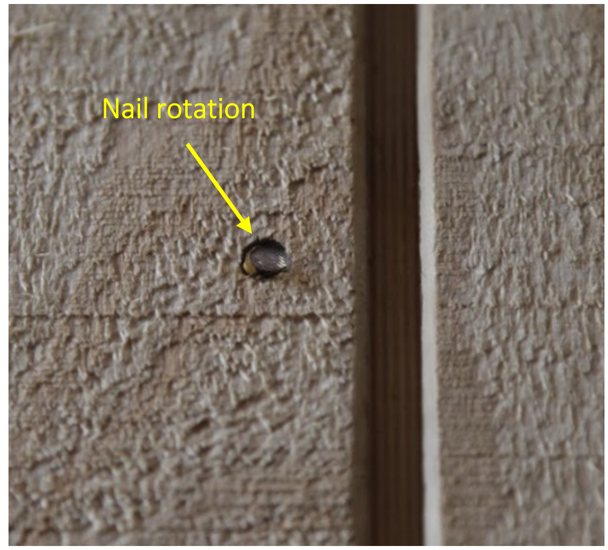


(d)

Figure 4.17 (continued).



(a)



(b)

(c)

(d)

Figure 4.18 Specimen A-23 damage state details at -1.4% drift ratio level @ Δ = -1.008 in.: (a) bottom middle exterior of wall view (south and middle panels); (b) middle exterior of wall view; (c) bottom of south-end interior of wall corner view; and (d) top middle exterior of wall view (south and middle panels).

4.2.10 Specimen A-24: 0% to 1.4% Drift Ratio Level

The photographs in Figure 4.19 show the initial state of Specimen A-24 prior to testing. Specimen A-23 is a retrofitted 6-ft-tall cripple wall with 5/8-in.-thick T1-11 wood structural panels. With the added retrofit, no plywood was attached to the interior of the cripple wall. Instead, the edge nailing spacing was reduced from 8 in. on center to 4 in. on center. In addition, an additional row of nails was attached to the underlying panel at the panel overlaps. These nails were also spaced at 4 in. on center. The height of the cripple wall was the same as the previous 6-ft-tall specimens (6 ft-1-1/2 in.), but the length of the cripple wall was 12 ft instead of 12 ft-4-1/2 in. The length was shortened to accommodate three 4-ft sections of T1-11 plywood. The T1-11 panels came in 4-ft × 8-ft sections, so no partial widths of the panels were used. An 1/8-in. gap is left between panels. The panels are fastened with 8d common hot-dipped galvanized nails. The nailing through the field is 12 in. on center.

Figure 4.20 shows details of the damage at the -1.4% drift. At this point, the sill plate has a cross-grain split on both ends of the wall which can be seen in Figure 4.20(a) and (c). There is uplift of the not only the corner studs but also the interior studs. The studs are also rotated as shown in Figure 4.20(b). Through the middle of the cripple wall, the sill plate is bending, and there is a ¼ in. gap between the sill plate and the foundation in the middle of the sill plate span; see Figure 4.20(e). Figure 4.20(f) shows the bending of the sill plate in the middle of its span along with the uplift and rotation of the studs in the middle of the cripple wall. Slight rotation of the panel has started to occur, and the nails have begun to rotate; see Figure 4.20(d).

(a)

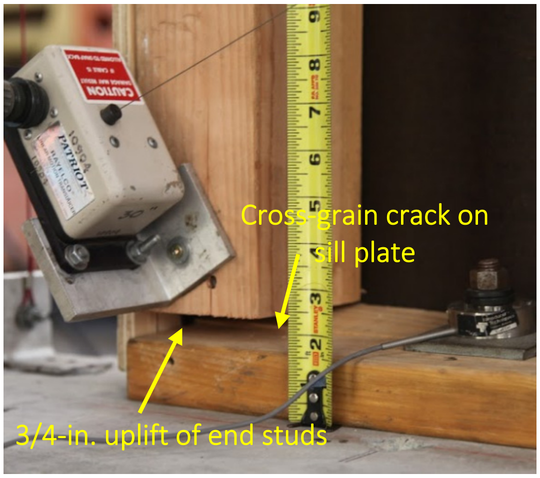
(b)

Figure 4.19 **Specimen A-24 pre-test photographs of the retrofitted 6-ft-tall cripple wall finished with T1-11 wood structural panels: (a) exterior elevation; (b) interior elevation; (c) north-end exterior of wall corner view; and (d) north-end interior of wall corner view.**

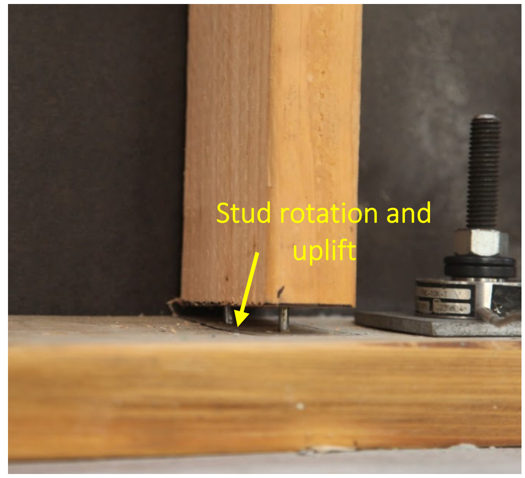
(c)

(d)

Figure 4.19 (continued).

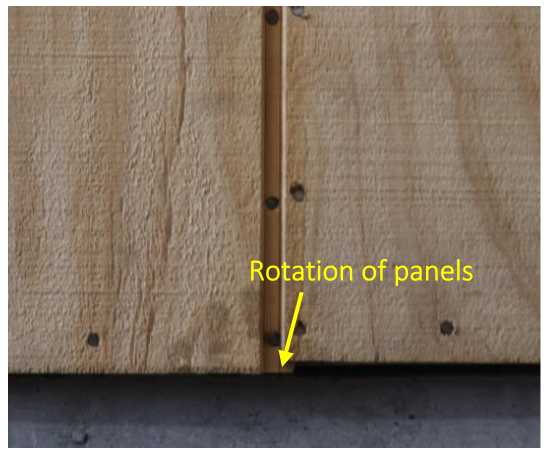


(a)

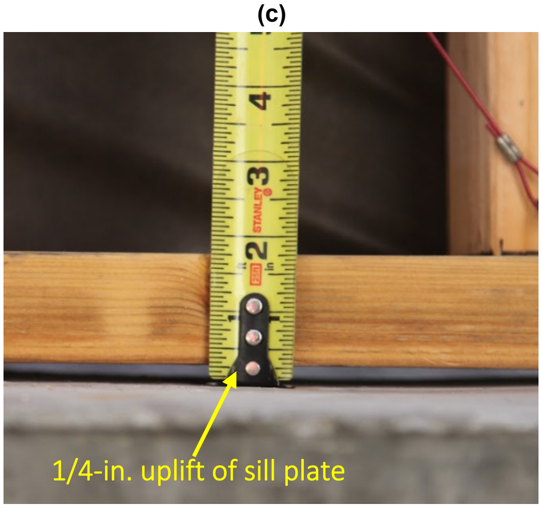


(b)

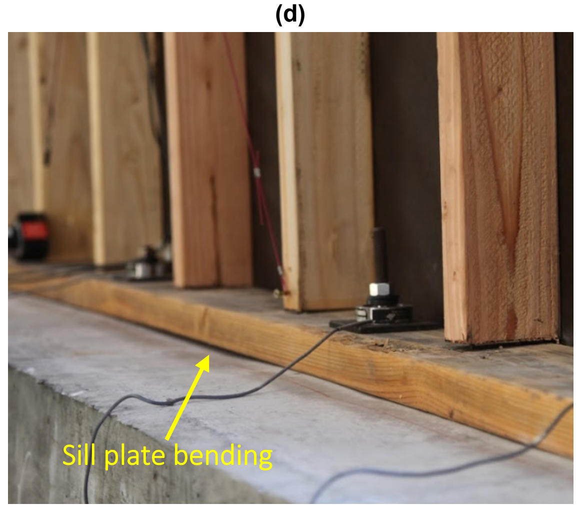
(c)



(d)



(e)



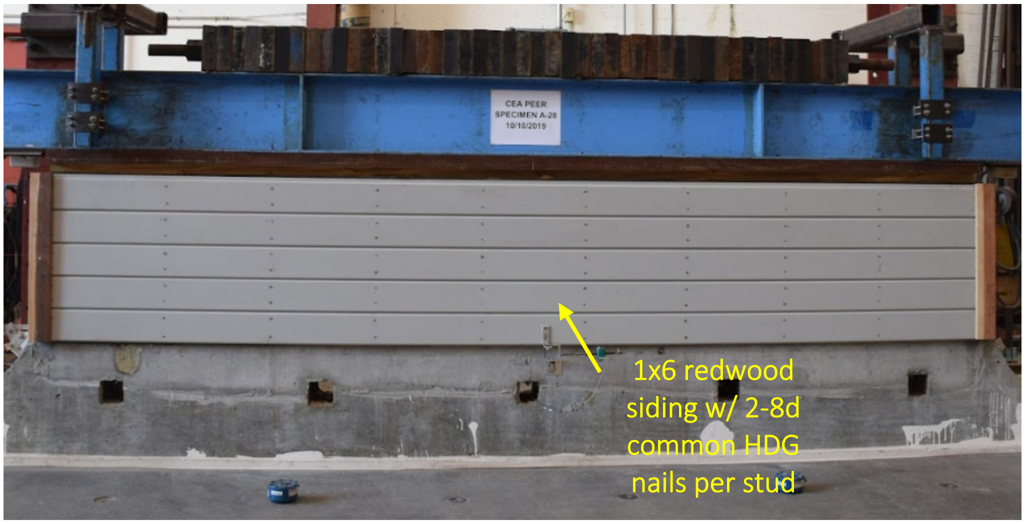
(f)

Figure 4.20 **Specimen A-24 damage state details at -1.4% drift ratio level @ Δ = -1.008 in. unless otherwise noted: (a) bottom of north-end of interior of wall corner view at +1.4% drift; (b) bottom of interior of wall view; (c) bottom of south-end of wall corner view; (d) bottom of middle exterior of wall view (south and middle panels); (e) bottom of middle interior of wall view; and (f) bottom of interior of wall view.**

4.2.11 Specimen A-28: 0.0% to 1.4% Drift Ratio Level

The photographs in Figure 4.21 show the initial state of Specimen A-28 prior to testing. Specimen A-28 is an existing 2-ft-tall cripple wall with horizontal siding over diagonal sheathing exterior finish. It was constructed with the same details as Specimen A-9. The difference between the two tests is the amount of vertical load applied on the cripple wall. Specimen A-9 had 450 plf imposed on it, which is representative of the weight of a two-story dwelling with heavy construction materials. Specimen A-29 had 150 plf of vertical load imposed on it. This weight correlated to the weight of a one-story dwelling with light construction materials. The vertical load came from steel plates placed along the horizontal steel transfer beam, instead of hydraulic jacks imposing two point loads on the steel beam, the configuration for all other cripple walls. The siding is 1 × 6 redwood fastened with two 8d common hot-dipped galvanized nails per stud. The diagonal sheathing boards 1 × 6 construction grade Douglas Fir were oriented at a 45° angle from southwest to northeast. There were 1/8-in. gaps between the sheathing boards and the siding boards. The dimensions of the wall were 12 ft-4-1/2 in. in length and 2 ft-1-1/2 in. in height.

No visible changes to the cripple wall finishes occurred from 0.0% to 0.8% drift. Figure 4.21 gives close-up photographs of the cripple wall at -1.4% drift. There is a 1/4-in. uplift of the end studs and a 1/4-in. uplift of the sill plate; see Figure 4.21(a). The increased uplift at the ends of the cripple wall compared with other specimens is due to the light vertical load as well as the diagonal sheathing. Figure 4.21(c) and (d) show the sheathing boards at +1.4% drift and -1.4% drift. There is no visible opening or closing of the gaps between the sheathing boards.



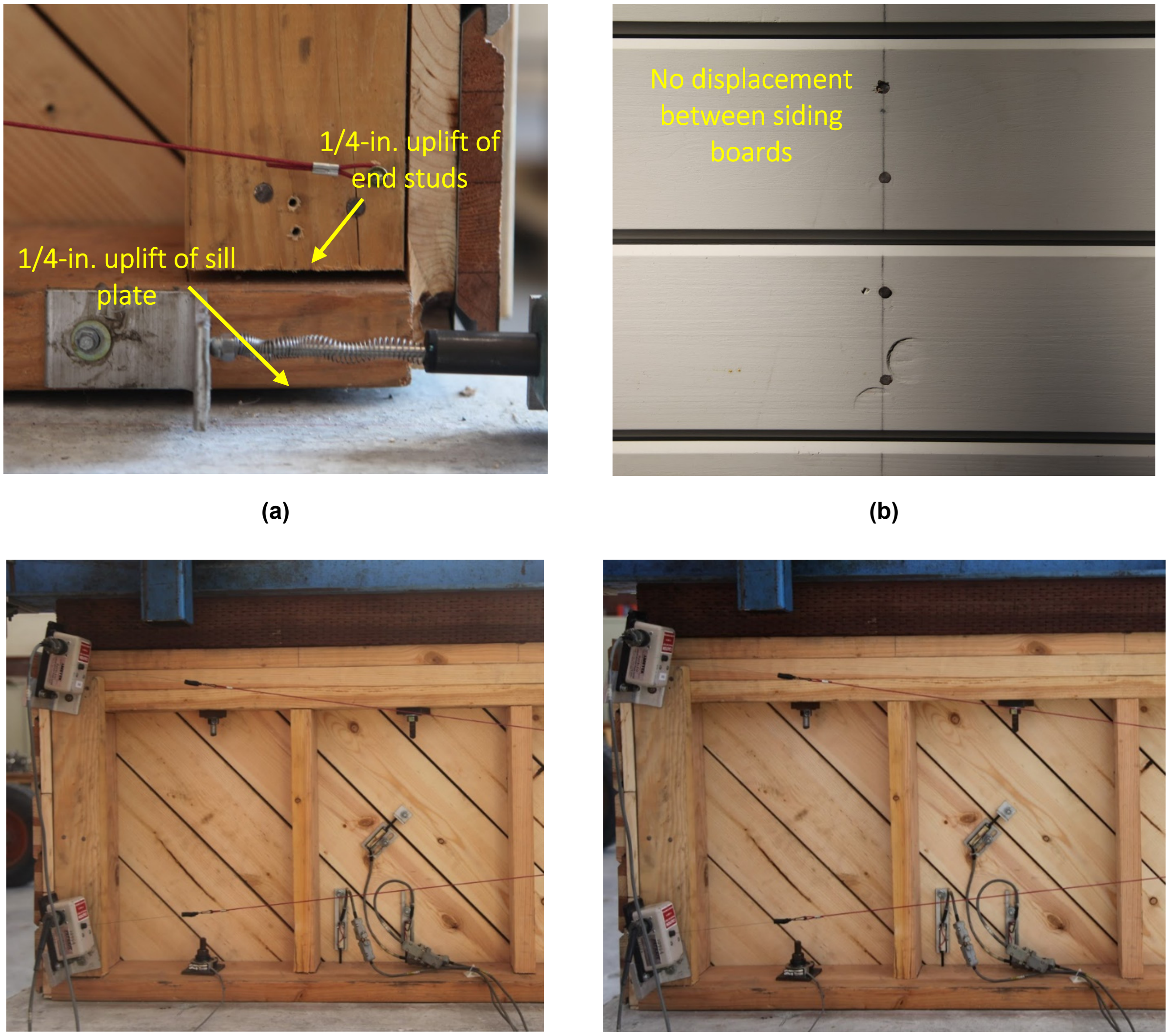
(a)

(b)

Figure 4.21 Specimen A-28 pre-test photographs for the existing 2-ft-tall cripple wall finished with horizontal siding over diagonal sheathing, light vertical load): (a) exterior elevation; (b) interior elevation; (c) north-end exterior of wall corner view; and (d) north-end interior of wall corner view.

(c) (d)

Figure 4.21 (continued).



(a) (b)

(c) (d)

Figure 4.22 Specimen A-28 damage state details at -1.4% drift ratio level @ Δ = -0.336 in. unless otherwise noted: (a) bottom of south-end of interior of wall view; (b) middle interior of wall view; (c) north-end interior of wall view at +1.4% drift; and (d) north-end interior of wall view.

4.3 DAMAGE CHARACTERISTICS AT LATERAL STRENGTH

A key damage state is that which occurs at the lateral strength for each cripple wall. The damage features presented in this section indicate that the cripple wall has reached peak capacity and any larger imposed drifts will result in a loss of capacity. By examining the damage states at this level, insight can be made as to how and why failure is occurring in a cripple wall. All lateral strengths for the eight cripple walls came between 4.0–12.0% of global drift ratio and 3.6–12% of relative drift ratio—relative drift defined as the drift of the cripple wall only, ignoring any sliding of the sill plate. The large range in drifts a peak load is a result of the exterior finishes, cripple wall height, and retrofit condition.

4.3.1 Specimen A-7 Lateral Strength

Figure 4.23 shows the interior and exterior views of the cripple wall for Specimen A-7 at strength (-4% drift). The lateral strength occurred at 4% global and relative drift ratio in both the push and pull direction. Specimen A-7 did not experience any sill slip due to the lack of capacity to overcome the frictional resistance keeping the cripple wall in place. Overall, there were not many visual damages that could be attributed to the cripple wall reaching peak capacity. A comparison of Figure 4.23 and Figure 4.3 (-1.4% drift of Specimen A-7) show little difference besides increased tilt of the cripple wall. The siding boards remained intact and uncracked on the exterior face as well as the corners of the specimen. There was no splitting or cracking of the framing members either. The most prominent change is the condition of the building paper, which began tearing at peak load.

Figure 4.24 shows close-up photographs of Specimen A-7 at -4% drift. There was no displacement between the top siding board and the uppermost top plate of the cripple wall [(Figure 4.24(a)] and an 1/8-in. displacement between the bottom siding board and the foundation; see Figure 4.24(b). The tearing of the building paper originated at the studs where the building paper was stapled and the siding boards were nailed, as seen in Figure 4.24(c). Gaps began increasing in the siding boards, which is most easily seen at the corners; see Figure 4.24(d). The displacement between the siding boards continued to increase through all increases in displacement. This can be seen in Figure 4.24(e) where the corner trim originally covers the nail pattern on the siding boards but now the nails are visible on the upper three siding boards. There was also slight uplift on the corners, as shown in Figure 4.24(f). The lateral strength per linear foot of Specimen A-7 was 186 in the push direction and 162 plf in the pull direction. By 12% drift, these loads had decreased to 135 plf and 127 plf, a 27% decrease in the push direction and a 22% decrease in the pull direction. Compared with other existing cripple walls with different exterior finishes, this is the smallest decrease in capacity for post-peak drift cycles by a wide margin. The reason for a drop of capacity is likely due to the framing rather than the horizontal siding boards. The nails connecting the framing members withdrew from the framing members as displacements were imposed on the wall, and they lost withdrawal strength with repeated cycles of being withdrawn in one direction and then the other direction. Specimen A-7 is unique compared to all other tested cripple walls in the sense that there are no visual cues that would indicate peak capacity had been reached. Much of the strength of the cripple wall came from the framing itself because of the low capacity of the specimen.

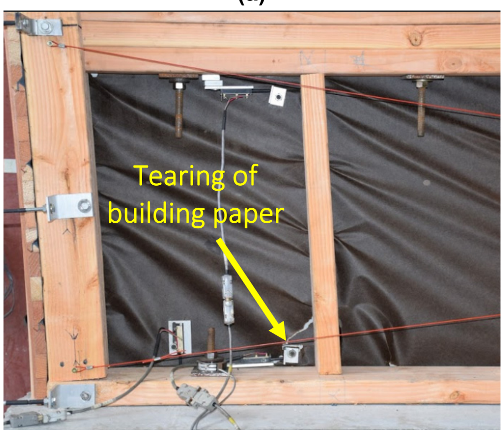
(a)

(b)

Figure 4.23 Specimen A-7 damage state at lateral strength at -4% drift ratio level @ Δ = -0.96 in.: (a) exterior elevation; and (b) interior elevation.

(a)

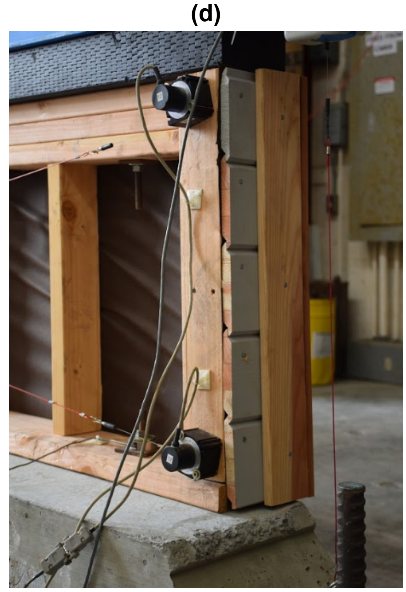
(b)



(c)

(d)

(e)



(f)

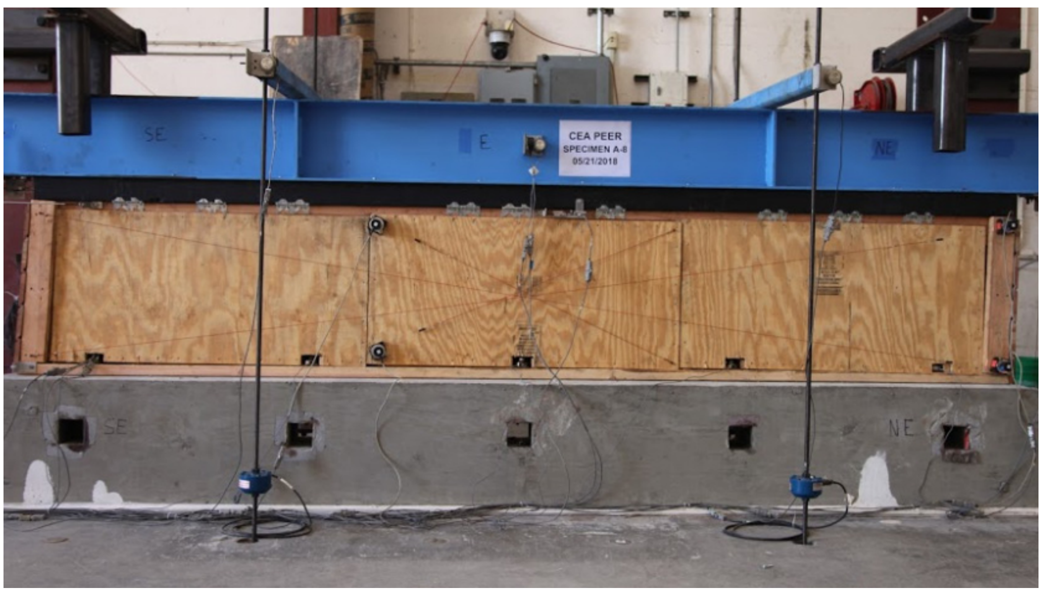
Figure 4.24 Specimen A-7 damage state details at lateral strength at -4% drift ratio level @ Δ = -0.96 in.: (a) top of middle exterior of wall view; (b) bottom of middle exterior of wall view; (c) north-end interior of wall view; (d) top of north-end interior of wall view; (e) south-end exterior of wall view; and (f) north-end interior of wall corner view.

4.3.2 Specimen A-8 Lateral Strength

Figure 4.25 shows the interior and exterior views of the cripple wall for Specimen A-8 at strength. The lateral strength occurred at 8% global drift ratio in the push direction and 7% global drift ratio in the pull direction and 6.4% relative drift ratio in the push direction and 4.7% relative drift ratio in the pull direction. The sill-to-foundation displacement accounted for 1.6% drift in the push direction and 2.3% drift in the pull direction. From Figure 4.42, it can be seen that the exterior face of the cripple wall shows little visual damage (only pronounced tilt of the wall), while the interior face shows extensive damages as the plywood panels cannot rotate to the degree that the cripple wall is tilted over.

Figure 4.24 shows key close-up photographs of the damage to Specimen A-8. There an 1/8-in. displacement between the top siding board and the uppermost top plate [Figure 4.24(a)], and there is a 1/2-in. displacement between the bottom siding board and the foundation [Figure 4.24(b)], which is the amount of sliding between the sill plate and foundation. In addition, as shown in Figure 4.43(f), there is 3/8-in. displacement between the siding boards. A view of the interior face of the cripple wall demonstrates why the cripple wall reached peak capacity. Figure 4.24(c) shows that the nails attaching the plywood to the framing had heavily rotated and in some cases either ripped through the edges of the plywood, pulled out from the framing, or tore through the plywood. This image shows that the plywood had displaced around a 1/2 in. from the blocking that it was originally flush with. This means that the nails attaching the plywood to the blocking have worked out of the blocking by close to a 1/2 in. At both bottom corners of the cripple wall, the plywood was crushed after abutting against the corner flat stud; see Figure 4.24(d). The corner of the wall was pushed outward by 1/4 in. off the sill plate due to contact with the plywood panel. Typically, nails connecting the blocking the plywood would tear through the plywood while nails further up the wall would pull out of the framing, as seen in Figure 4.24(e). Regardless if the nails pull out of the framing or tear through the plywood, their mobilization reduces the amount of shear resistance that the plywood is able to provide. In many cases, the blocking of the cripple wall split at the plywood-to-blocking nails points of connection, which greatly reduced the capacity of the nail as there is less frictional resistance required to pull the nail out of the blocking.

(a)



(b)

Figure 4.25 Specimen A-8 damage state at lateral strength at +8% drift ratio level @ Δ = +1.92 in. and at -7% drift ratio level @ Δ = -1.68 in.: (a) exterior elevation at +8% drift; and (b) interior elevation at -7% drift.

Figure 4.26 **Specimen A-8 damage state details at lateral strength at +8% drift ratio level @ Δ = +1.92 in.: (a) top exterior of wall view; (b) bottom exterior of wall view; (c) bottom of north-end interior of wall view; (d) bottom of middle interior of wall view (north and middle panels); (e) middle interior of wall view (south and middle panels); and (f) middle exterior of wall.**

4.4.3 Specimen A-9 Lateral Strength

Figure 4.27 shows the interior and exterior views of the cripple wall for Specimen A-9 at strength in the push direction, and Figure 4.29 shows the interior and exterior views of the cripple wall for Specimen A-9 at strength in the pull direction. The lateral strength occurred at 7% global drift ratio in the push direction and 10% global drift ratio in the pull direction and 5.0% relative drift ratio in the push direction and 5.2% relative drift ratio in the pull direction. The sill-to-foundation displacement accounted for 2.0% drift in the push direction and 4.8% drift in the pull direction. Because the wall achieved strength at largely different displacement amplitudes, they will be treated separately in this discussion.

Figure 4.28 shows close-up photographs of the cripple wall at strength in the push direction (+7% drift). On the finish face, a crack formed in the corner trim on the south corner; see Figure 4.28(d). Also, on the finish face, no displacement between the siding and the top plate occurred, and the sheathing boards protrude on the top of the cripple wall, as seen in Figure 4.28(c). If the cripple wall were to have a floor above, it is likely that the sheathing boards would be protruding at the base of the cripple wall due to the fact that the diagonal sheathing boards would run from the cripple wall up to the first floor, increasing their resistance to move upward. Figure 4.28(e) and (f) show the tilt of the cripple wall. There is a 1/4-in. displacement between the siding boards, with the displacement between the top siding two board and the bottom two siding boards slightly larger than all of the other interfaces. Figure 4.28(a) and (b) show the increased gaps that formed between the sheathing boards. In addition, there are several cracks in the sheathing boards originating from where the sheathing was nailed to the studs. These cracks are partly why the cripple wall reached capacity in the push direction. The other reason for peak capacity was caused by the nails of connecting the sheathing to the framing beginning to withdraw from the sheathing. This is not visible with the horizontal siding covering the sheathing, but since the sheathing boards are protruding out of the top of the cripple, the nails must be pulling out of the framing.

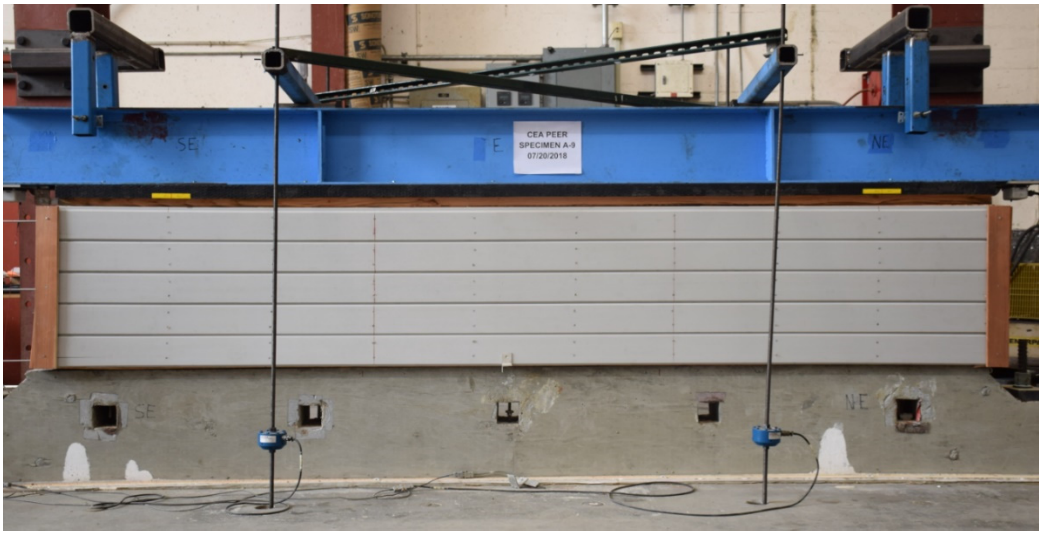
From 7% drift to 10% drift, Specimen A-9 continued to gain capacity in the pull direction. This is due to the gaps between the sheathing boards closing, and the boards bearing on each other as seen in Figure 4.29(b). Once the sheathing boards began to bear down on each other, they began to act in unison like a wood structural panel. This phenomenon is the reason that the capacity of Specimen A-9 in the pull direction, 1713 plf, is comparable with that of Specimen A-7 (1831 plf averaged between push and pull) and Specimen A-13 (1770 plf averaged between push and pull). It is also the reason that the lateral strength in the push direction is 48% larger in the pull direction than the push direction. The strength that the sheathing provides is so large that it caused the studs to laterally displace by 1/4 in., as shown in Figure 4.30(a), which is the first instance of this happening for any existing cripple wall tested in this program. The large loads also caused the anchor bolts to bend significantly, and a large crack formed in the sill plate at the south end of the wall, as seen in Figure 4.30(c) and Figure 4.30(f), respectively. Cracks also formed on studs; see Figure 4.30(b). On the exterior face, additional cracks formed on the siding boards at the corners, as shown in Figure 4.30(e).

(a)

(b)

Figure 4.27 Specimen A-9 damage state at lateral strength in push direction at +7% drift ratio level @ Δ = +1.68 in: (a) exterior elevation; and (b) interior elevation.

Figure 4.28 Specimen A-9 damage state details at lateral strength in push direction +7% drift ratio level @ Δ = +1.68 in.: (a) south-end interior of wall view; (b) north-end interior of wall view; (c) top exterior of middle of wall view; (d) top of south-end exterior of wall corner view; (e) north-end exterior of wall view; and (f) middle exterior of wall view.



(a)

(b)

Figure 4.29 Specimen A-9 damage state of at lateral strength in pull direction at -10% drift ratio level @ Δ = -2.40 in.: (a) exterior elevation; and (b) interior elevation.

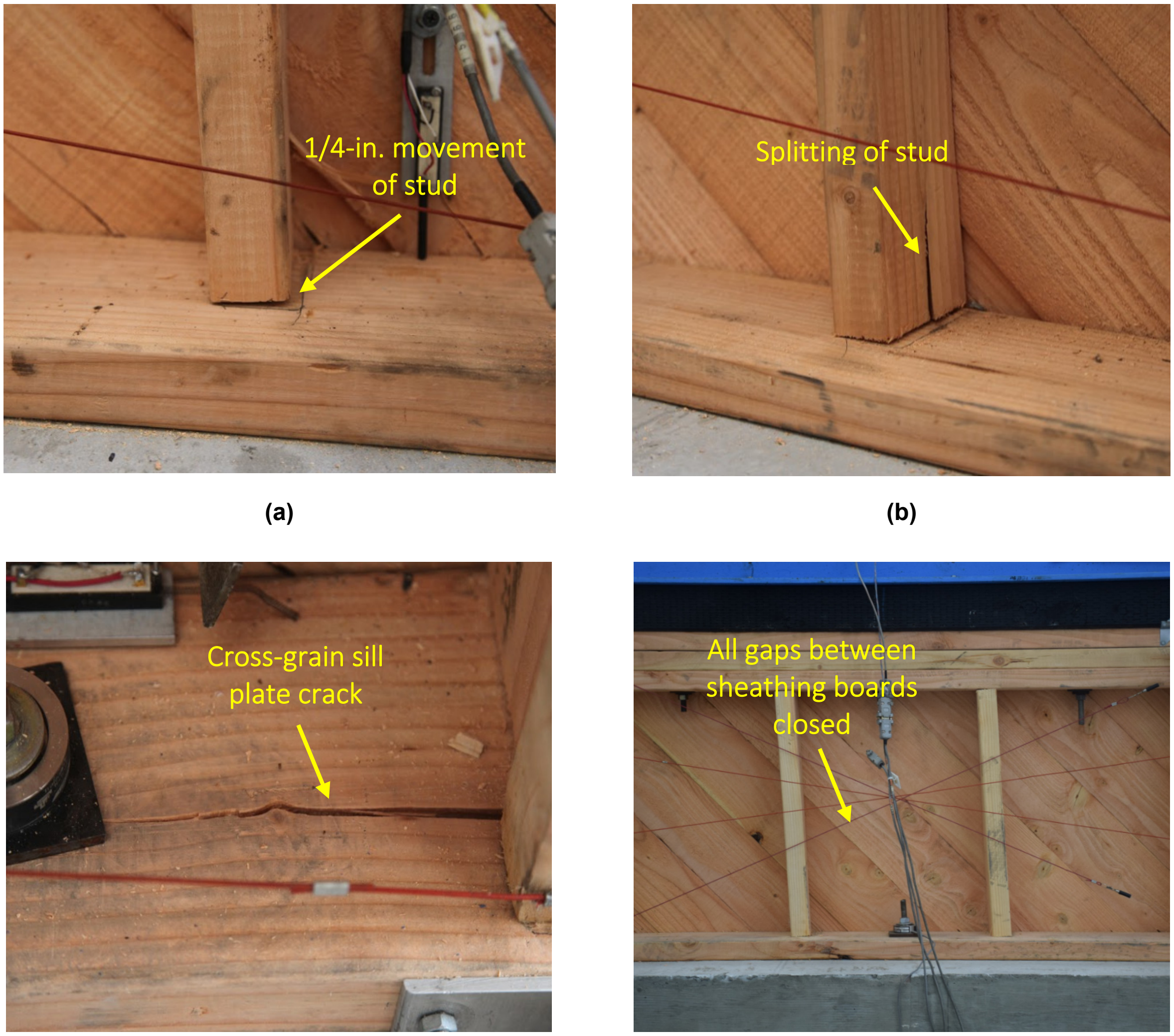


Figure 4.30 Specimen A-9 damage state details for at lateral strength in pull direction at -10% drift ratio level @ Δ = -2.40 in.: (a) bottom interior of wall view; (b) bottom interior of wall view; (c) bottom of south-end interior of wall view; and (d) middle interior of wall view.

4.3.4 Specimen A-10 Lateral Strength

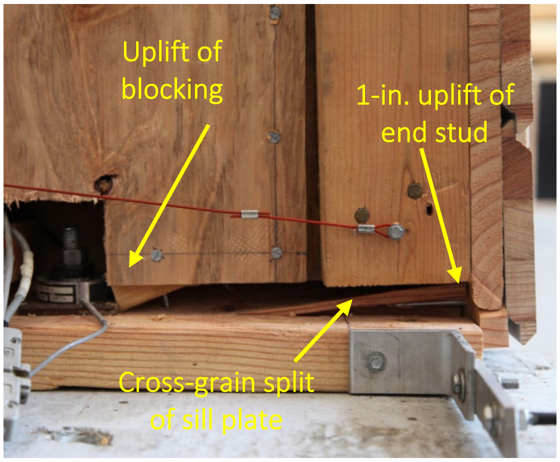
Figure 4.31 shows the interior and exterior views of the cripple wall for Specimen A-10 at lateral strength. The lateral strength occurred at 8% global drift ratio in the push direction and 7% global drift ratio in the pull direction, and 4.5% relative drift ratio in the push direction and 3.6% relative drift ratio in the pull direction. The sill-to-foundation displacement accounted for 3.5% drift in the push direction and 3.4% drift in the pull direction. Unlike existing counterpart Specimen A-10, the response of the cripple wall was close to symmetric, indicating that the plywood originally dominated the response of the cripple. It should be noted that the cripple wall itself never experienced a drop of capacity in the pull direction. The decrease in lateral strength at subsequent displacement cycles was due a fracture in an anchor bolt. It is expected that the cripple wall would have continued to increase in capacity as there was additional anchorage, and it is likely that the response would become asymmetric as was the case with Specimen A-9.

Figure 4.32 shows close-up photographs of key damages to the cripple wall. The most significant damage to the cripple wall was experienced by the anchor bolts and the sill plate. At both ends, the sill plate cracked as the cripple wall uplifted, as shown in Figure 4.49(a) and (b). Figure 4.32(c) and (d) shows bent anchor bolts. These anchor bolts would fracture over increases in the next two displacement cycles. There was significant uplifting of the plywood panels and the blocking; see Figure 4.32(a), (b), and (e). Most of the blocking uplifted but did not split, which is the opposite of that experienced by Specimen A-8. This can be attributed to the diagonal sheathing providing a vertical force component to the cripple wall due to their 45° orientation. Minimal rotation of the nails attaching the plywood to the framing occurred, and no nails tore through the plywood or were pulled out of the framing; see Figure 4.32(g). On the exterior face, there were less than 1/8-in. displacements between the siding boards, which is the lowest of any cripple wall with horizontal siding. Both of these factors indicate that the cripple wall had additional capacity.

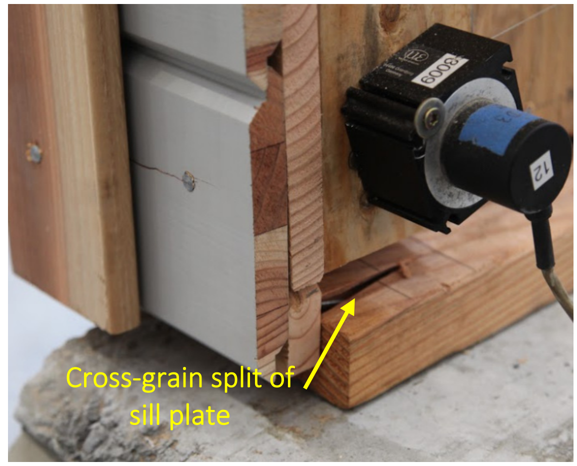
(a)

(b)

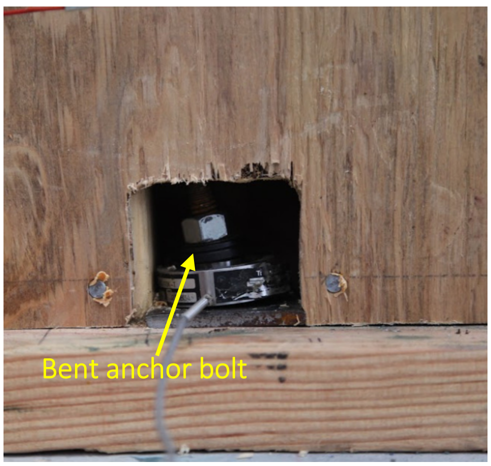
Figure 4.31 **Specimen A-10 damage state at lateral strength at +7% drift ratio level @ Δ = +1.68 in. and at -8% drift ratio level @ Δ = -1.92 in.: (a) exterior elevation at +7% drift; (b) interior elevation at +7% drift.**



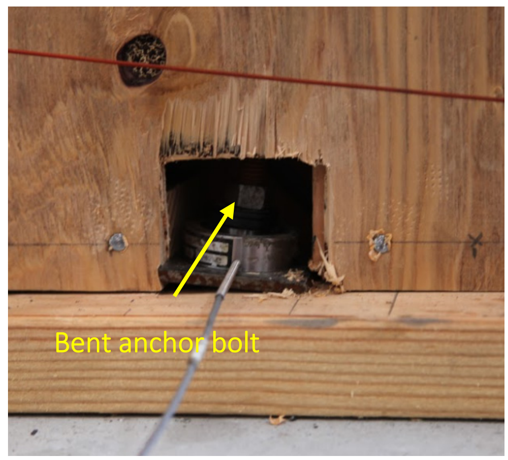
(a)



(b)

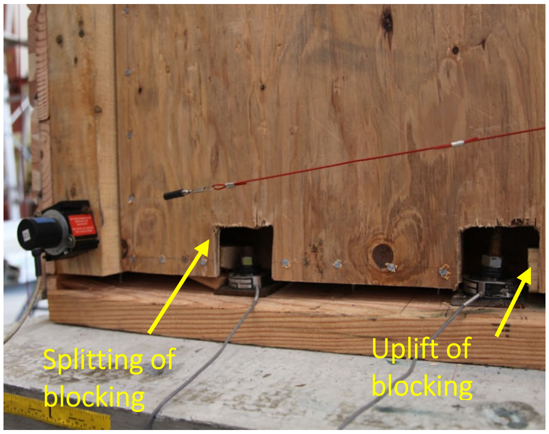


(c)

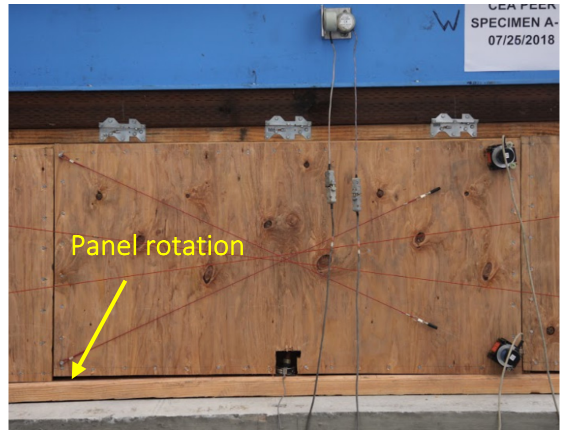


(d)

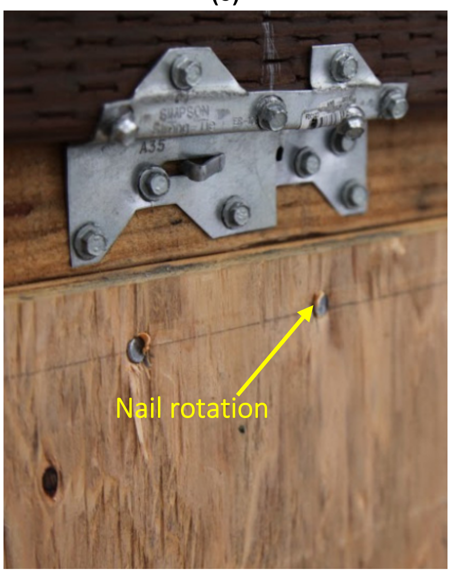
Figure 4.32 Specimen A-10 damage details at lateral strength at +7% drift ratio level @ Δ = +1.68 in. and at -8% drift ratio level @ Δ = -1.92 in.: (a) bottom of south-end interior of wall view at -8 drift; (b) bottom of north-end interior of wall corner view at -8 drift; (c) bottom of middle interior of wall view at +7% drift; (d) bottom of middle interior of wall view at -8% drift; (e) bottom of south-end interior of wall view at -8% drift; (f) middle interior of wall view at -8% drift; (g) top of middle interior of wall view at -8% drift; and (h) middle exterior of wall view at +7% drift.



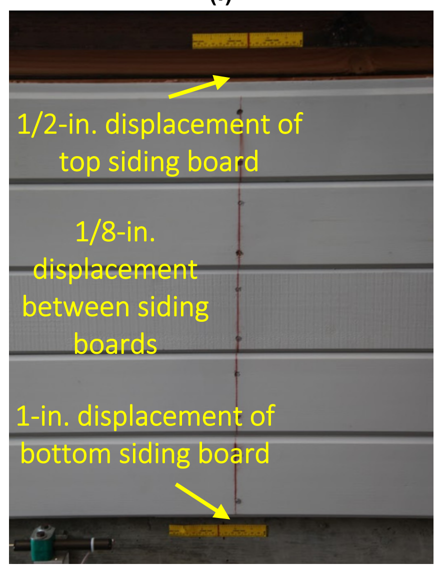
(e)



(f)



(g)



(h)

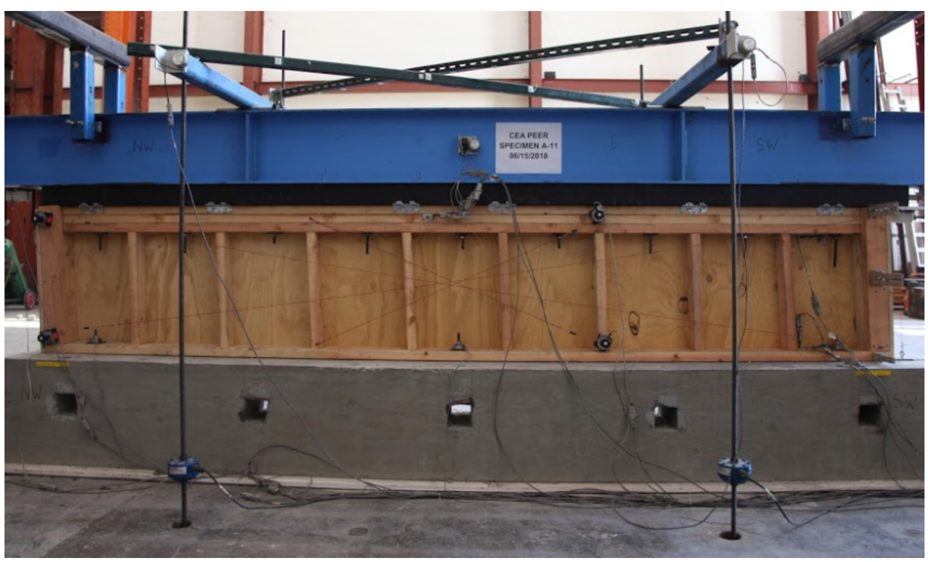
Figure 4.32 (continued).

4.3.5 Specimen A-11 Lateral Strength

Figure 4.33 shows the interior and exterior views of the cripple wall for Specimen A-11 at lateral strength (-5% drift). The lateral strength occurred at 6% global drift ratio in the push direction and 5% global drift ratio in the pull direction, and 4.8% relative drift ratio in the push direction and 4.6% relative drift ratio in the pull direction. The sill-to-foundation displacement accounted for 1.2% drift in the push direction and 0.4% drift in the pull direction. From 5% to 6% global drift ratio, the loads increased by 0.5% in the push direction and decreased by 3% the pull direction. Overall, the hysteretic response up to peak was nearly symmetric.

Figure 4.34 shows close-up photographs of the cripple at lateral strength. At strength, rotations of the panels occurred, which were visible from a distance as shown in Figure 4.33(a). Because the panels were not nailed on one edge—only compressed by the overlying panel—there was less resistance to rotation and ultimately less capacity of the cripple wall if compared with panels having been fastened around their entire perimeter. Figure 4.34(c) and (d) show the underlying panel displacing from the overlying panel as well as the nails on the overlying panel tearing through the T1-11 plywood. The nails on the top and bottom of the panel had rotated heavily and were near pulling through the grooves in the T1-11 panel, as shown in Figure 4.34(e). This photograph shows that the top of the panels had displaced by 1/2 in. relative to the uppermost top plate, with the nails slightly withdrawn from the top plate. At subsequent displacement cycles, the cripple wall would lose capacity to these nails as full tearing through the grooves in the T1-11 panels occurred, leading to a dramatic loss in capacity. The corner trim boards skirted out at the bottom due to the bottom of the T1-11 panels pushing them outward as the bottom, creating a gap between the siding boards [Figure 4.34(f)] as well as a gap between the sill plate and the T1-11 panels.

(a)



(b)

Figure 4.33 Specimen A-11 damage state at lateral strength at -5% drift ratio level @ Δ = -1.20 in.: (a) exterior elevation; and (b) interior elevation.

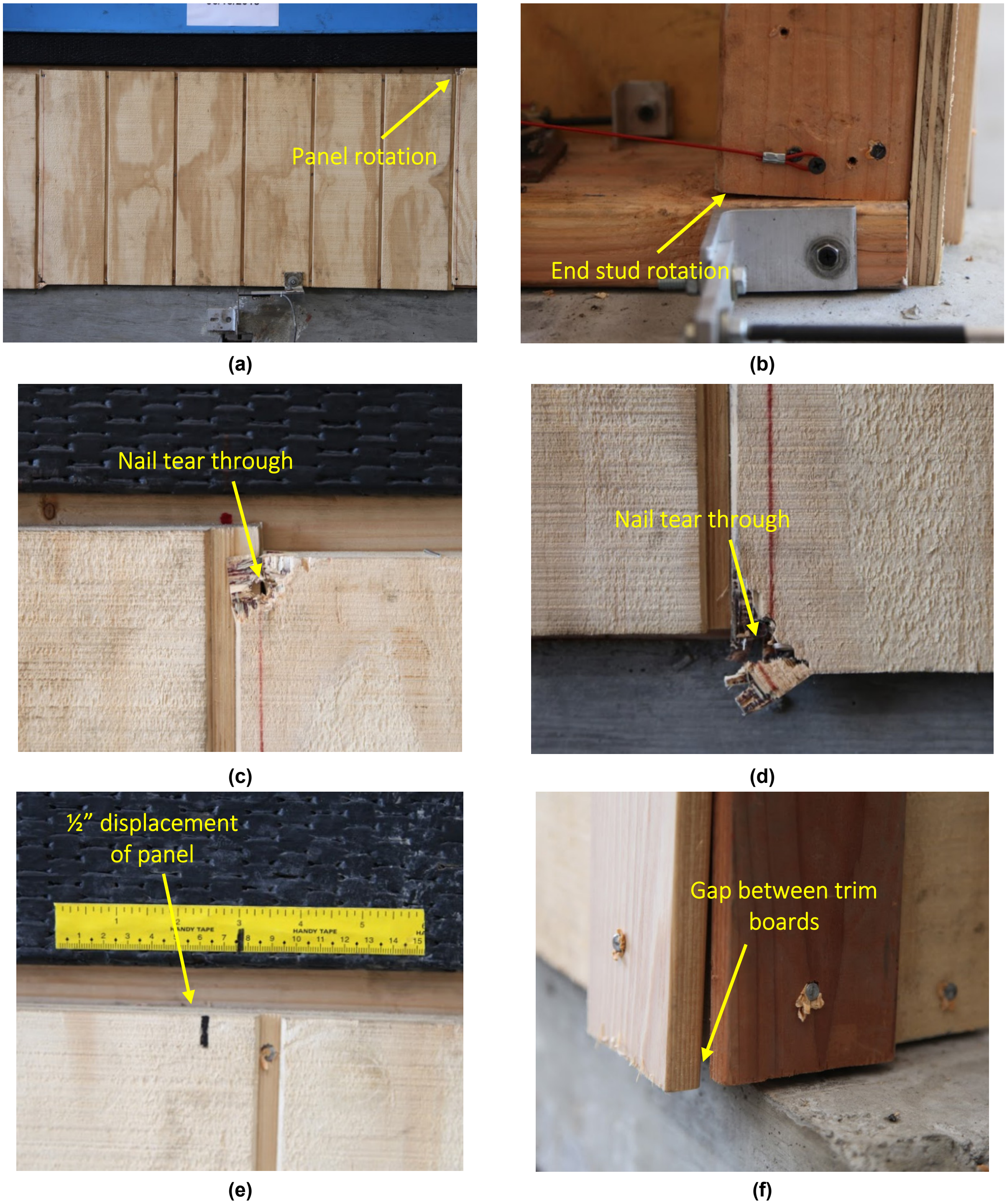
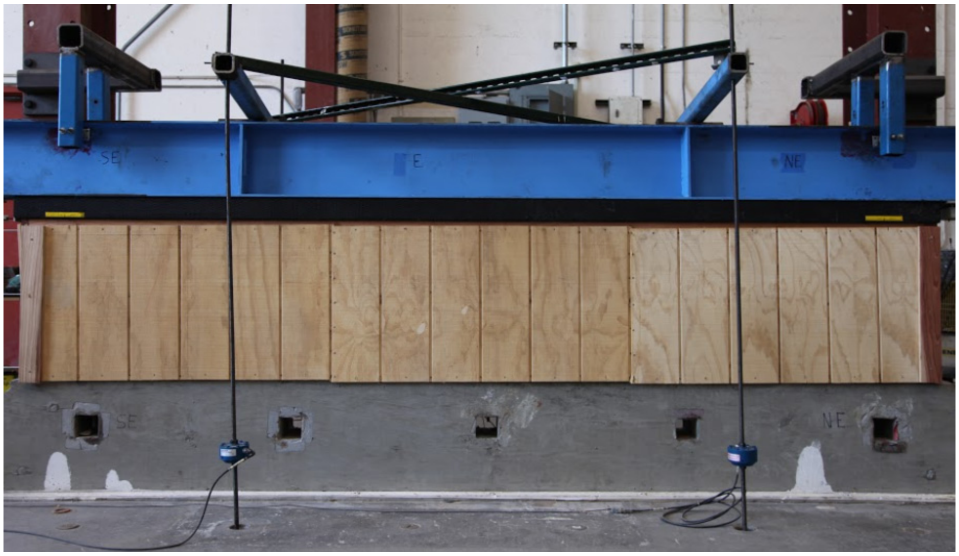


Figure 4.34 Specimen A-11 damage details at lateral strength at lateral strength at -5% drift ratio level @ Δ = -1.20 in.: (a) middle exterior of wall view; (b) bottom of south-end interior of wall view; (c) top of middle exterior of wall view (south and middle panels); (d) bottom of middle exterior of wall view (south and middle panel); (e) top of south-end exterior of wall view; and (f) bottom of north-end exterior of wall corner view.

4.3.6 Specimen A-12 Lateral Strength

Figure 4.35 shows the interior and exterior views of the cripple wall for Specimen A-12 at lateral strength (+7% drift). The lateral strength occurred at 7% global drift ratio in the push direction and 6% global drift ratio in the pull direction, and 4.0% relative drift ratio in the push direction and 4.4% relative drift ratio in the pull direction. The sill-to-foundation displacement accounted for 2.0% drift in the push direction and 1.6% drift in the pull direction. From 5% to 7% global drift ratio, the lateral strength increased by 2.9% in the push direction and 1.1% in the push direction. The cripple wall reached 80% of its peak load in the push and pull direction by 3% global drift ratio. Therefore, the cripple wall reached strength early and maintained its load for a large increase in displacement amplitude. Overall, Specimen A-12 had one of the largest drift capacities and symmetric responses of the cripple walls tested in this program. This can be accredited to the properties of the T1-11 panels and the even nailing over the entire perimeter of the panel.

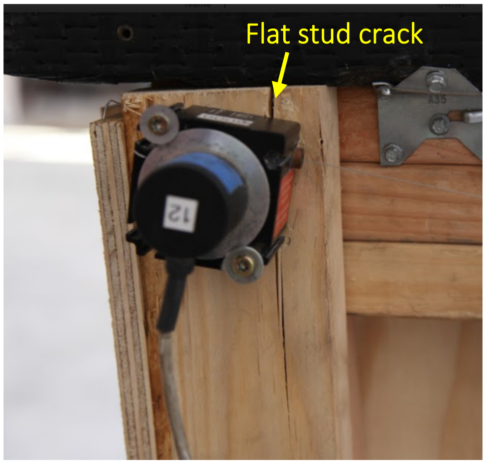
Figure 4.36 shows close-up photographs of key areas, indicating the cripple wall had reached capacity. As with Specimen A-11, the top of the plywood panels had displaced 1/2 in. relative to the uppermost top plate, as shown in Figure 4.36(b), indicating that the fastening nails had slightly pulled out of the framing as well as having slightly pulled through the T1-11 panels for a total of around 1/2 in. At the panel overlaps, the nails on the underlying panel ripped through the edge of the panel, and the nails on the overlapping panel were close to ripping through the panel. In some cases, the nails only rotated or pulled slightly out of the framing; see Figure 4.36(d) and (f). There was and insipient tear through the nails in the grooves whereas the nails between the grooves had more heavily rotated; see Figure 4.36(b). The nails between the grooves were much more resistance to tearing through the panel due to the increased thickness of the panels in these areas. Figure 4.36(c) and (e) show a gap that formed between the corner trim boards at the bottom of the cripple wall, as was the case with its existing counterpart. This occurred as the bottom of the T1-11 panels skirted out. The panels maintained a close connection to the top plate and pulled away from the sill plate as displacement amplitudes increased.



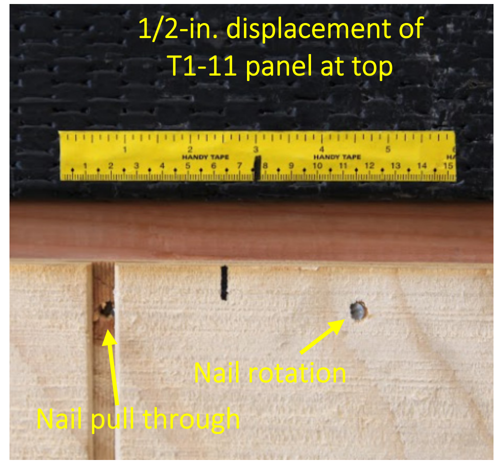
(a)

(b)

Figure 4.35 Specimen A-12 damage state at lateral strength at +7% drift ratio level @ Δ = +1.68 in.: (a) exterior elevation; (b) interior elevation.



(a)



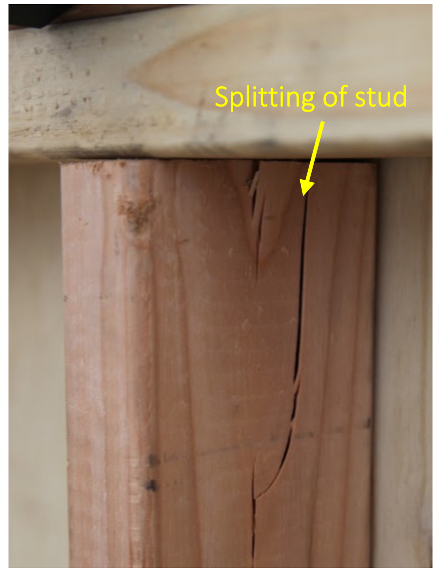
(b)

(c)

(d)

Figure 4.36 Specimen A-12 damage details of damage at lateral strength at +7% drift ratio level @ Δ = +1.68 in.: (a) top of south-end interior of wall view; (b) top of north-end exterior of wall view; (c) bottom of north-end exterior of wall corner view; (d) top of middle exterior of wall view (north and middle panel); (e) south-end exterior of wall view; and (f) top of middle interior of wall view.

(e)



(f)

Figure 4.36 (continued).

4.3.7 SPECIMEN A-13 LATERAL STRENGTH

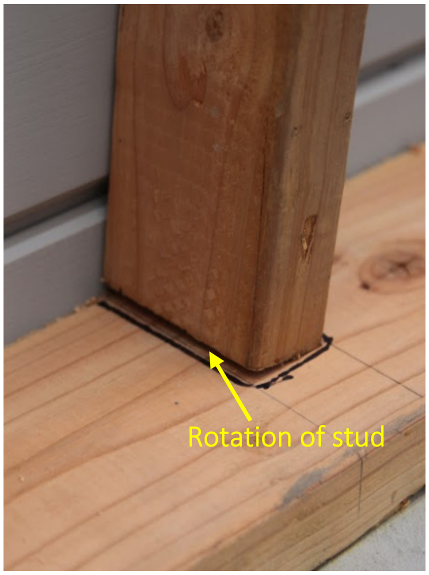
Figure 4.37 shows the interior and exterior views of the cripple wall for Specimen A-13 at lateral strength (-12% drift), which occurred at 11% global and relative drift ratio in the push direction and 12% global and relative drift ratio in the pull direction. Specimen A-13 did not experience any sill slip due to the lack of capacity to overcome the frictional resistance, thereby keeping the cripple wall in place. From 4% to 12% drift, there was only a 9.2% increase in capacity in the push direction and a 11.4% increase in capacity in the pull direction. The cripple wall showed no indications that the capacity of the wall was dropping at 12% drift. Due to the height and finish material of specimen, this was the most flexible cripple wall tested. It was the only cripple wall which a monotonic push was initiated before a drop in peak load occurred. Limitations of the instrumentation prevented larger displacements from being imposed. Specimen A-13 was also the weakest of any specimen by a wide margin. With an average peak load per linear foot between push and pull loading of 93 plf, what little resistance being provided was due to the nailing of the siding boards, the friction between the overlap of the siding boards, and the framing. There were no visual cues on both the interior and exterior face that the cripple wall showed damage to the siding boards, nails fastening the siding to the framing, or the framing; see Figure 4.37. The cripple wall was only heavily tilted. Figure 4.55 shows close-up photographs of the specimen at lateral strength. At this point, the displacement between each panel had reached a ½ in.; see Figure 4.38(a). The bottom of the corner trim boards did not displace from each other, but they did at the top, opening a gap between the trim boards; see Figure 4.38(b), (c), and (d). There was significant rotation of the studs as shown by Figure 4.38(e).

(a)

(b)

Figure 4.37 Specimen A-13 damage state of at lateral strength at -12% drift ratio level @ Δ = -8.64 in.: (a) exterior elevation; and (b) interior elevation.

Figure 4.38 Specimen A-13 damage details at lateral strength at -12% drift ratio @ Δ = -8.64 in. unless otherwise noted: (a) middle exterior of wall view; (b) bottom of north-end exterior of wall corner view; (c) bottom of south-end of wall corner view; (d) top of south-end exterior of wall corner view; (e) bottom of middle interior of wall view; and (f) north-end exterior of wall view at +11% drift.



(e)

(f)

Figure 4.38 (continued).

4.3.8 Specimen A-14 Lateral Strength

Figure 4.39 shows the interior and exterior views of the cripple wall for Specimen A-14 at lateral strength (-5% drift), which occurred at 5% global drift ratio in the push and pull direction and 3.5% relative drift ratio in the push and pull direction. The sill-to-foundation displacement accounted for 1.5% drift in the push and pull direction. Specimen A-14 had the most symmetric hysteretic response up to lateral strength of any of the cripple walls tested in this report. The damage accrued by lateral strength were symmetric on both ends of the cripple wall. Due to the taller height and the large lateral strength, this cripple wall experienced the most uplift of any of the cripple walls even with the presence of hold-downs at both corners.

Figure 4.40 shows close-up photographs of Specimen A-14 at lateral strength. At peak load the plywood panels detached from the top plate by way of the nails pulling through the plywood, as seen in Figures 4.40(a) and (b). The nails remained fastened to the blocking at the sill plate and caused the blocking to uplift; see Figure 4.40(c). The nails pulled through the plywood at the top instead of the bottom of the cripple wall because the top plates were more restrained than the blocking attached to the sill plate. Once the plywood panels had lost attachment at the top, the cripple wall lost capacity. Intrinsic to a taller wall is increased flexure. This is shown by the detachment of the corner studs added for plywood attachment at the top of the cripple wall and not the bottom; see Figure 4.40(d). Not only was there uplift of the blocking, but the sill plate experienced 3/8-in. uplift at lateral strength. On the exterior face of the cripple wall, cracks formed on the upper siding boards at the corners, as shown in Figure 4.40(e). The corner trim boards remained uncracked.

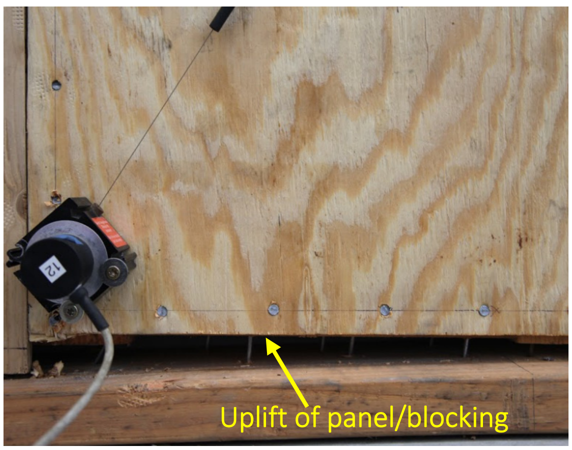
(a)

(b)

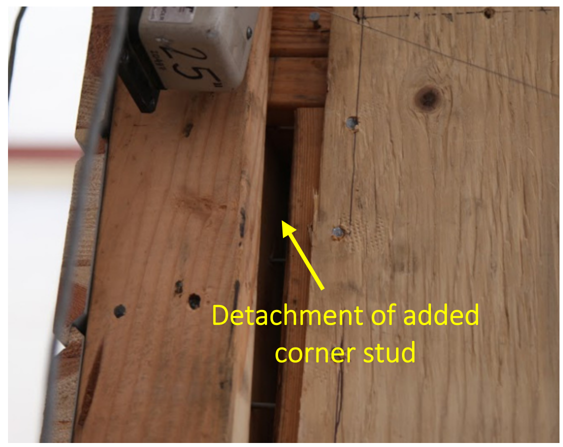
Figure 4.39 Specimen A-14 damage state at lateral strength at -5% drift ratio level @ Δ = -3.60 in.: (a) exterior elevation; and (b) interior elevation.

(a)

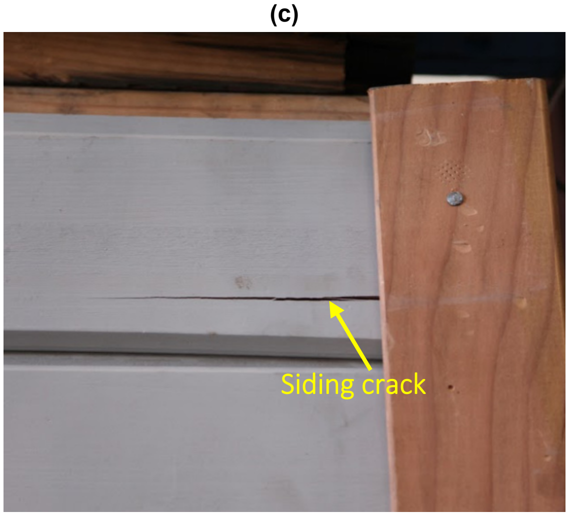
(b)



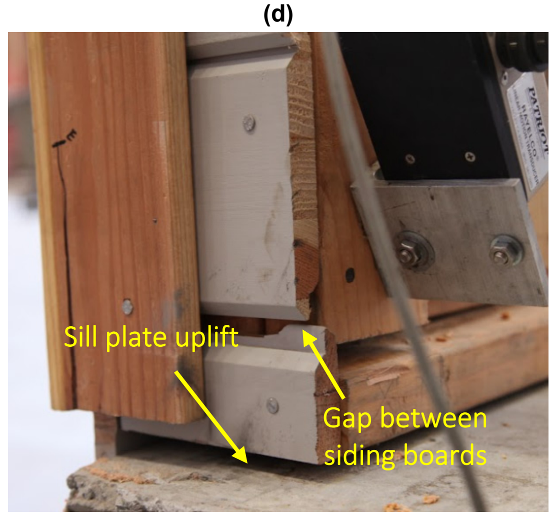
(c)



(d)



(e)



(f)

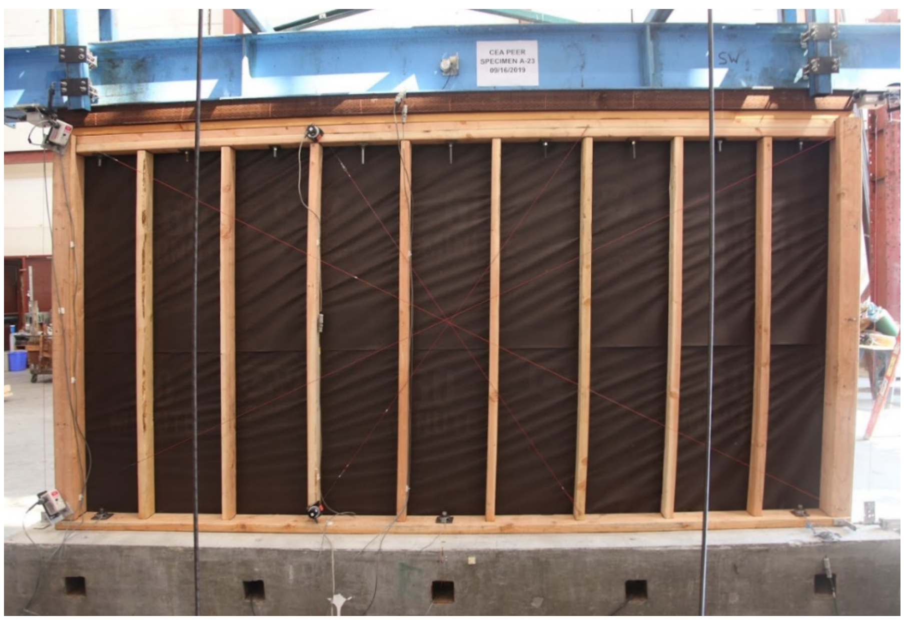
Figure 4.40 **Specimen A-14 damage state details at lateral strength at +5% drift ratio @ Δ = +3.60 in. and -6% drift ratio @ Δ = -4.32 in.: (a) top of north-end interior of wall view at 5% drift; (b) top of south-end interior of wall view at +5% drift; (c) bottom of north-end interior of wall view at -6% drift; (d) top of north-end interior of wall view at -6% drift; (e) top of north-end exterior of wall at -6% drift; and (f) bottom of north-end of wall corner view at +5% drift.**

4.3.9 Specimen A-23 Lateral Strength

Figure 4.41 shows the exterior and interior elevations of the cripple wall at lateral strength (-2% drift), which was achieved at 3% global and relative drift ratio in the push direction and 2% global and relative drift ratio in the pull direction, i.e., that the cripple wall did not have any displacement of the sill plate relative to the foundation at lateral strength. The sill plate did not displace because the lateral strength of the cripple wall was not higher than the resistance to displacement from the friction between the sill plate and the foundation as well as the anchor bolts. At lateral strength, the T1-11 panels had rotated heavily, as shown in Figure 4.41(a). Little damage was visible from the elevation view of the interior face. The building paper showed no tearing, but the studs were bending.

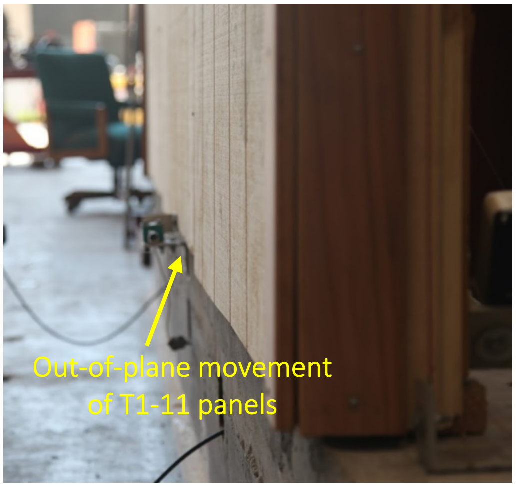
Figure 4.42 gives close-up photographs of the specimen at lateral strength. Figure 4.42(a) shows that the panels were pulling away from the sill plate, with the largest gap formed at the panel overlap where the underlying panel had no nails on that edge. At the sill plate, many of the nails had pulled through the T1-11 panels; see Figure 4.42(e). The sill plate shows less uplift and rotation than in previous displacement cycles due to the reduced amount of nailing attaching the sill plate to panels, i.e., the nails pulling through the panel at the sill plate; see Figure 4.42(b). At the corners, the studs had uplifted 1/4 in. from the sill plate. Along the overlap joints of the T1-11 panels, the underlying panel displaced relative to the overlying panel due to the confinement of the overlying panel on the underlying panel being primary source of resistance to its displacement; see Figure 4.42(f).

(a)

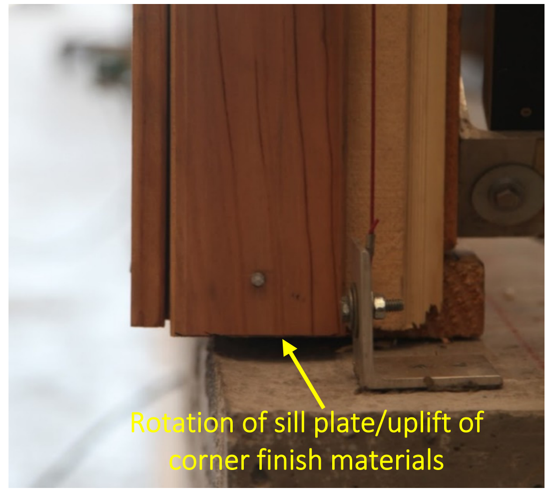


(b)

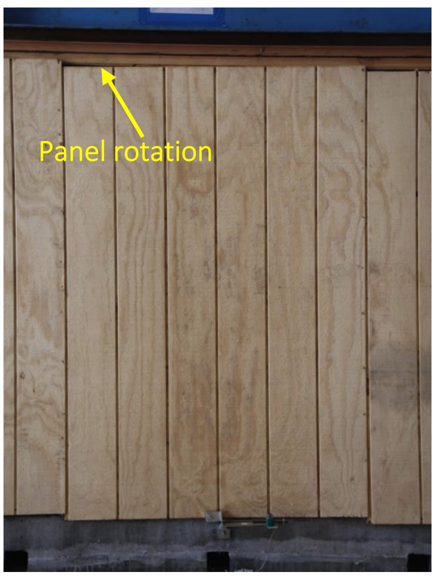
Figure 4.41 Specimen A-23 damage state at lateral strength at -2% drift ratio level @ Δ = -1.44 in.: (a) exterior elevation; and (b) interior elevation.



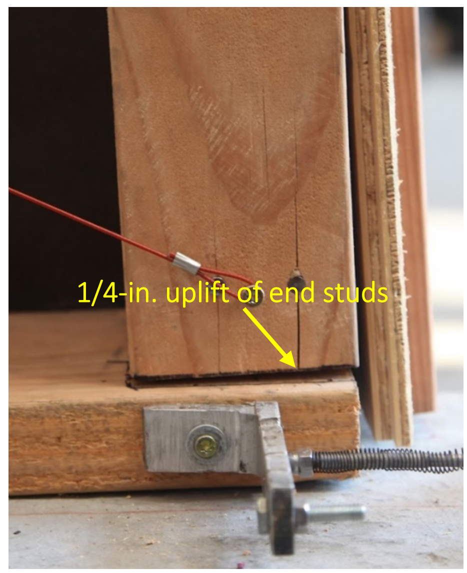
(a)



(b)

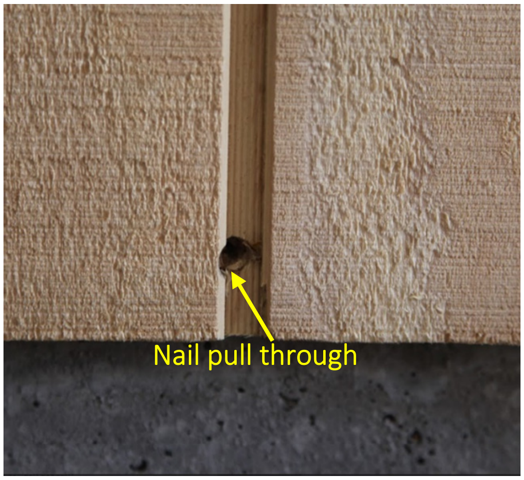


(c)

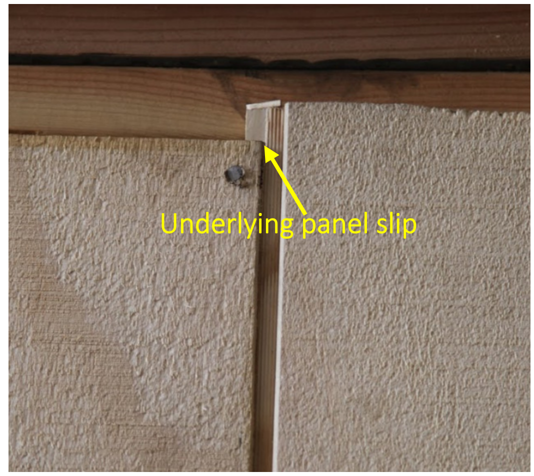


(d)

Figure 4.42 Specimen A-23 damaged details at lateral strength at -2% drift ratio @ Δ = -1.44 in. unless otherwise noted: (a) bottom of north-end exterior of wall corner view; (b) bottom of north-end of wall corner view; (c) middle exterior of wall view at +3% drift; (d) bottom of north-end interior of wall view at +3% drift; (e) bottom of middle exterior of wall; and (f) top of middle exterior of wall view (south and middle panels)



(e)



(f)

Figure 4.42 (continued).

4.3.10 Specimen A-24 Lateral Strength

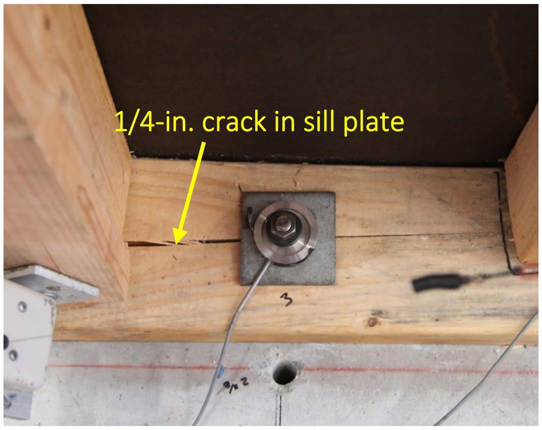
Figure 4.43 shows the interior and exterior views of the cripple wall for Specimen A-24 at lateral strength, which was achieved at 3% global and 2.9% relative drift ratios in the push direction and 2% global and relative drift ratios in the pull direction. The retrofitted cripple wall had little displacement of the sill plate relative to the foundation at lateral strength. While the strength of the wall was high enough to overcome the frictional resistance to the sill plate displacing along the foundation, the increased amount of anchor bolts prevented the movement of the sill plate from occurring. From the interior and exterior elevation, the cripple wall almost appears trapezoidal, i.e., there was significant uplift. The T1-11 panels also showed visible rotation from far away; see Figure 4.43(a).

Close-up photographs of the damage can be seen in Figure 4.44. The corner studs were uplifted 1 in. from the sill plate on the interior face of the cripple wall and uplifted as well along the span of the sill plate. At this point, three-quarters of the studs showed visible uplift, with studs close to the center of the wall uplifted around 3/4 in.; see Figure 4.44(d). The cross-grain cracks in the sill plate at the ends of the wall continued to increase in size and length. The sill plate remained resting on the foundation along the interior face of the wall, while it is rotated and uplifted on the exterior face of the wall; Figure 4.44(h). From the finish face, a visible gap in the center of wall between the sill plate and the foundation can be seen in Figure 4.44(b). Along the T1-11 panels, nails that were connecting the thin sections of the 5/16-in.-thick T1-11 panels to the framing had pulled through or torn through the edges of the panel. The nails fastened to the 5/8-in.-thick sections of the T1-11 panels rotated; see Figure 4.44(e) and (f). There was visible bending of the studs as well; see Figure 4.44(g).

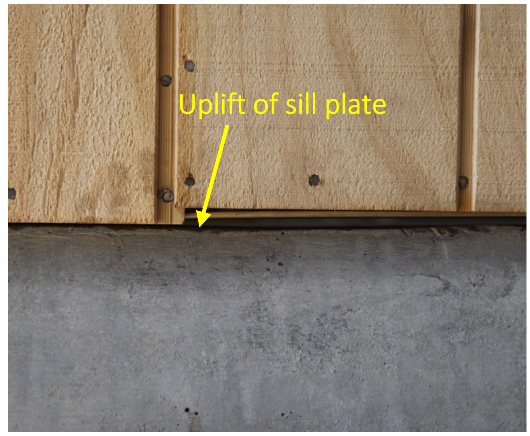
(a)

(b)

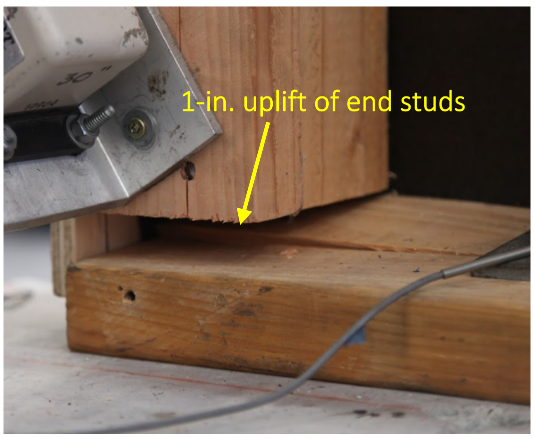
Figure 4.43 **Specimen A-24 damage state of at lateral strength at +3% drift ratio @ Δ = +2.16 in. and -2% drift ratio @ Δ = -1.44 in.: (a) exterior elevation at -2% drift; and (b) interior elevation at +3% drift.**



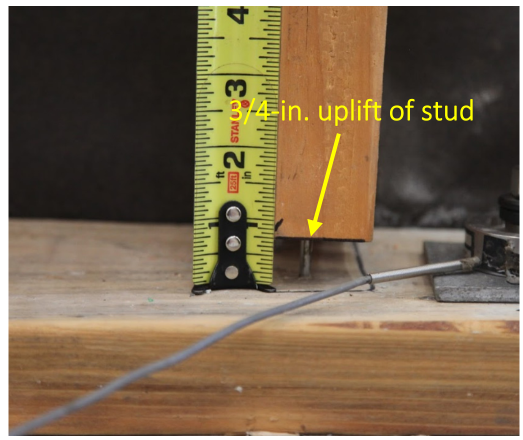
(a)



(b)

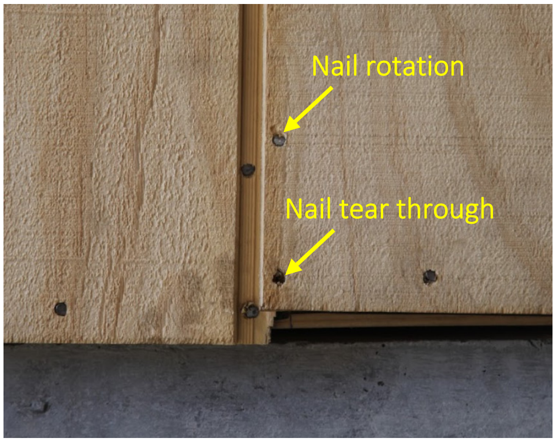


(c)

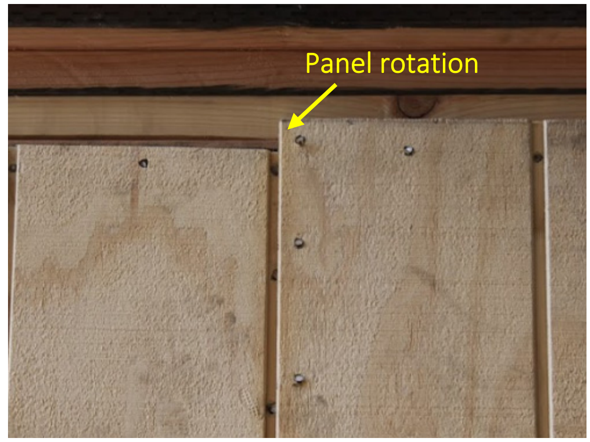


(d)

Figure 4.44 Specimen A-24 damage details at lateral strength at -2% drift ratio @ Δ = -1.44 in. unless otherwise noted: (a) bottom of north-end interior of wall view; (b) bottom of exterior of wall view (north and middle panels); (c) bottom of north-end interior of wall view; (d) bottom of middle interior of wall view; (e) bottom of exterior of wall view (south and middle panels); (f) top of exterior of wall view (south and middle panels): (g) south-end interior of wall view at +3% drift; and (h) bottom of south-end of wall corner view.



(e)



(f)

(g)

(h)

Figure 4.44 (continued).

4.3.11 Specimen A-28 Lateral Strength

Figure 4.45(a) and (b) shows the interior and exterior views of the cripple wall for Specimen A-28 at lateral strength in the push direction. Figure 4.45(c) and (d) shows the interior and exterior views of the cripple wall for Specimen A-28 at lateral strength in the pull direction. The lateral strength occurred at 5% global drift and 3.4% relative drift ratios in the push 7% global drift and 4.2% relative drift ratios in the pull direction. The displacement of the sill plate relative to the foundation at lateral strength accounted for 1.6% drift in the push direction and 2.8% drift in the pull direction. Exterior and interior elevation of the cripple wall at lateral strength in both loading directions is shown. On the interior face of the cripple wall, the gaps between the sheathing boards have widened at lateral strength in the push direction [Figure 4.45(b)], and the gaps had closed at lateral strength in the pull direction; see Figure 4.45(d). The hysteretic response is highly asymmetric with a lateral strength of 786 plf in the push direction and 1123 plf in the pull direction.

This asymmetry is due to the orientation of the sheathing boards. When the gaps between the sheathing boards are fully closed, the bearing between the sheathing boards causes the strength of the wall to dramatically increase.

Figure 4.46 shows close-up photographs of Specimen A-28 at lateral strength. The opening and closing of the gaps between the sheathing boards can be seen in Figure 4.46(c) and (d). The damage to the sill plate can be seen in Figure 4.46(a) and (b). Cracking of the sill plate on both ends extended through two stud bays at each end. The sill plate had uplifted across half of its span. When compared with the behavior of Specimen A-9, the increased amount of uplift is due to the decreased vertical load imposed on the cripple wall. There is also uplift attributed to the gap between the top plates and the studs, which was not observed in Specimen A-9. On the finish face of the wall, displacement between siding boards is visible as well as increased uplift of the sheathing boards; see Figure 4.46(e). Some cracking of the siding boards occurred at the corners, as seen in Figure 4.46(f). The nails attaching the siding to the sheathing and framing are slightly withdrawn; see Figure 4.46(g).

(a)

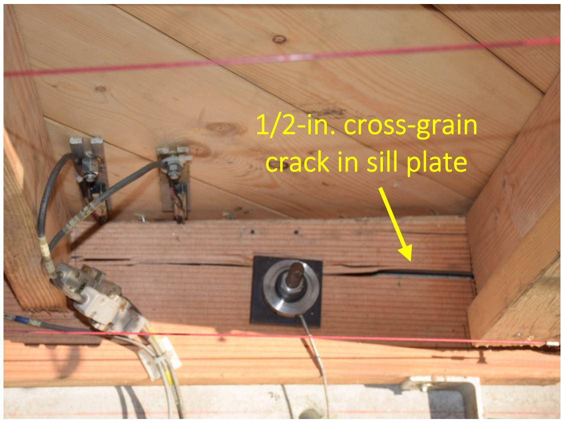
(b)

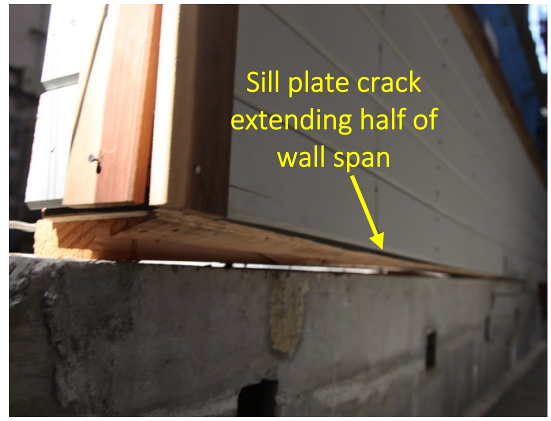
Figure 4.45 Specimen A-28 damage state at lateral strength at +5% drift ratio @ Δ = +1.20 in. and -7% drift ratio @ Δ = -1.68 in.: (a) exterior elevation at +5% drift; (b) interior elevation at +5% drift; (c) exterior elevation at -7% drift; and (d) interior elevation at -7% drift.

(c)

(d)

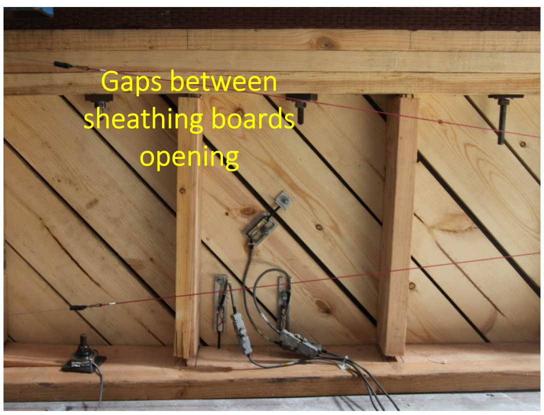
Figure 4.45 (continued).





(a)

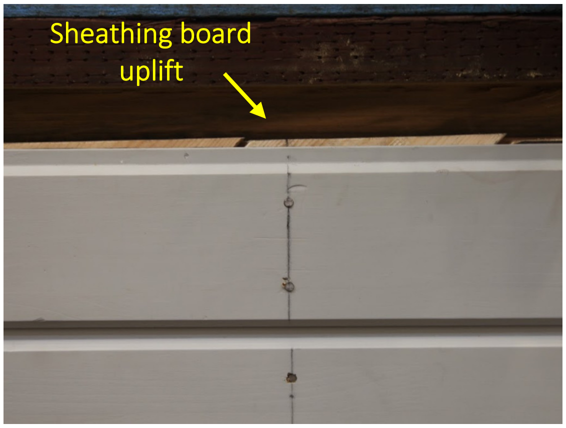
(b)



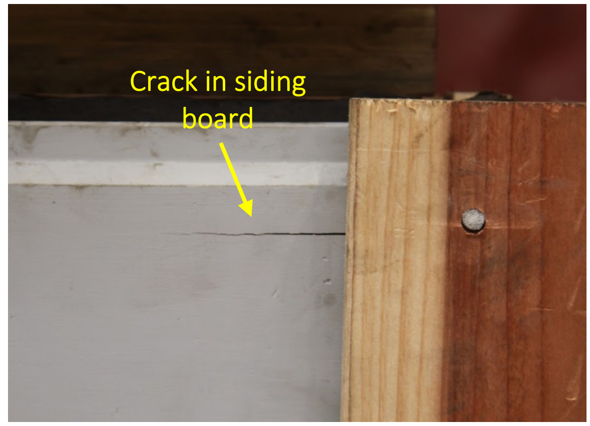
(c)

(d)

Figure 4.46 Specimen A-28 damage details at lateral strength at +5% drift ratio @ Δ = +1.20 in. unless otherwise noted: (a) bottom of south-end interior of wall view; (b) bottom of south-end exterior of wall corner view; (c) north-end interior of wall view; (d) north-end interior of wall view at -7% drift; (e) top of middle exterior of wall view; (f) middle interior of wall view; and (h) middle interior of wall view.



(e)



(f)

(g)

(h)

Figure 4.46 (continued).

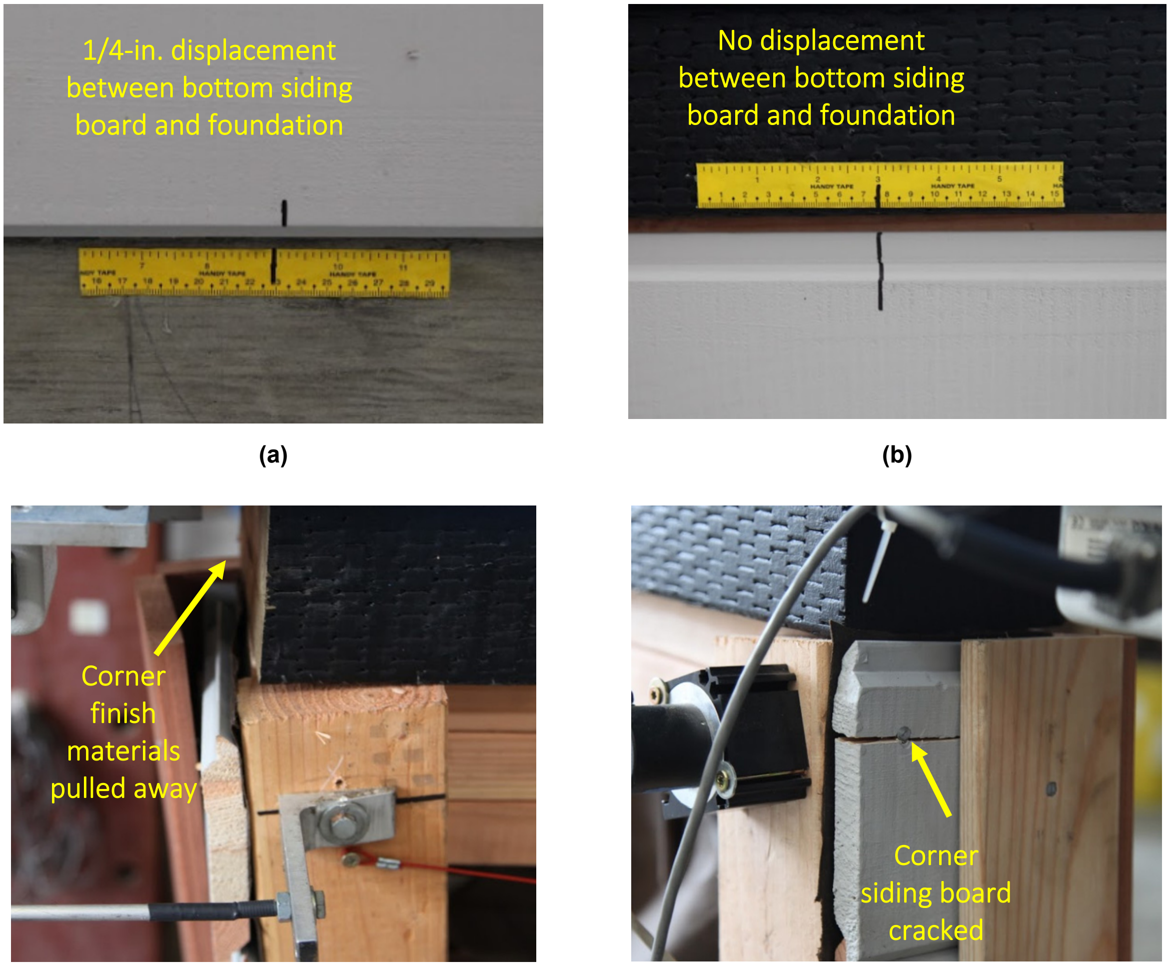
4.4 DAMAGE CHARACTERISTICS POST-STRENGTH

The damage state at 20% residual strength or an 80% drop in strength is important to show the state of the cripple wall near failure. Not all of the cripple walls dropped 80% in load, and those that did not will be noted in the subsections. When an 80% loss of strength in the cripple wall occurred, the loading protocol called for a monotonic push to be imposed for the subsequent drift amplitude. At this point, sufficient post-strength and residual strength characteristics had been defined for the wall. This section documents the evolution of damage from the displacement level after peak strength had occurred to displacement level the corresponded to an 80% drop in strength.

4.4.1 Specimen A-7 Post-Strength to Performance

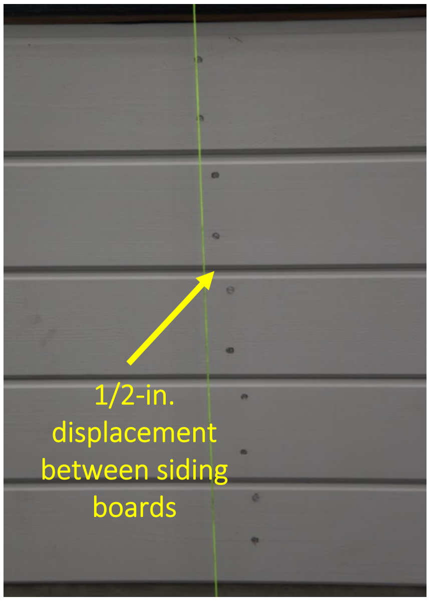
Figure 4.47 shows close-up photographs of the cripple wall at various post-strength drift ratio levels. The siding boards remained uncracked on the face of the cripple wall throughout the entire test. At the corner, some siding boards on the top and bottom of the cripple wall split, as shown in Figure 4.47(d). Cracks to siding boards at the base of the wall were due to the bearing of the siding boards on the foundation. Figure 4.47(a) and (b) shows an almost negligible displacement of the bottom siding board to the foundation and the top siding board to the uppermost top plate. This is due to the low capacity of the cripple wall. The displacement of the siding boards relative to each other was constant at each interface and reached nearly 1/2 in. by 12% drift; see Figure 4.47(e). The top siding board pushed away the corner trim boards as the cripple wall displacement increased, as seen in Figure 4.47(a). Large tears continued to propagate across the building paper as the displacement increased.

Figure 4.48 shows photographs of the cripple wall in the residual state after a monotonic push to +5.0 in. (+20.8% drift ratio). Specimen A-7 never dropped below 80% in strength. From 4% drift where strength occurred, to 12% drift where the monotonic push was initiated, the load only dropped 25.4% in the push direction and 22.2% in the pull direction. When the monotonic push to +5.0 in. (+20.8% drift ratio) was initiated, the capacity of the cripple wall came within 3.6% of its peak load. This is due to the increased resistance of the nail couples fastening the siding boards to the framing. The residual state of the cripple wall had a displacement of +4.78 in. (+19.9% drift ratio).

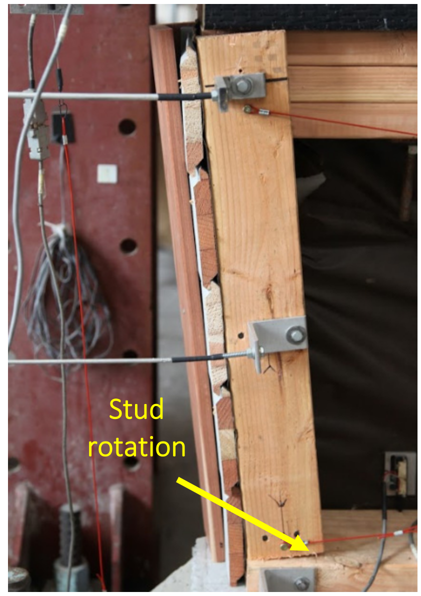


(a) (b) (c) (d)

Figure 4.47 (a) bottom exterior of wall view at +9% drift; (b) top exterior of wall view at +9% drift; (c) top of south-end interior of wall view at +11% drift; (d) top of north-end interior of wall corner view at +11% drift; (e) middle exterior of wall view at -12% drift; and (f) south-end interior of wall view at +11% drift.



(e)



(f)

Figure 4.47 (continued).

(a)

(b)

Figure 4.48 Specimen A-7 post-test photographs with lateral load = 0 kips, residual displacement = +4.78 in. @ +19.9% drift ratio after monotonic push to +5.0 in. @ +20.8% drift ratio: (a) exterior elevation; (b) interior elevation; (c) south-end exterior of wall view; and (d) north-end interior of wall view.

(c) (d)

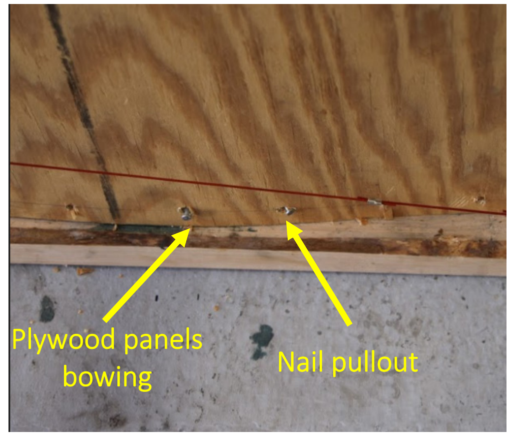
Figure 4.48 (continued).

4.4.2 Specimen A-8 Post-Strength to Performance

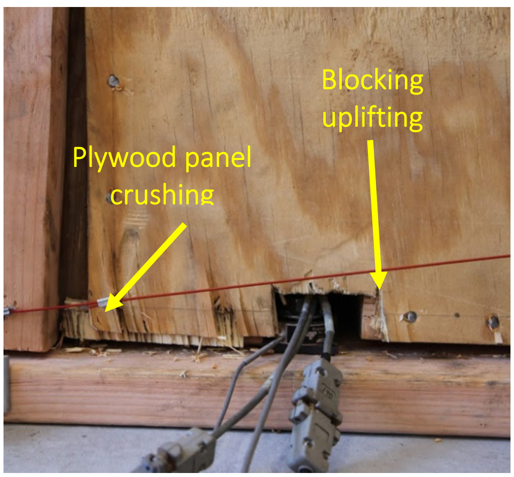
Figure 4.49 shows close-up photographs of the cripple wall at various post-strength drift ratio levels. The cripple wall achieved strength at 8% global drift ratio in the push direction and 7% global drift ratio in the pull direction. It should be noted that the lateral loads in either direction from 7% to 8% were within 1% of each other. These loads dropped by 40% by 13% drift and dropped by 80% by 15% drift. The drop of loads occurred as the nails attaching the plywood to the framing had torn through the plywood or pulled out from the framing. The most common scenario was for the nails to pull through the plywood or tear through the edges of the plywood and remain attached to the framing, as seen in Figure 4.49(b), (c), and (d). The plywood panel would remain attached to the top plate [Figure 4.49(a) and loss connection to the blocking at the sill plate and the studs. Once multiple edges of the plywood had detached from the framing, the capacity of the plywood dramatically decreased. The bottom of the corner plywood panels pushed out the corners of the cripple walls by 1 in. on the north end, as seen in Figure 4.49(d). The plywood panels bowed out in the middle [Figure 4.49(b)] and crushed at the corners; see Figure 4.49(c) and (d). On the exterior face of the cripple wall, the displacement between the siding boards increased consistently with increasing displacement, reaching 1/2 in. by 15% drift, as shown in Figure 4.49(f). The corner trim boards remained uncracked.

Figure 4.50 shows photographs of the cripple wall in the residual state after a monotonic push to +5.0 in. (+20.8% drift ratio). The monotonic push was initiated after the 15% drift ratio cycle group, which is where an 80% drop in strength occurred. The residual state of the cripple wall had a displacement of +4.72 in. (+19.7% drift ratio).

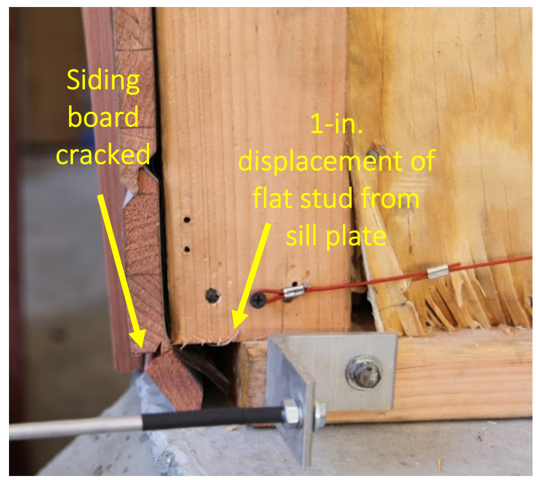
(a)



(b)



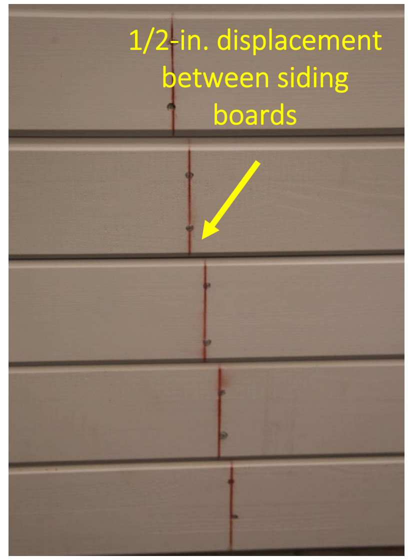
(c)



(d)

Figure 4.49 **Specimen A-8 damage state details at various post-strength drift ratio levels: (a) top interior of wall view at -9% drift (south and middle panels); (b) bottom interior of wall view at +10% drift (south and middle panels); (c) bottom of south-end interior of wall view at -11% drift; (d) bottom of south-end interior of wall view at +12% drift; (e) bottom of south-end of wall view at +13% drift; and (f) middle exterior of wall view at -15% drift.**

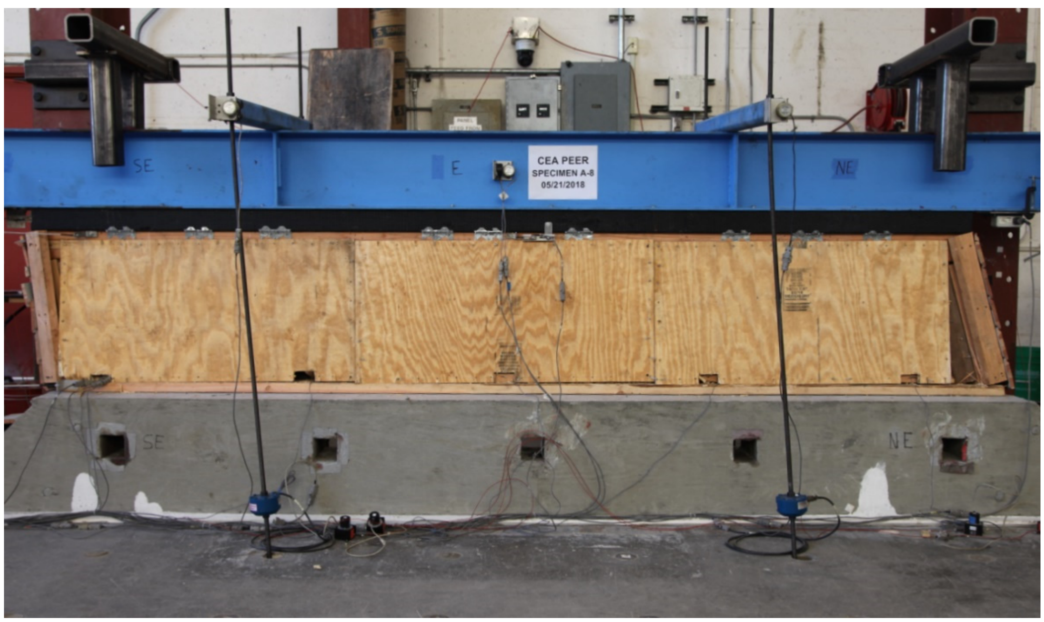
(e)



(f)

Figure 4.49 (continued).

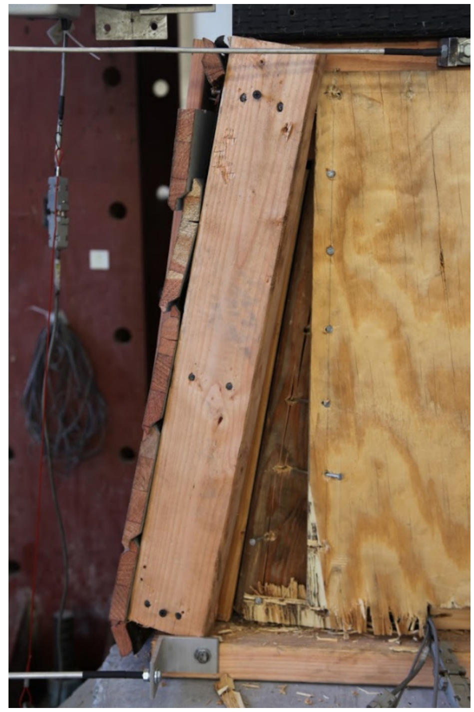
(a)



(b)

Figure 4.50 Specimen A-8 post-test photographs with lateral load = 0 kips at residual displacement = +4.72 in. @ +19.7% drift ratio after monotonic push to +5.0 in. @ +20.8% drift ratio: (a) exterior elevation; (b) interior elevation; (c) south-end of wall corner view; and (d) north-end interior of wall view.

(c)



(d)

Figure 4.50 (continued).

4.4.3 Specimen A-9 Post-Strength to Performance

Figure 4.51 shows close-up photographs of the cripple wall at various post-strength drift ratio levels. The cripple wall reached strength at 7% drift in the push direction and 10% drift in the pull direction. From 8% to 10% drift, the lateral strength in the push direction dropped 4.3%; by the end of the test, the push strength only dropped 21.3%. The peak load in the push direction dropped 6.4% from 10% to 12% drift. The cripple wall maintained close to peak load in both directions until the anchor bolts fractured, resulting in massive drops of capacity. If there had been additional anchorage, the specimen would have likely held close to peak load for larger displacement amplitudes, as indicated by the sheathing boards still being mostly intact when the anchor bolts had all fractured. The loss of capacity is a result of cracks forming in the sheathing boards as well as mobilization of the nails attaching the sheathing to the framing. Cracking of the sheathing boards can be seen in Figure 4.51(d), (e), and (g). Instead, the anchor bolts fractured, and a large crack formed in the north end of the sill plate; see Figure 4.51(e), (g), and (h). There was significant uplift experienced by the cripple wall, which opened up gaps between the studs and the sill plate as well as the studs and the top plates [Figure 4.51(b), (c), and (d)] and caused the sill plate to uplift 1/4 in.; see Figure 4.51(a). Specimen A-9 exhibited the most visible damage to the exterior face of all the cripple walls containing horizontal siding. In Figure 4.51(f), a 1/2 in. gap between the top two and bottom two siding boards had accumulated by +11% drift. The corner trim boards remained intact.

Figure 4.52 shows photographs of the cripple wall in the residual state after the reaching the target displacement in the pull direction of the 13% drift ratio cycle group. The decision was made to end the test at this point due to the fracturing of all anchor bolts, which subsequently resulted in an 80% reduction in capacity for the specimen. The residual state of the cripple wall had a displacement of -2.61 in. (-10.9% drift ratio).

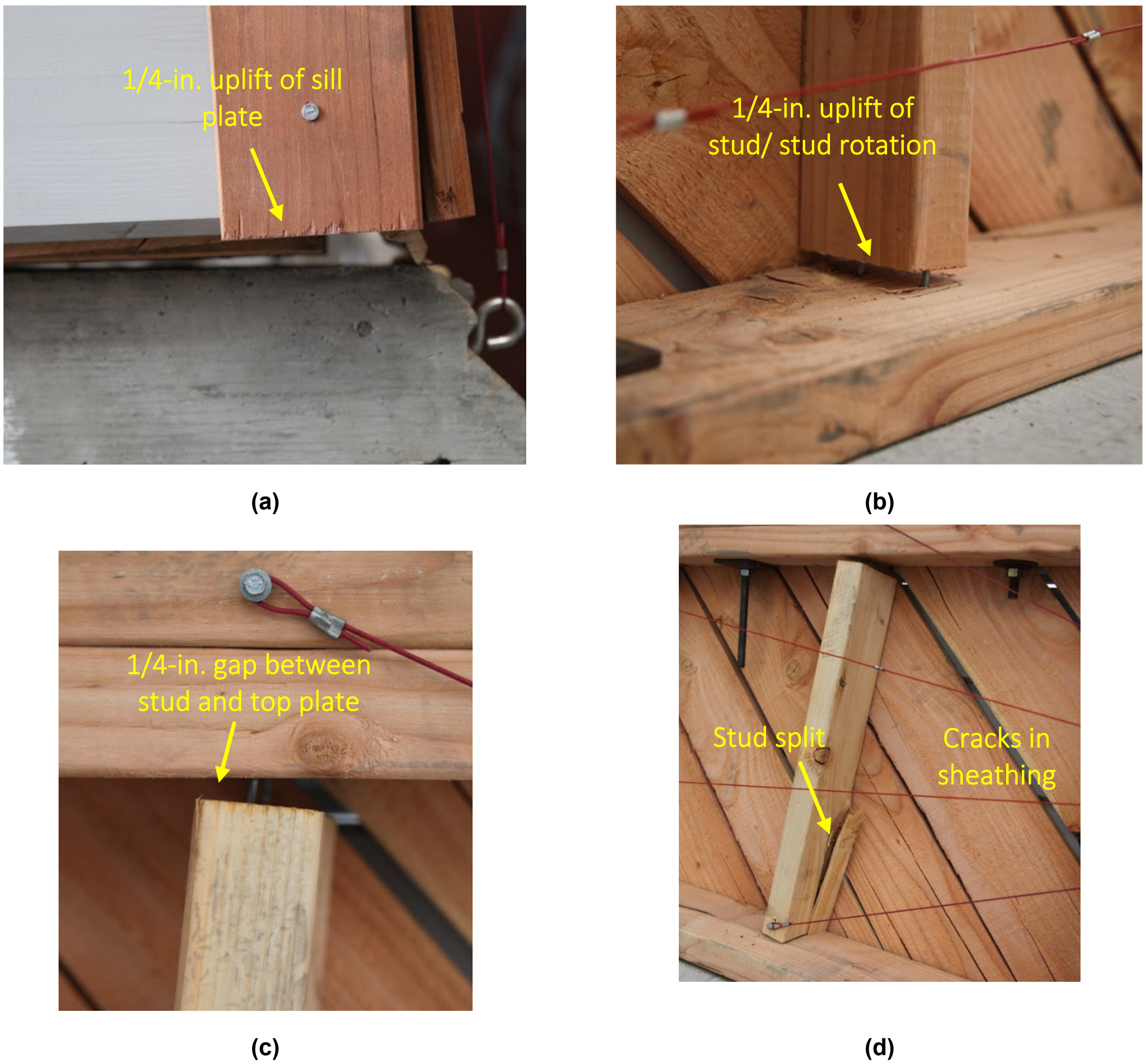
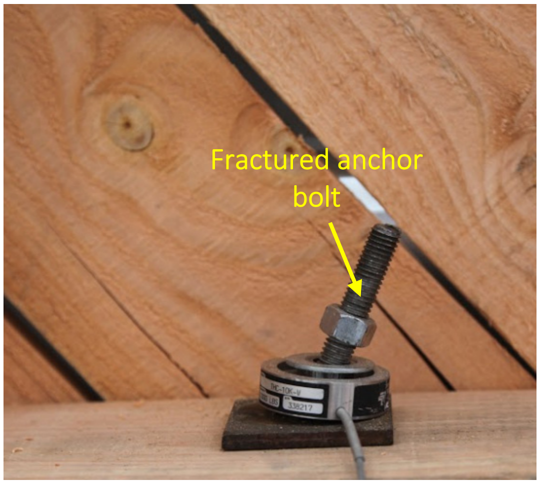
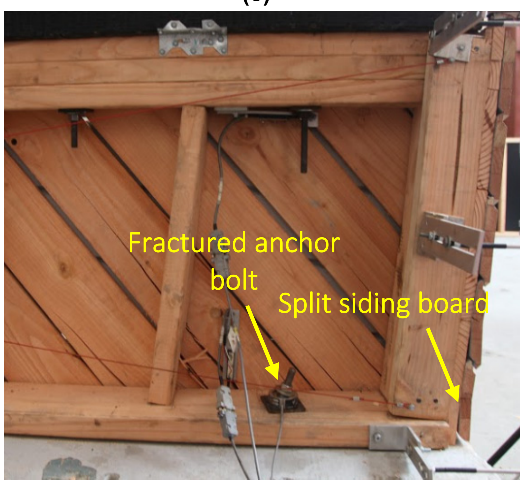


Figure 4.51 Specimen A-9 details of damage states at various post-strength drift ratio levels: (a) bottom of north-end exterior of wall view at +9% drift; (b) bottom of middle interior of wall view at +9% drift; (c) top of middle interior of wall view at +9% drift; (d) middle interior of wall view at +10% drift; (e) bottom of middle interior of wall view at +11% drift; (f) middle exterior of wall view at +11% drift; (g) south-end interior of wall at +12% drift; and (h) bottom of north-end interior of wall at +12% drift.

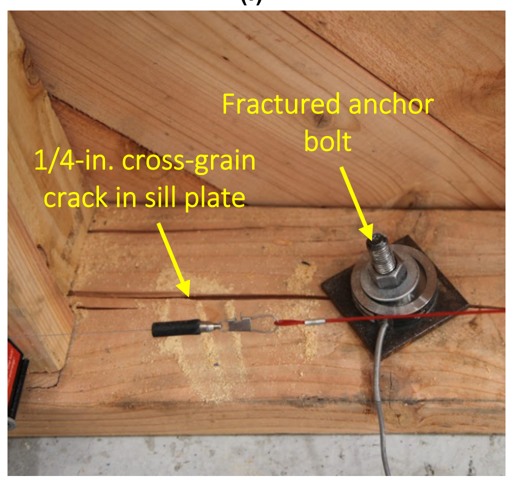


(e)

(f)

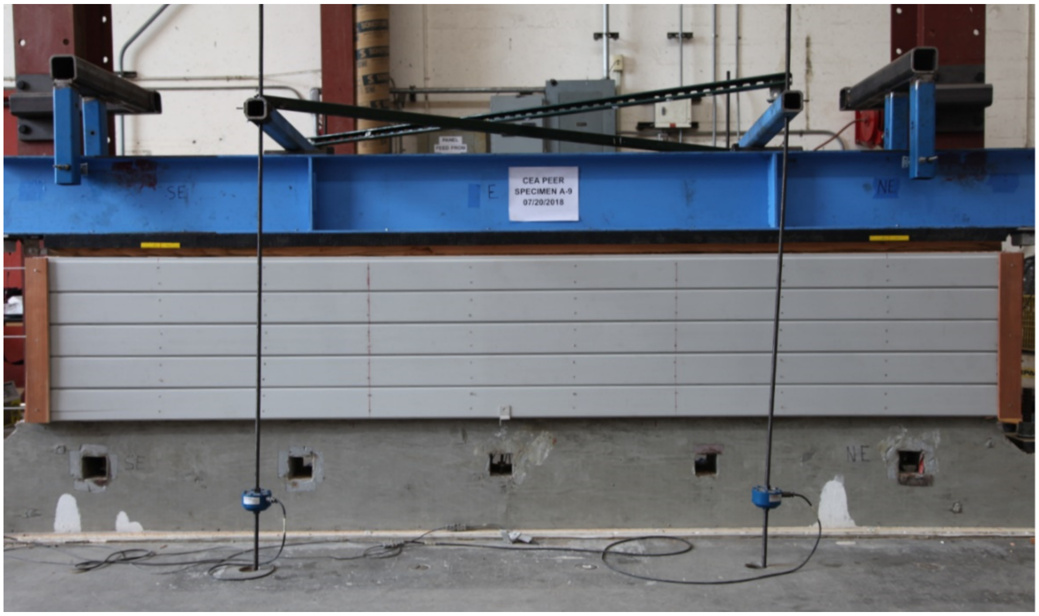


(g)



(h)

Figure 4.51 (continued).



(a)

(b)

Figure 4.52 **Specimen A-9 post-test photographs with lateral load = 0 kips at residual displacement = -2.61 in. @ -10.9% drift ratio after finishing first cycle of -13% drift ratio cycle group: (a) exterior elevation; (b) interior elevation; (c) south-end exterior of wall corner view; and (d) north-end interior of wall corner view.**

(c) (d)

Figure 4.52 (continued).

4.4.4 Specimen A-10 Post-Strength to Performance

Figure 4.53 shows close-up photographs of the cripple wall at various post-strength drift ratio levels. The cripple wall reached peak load at 7% drift in the push direction and 8% drift in the pull direction. From 6% to 9% drift, the lateral load in the push direction increased 3.4% to achieve peak load and then decreased 1.8%, and the lateral load in the push direction increased 3.7% to achieve peak load and then decreased 9.5%. From 9% to 12% drift, the peak push load decreased 83%, and the peak pull load decreased 84%. The sudden massive drops of capacity are a result of the anchor bolt fractures on the specimen. At 8% drift, the first anchor bolt fractured. At 9%, four more anchor bolts fractured, and at 10%, the last two anchor bolts fractured. Looking at Figure 4.54, both the exterior and interior of the cripple walls remain relatively intact, showing little physical damage compared to other cripple walls at these drift ratio levels. The cripple wall itself had not reached peak capacity as evidenced by the small amount of rotation in the nails fastening the plywood to the framing; see Figure 4.53(c) and (d). The exterior face of the cripple wall shows around 1/8 in. of displacement between the siding boards, as seen in Figure 4.53(b). Typically, at this drift ratio level, the horizontal siding boards were displaced by around 1/2 in. between each other. By 12% drift, the cripple wall was sliding on top of the foundation. Only 0.5% of the imposed drift was being carried by the cripple wall itself. The average lateral load at 12% drift was 4.85 kips, which can be taken as the amount of force required to overcome to frictional resistance at wood–concrete interface of the sill plate and the foundation. If sufficient anchorage had been present, it would be expected that the peak strengths would be significantly larger than what were experienced. In addition, Specimen A-10 would likely start to show asymmetry due to the orientation of the sheathing boards, as was observed in Specimen A-9. Had sufficient anchorage been available, the combination of exterior finish material and the plywood retrofit would likely have caused the sill plate to split before the sheathing materials lost capacity.

Figure 4.54 shows photographs of the cripple wall in the residual state after the reaching the target displacement in the pull direction of the 12% drift ratio cycle group. The decision was made to end the test at this point due to the fracturing of all anchor bolts, which subsequently results in an 80% reduction in capacity for the specimen. The residual state of the cripple wall had a displacement of -2.81 in. (-11.7% drift ratio).

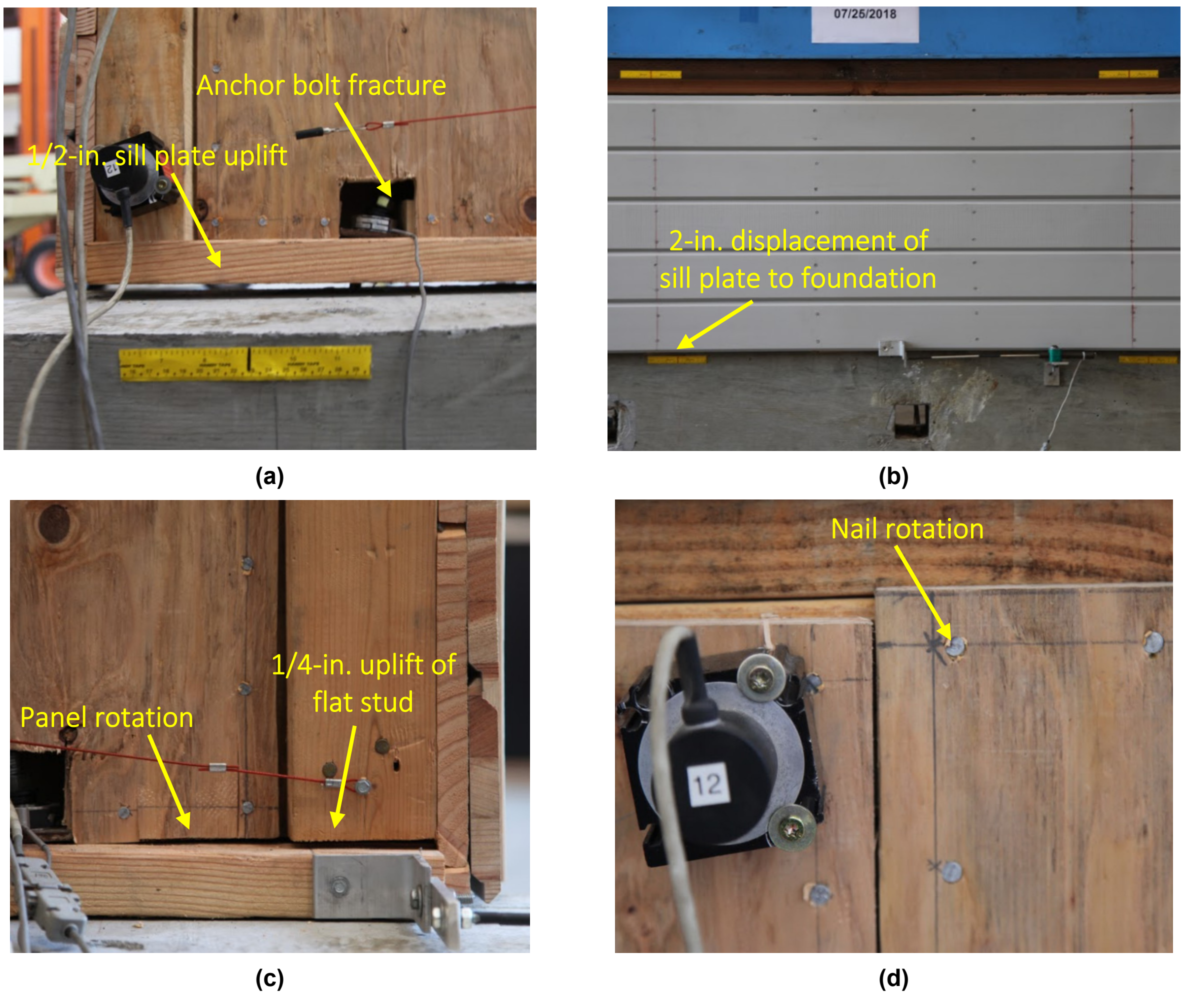


Figure 4.53 Specimen A-10 damage state details at various post-strength drift ratio levels: (a) bottom north-end exterior of wall view at +9% drift; (b) middle interior of wall view +9% drift; (c) bottom of south-end interior of wall view at +10% drift; and (d) top of middle interior of wall view at +10% drift (south and middle panels).

(a)

(b)

Figure 4.54 **Specimen A-10 post-test photographs with lateral load = 0 kips at residual displacement = -2.81 in. @ -11.7% drift ratio after finishing first cycle of -12% drift ratio cycle group: (a) exterior elevation; (b) interior elevation; (c) south-end exterior of wall corner view; and (d) north-end interior of wall view.**

(c)

(d)

Figure 4.54 (continued).

4.4.5 Specimen A-11 Post-Strength to Performance

Figure 4.55 shows close-up photographs of the cripple wall at various post-strength drift ratio levels. The cripple wall reached strength at 6% drift in the push direction and 5% drift in the pull direction. From 4% to 6% drift, the lateral load in the push and pull directions remained consistent with a 6.2% increase in lateral push load and the same lateral pull load (3.1% increased to peak load and then a 3.1% decrease from drift a peak load to 6% drift). After 6% drift, the cripple wall lost 62.3% of its load in the push direction and 61.9% of its load in the pull direction by 9% drift. By 13% drift, there was an 80% decrease in peak strength for both directions of loading. The loss of capacity was a result of the detachment of the T1-11 panels. From all photographs in Figure 4.56, it can be seen that the bottom of the plywood panels has fully detached from the sill plate and skirt outward. The only edge that remains fully attached is the top of the panels to the top plate. Cracks formed on the corner trim boards as the outward movement of the T1-11 panels increased. In addition, a crack formed in the sill plate at the location of the nailing of the T1-11 panel to the sill plate at the north end of the wall.

Figure 4.57 shows photographs of the cripple wall in the residual state after a monotonic push to +5.0 in. (+20.8% drift ratio). The monotonic push was initiated after the 13% drift ratio cycle group, which is where there was an 80% drop in strength. The residual state of the cripple wall had a displacement of +4.72 in. (+19.7% drift ratio).

(a) (b) (c) (d)

Figure 4.55 Specimen A-11 damage details for post-strength drift ratio levels: (a) south-end exterior of wall corner view at -9% drift; (b) exterior view of wall looking south at -9% drift; (c) bottom of middle interior of wall view at -9% drift; and (d) bottom of north-end exterior of wall corner view at +11% drift.

(a)

(b)

Figure 4.56 Specimen A-11 post-test photographs with lateral load = 0 kips at residual displacement = +4.72 in. @+19.7% drift ratio after monotonic push to +5.0 in. @ +20.8% drift ratio: (a) exterior elevation; (b) interior elevation; (c) north-end exterior of wall corner view; and (d) south-end of wall corner view.

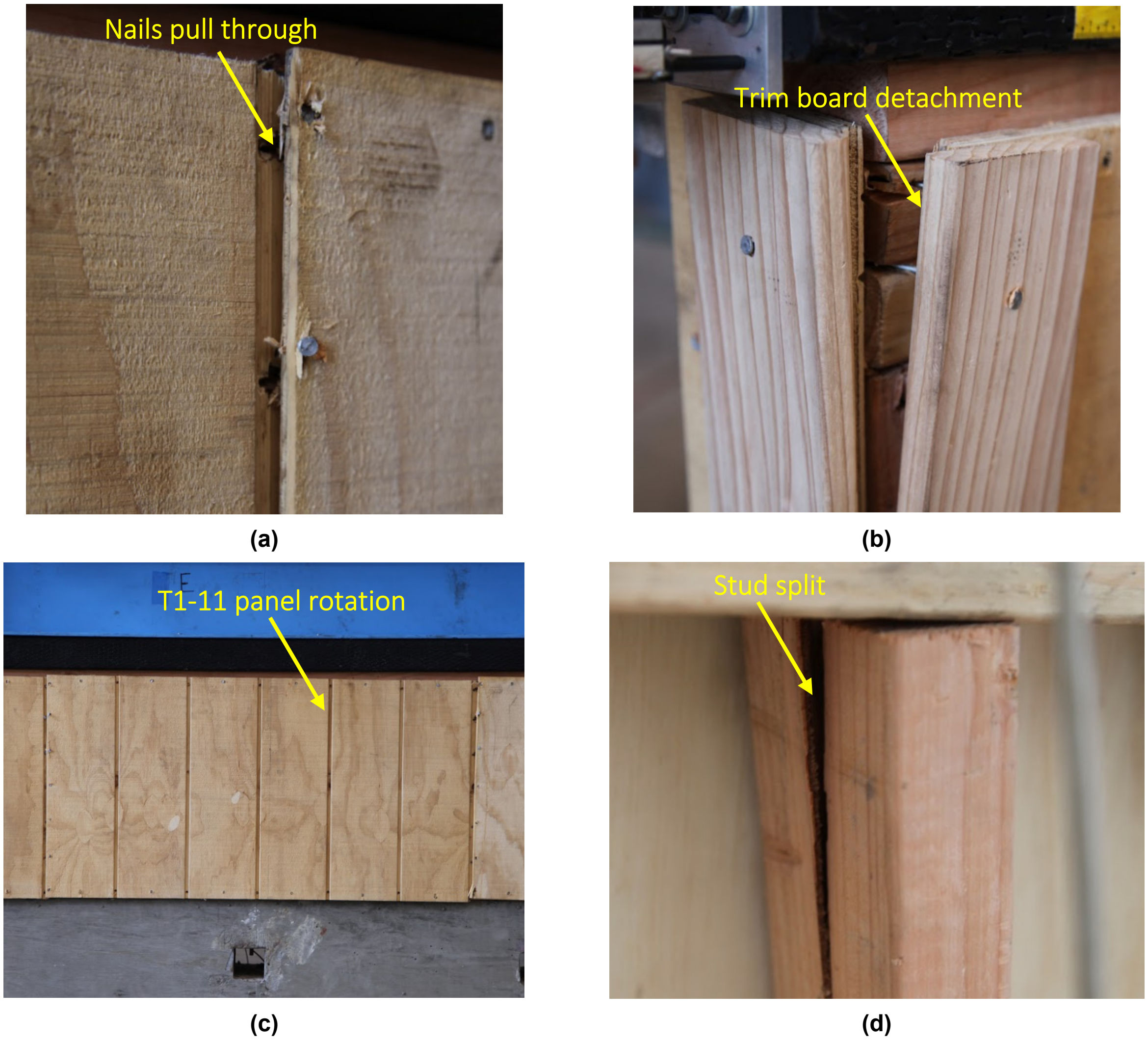
(c) (d)

Figure 4.56 (continued).

4.4.6 Specimen A-12 Post-Strength to Performance

Figure 4.57 shows close-up photographs of the cripple wall at various post-strength drift ratio levels. The cripple wall reached strength at 7% drift in the push direction and 6% drift in the pull direction. From 5% to 8% drift, the lateral load in the push direction increased 3.6% to achieve peak load and then decreased 3.5%; the lateral load in the push direction increased 3.5% to achieve peak load and then decreased 7.9%. By 10% drift, the loads had dropped 23.5% in the push direction and 40.2% in the pull direction. Each push and pull from 10% drift onward caused a large drop of capacity until the cripple wall lost 82% of its capacity by 12% drift. The peak load of the monotonic push to 20% drift was only 10% of the peak strength (1.43 kips). This is the lowest lateral load for a monotonic push of any of the retrofitted cripple walls. The loss of load is a consequence of the T1-11 panels losing attachment to the cripple wall at most of the attachment locations. At 9% drift, the nails on the underlying panel had torn through the edge of the T1-11 plywood, as seen in Figure 4.57(a). The corner trim boards had also detached from the exterior face of the framing. By 11% drift, there was pronounced rotation of the panels as the nails had either rotated or fully lost connection between the T1-11 panels and the framing; see Figure 4.57(c). At 13% drift, the attaching of the T1-11 plywood to the top plates had almost fully withdrawn from the top plate, as seen in Figure 4.57(f). The corner trim board began to crack at this drift ratio level as well; see Figure 4.57(e). As with all retrofitted specimens that failed due to the loss of capacity in the actual cripple wall (no anchor bolt fractures), the loss of capacity occurred when multiple edges of the plywood detached from the framing.

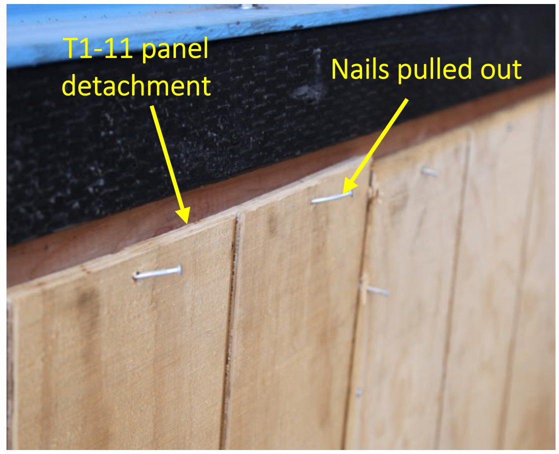
Figure 4.58 shows photographs of the cripple wall in the residual state after a monotonic push to +5.0 in. (+20.8% drift ratio). The monotonic push was initiated after the 12% drift ratio cycle group, which is where there was an 80% drop in strength from peak. The residual state of the cripple wall had a displacement of +4.12 in. (+17.2% drift ratio).



(a) (b) (c) (d)

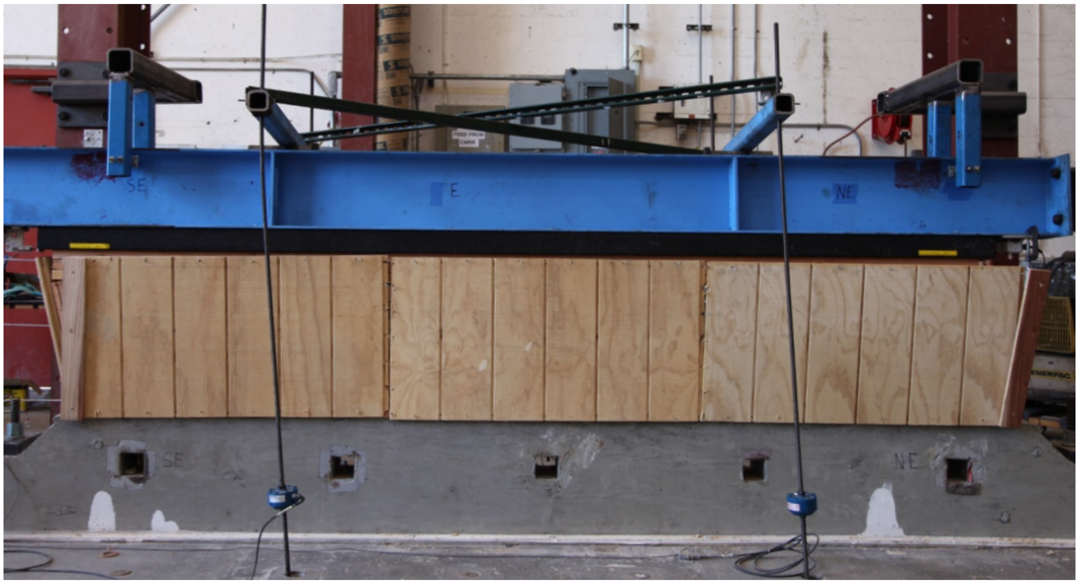
Figure 4.57 **Specimen A-12 damage details for post-strength drift ratio levels: (a) top exterior of wall view at -9% drift (north and middle panels); (b) top of south-end exterior of wall corner view at +9% drift; (c) middle exterior of wall view at -9% drift; (d) top of middle interior of wall view at -11% drift; and (d) top of north-end exterior of wall corner view at -11% drift; and top of middle exterior of wall view at -13% drift (south and middle panels).**

(e)

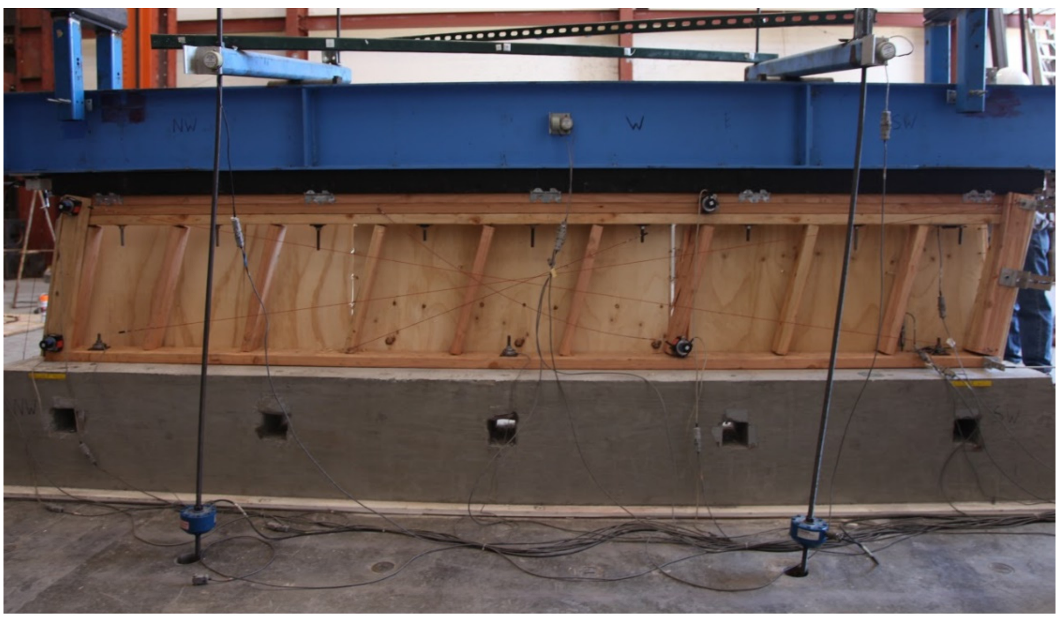


(f)

Figure 4.57 (continued).



(a)



(b)

Figure 4.58 **Specimen A-12 post-test photographs with lateral load = 0 kips at residual displacement = +4.12 in. @ +17.2% drift ratio after monotonic push to +5.0 in. @ +20.8% drift ratio: (a) exterior elevation; (b) interior elevation; (c) north-end exterior of wall corner view; and (d) south-end exterior of wall view.**

(c) (d)

Figure 4.58 (continued).

4.4.7 Specimen A-13 Post-Strength to Performance

Figure 4.59 shows photographs of the cripple wall in the residual state after a monotonic push to +15.0” in. (+20.8% drift ratio). The residual state of the cripple wall had a displacement of +14.3 in. (+19.9% drift ratio). The lateral strength in the push direction occurred at 11% drift, and the lateral strength in the pull direction occurred at 12% drift. From 11% to 12% drift, the peak push load decreased by 0.9%. The monotonic push was initiated after the 12% drift ratio cycle group. Because there was no significant post-strength loads for Specimen A-13, there will not be any further discussion.

(a)

(b)

Figure 4.59 **Specimen A-13 post-test photographs with lateral load = 0 kips at residual displacement = +14.3 in. @ +19.9% drift ratio after monotonic push to +15.0 in. @ +20.8% drift ratio: (a) exterior elevation; (b) interior elevation; (c) north-end exterior of wall corner view; and (d) north-end exterior of wall view.**

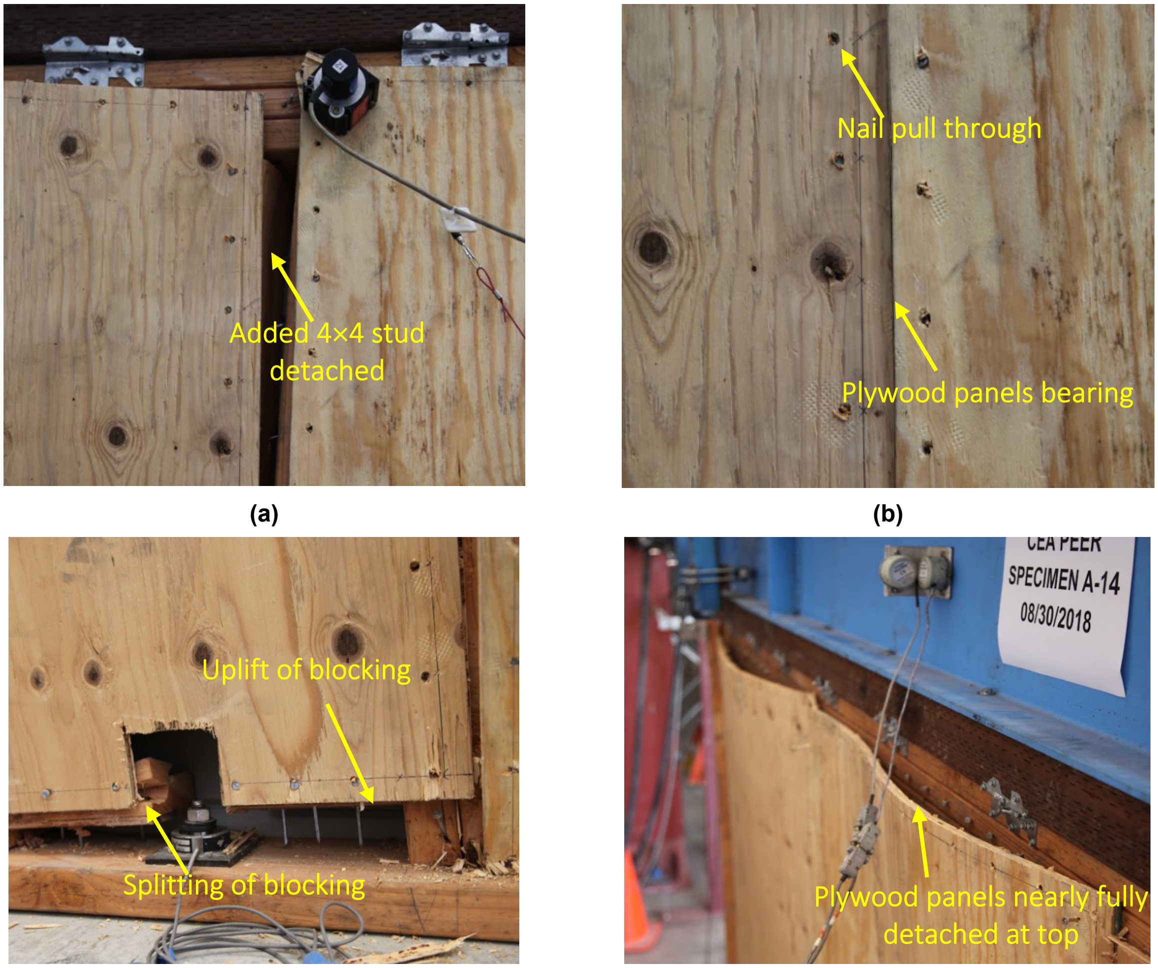
(c) (d)

Figure 4.59 (continued).

4.4.8 Specimen A-14 Post-Strength to Performance

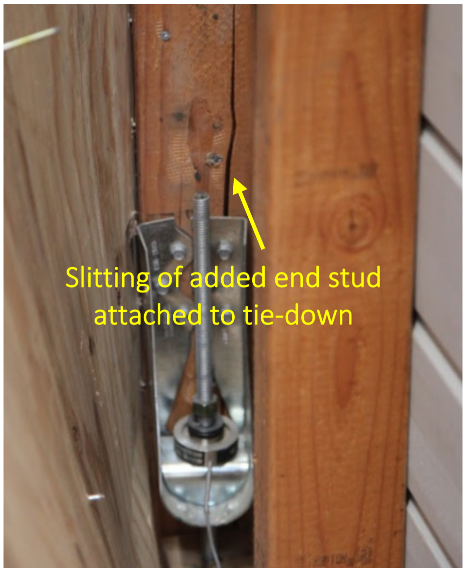
Figure 4.60 shows close-up photographs of the cripple wall at various post-strength drift ratio levels. The cripple wall reached peak load at 5% in the push direction and pull directions. From 5% to 6% drift, the lateral load in the push direction decreased by 8.6%, and the lateral load in the push direction decreased by 2.7%. By 8% drift, the push load had decreased 75.7% from peak, and the pull load had decreased 62.3%. These large drops of capacity from 6% to 8% drift are the result of the plywood nearly fully detaching from the top plate across the entire wall, as seen in Figure 4.60(b) and (d). The detachment occurred when the nails pulled through the plywood and remained attached to the framing. At the bottom of the cripple wall, the opposite occurred. The plywood panels remained attached to the nails. Significant uplift of the panels occurred as the blocking either split or was uplifted with the panels; see Figure 4.60(c) and (f). Along the studs there was a combination of nails pulling through the plywood and nails pulling out of the framing. The large amounts of uplift forces were carried in tension by the hold-downs on both ends. Eventually, this caused the stud attaching the hold-down to the framing to split, as seen in Figure 4.60(f).

Figure 4.61 shows photographs of the cripple wall in the residual state after a monotonic push to +13.0 in. (+18.1% drift ratio). The monotonic push was initiated after the 10% drift ratio cycle group, which is where an 80% drop in strength occurred. The residual state of the cripple wall had a displacement of +11.42 in. (+14.9% drift ratio).



(a) (b) (c) (d)

Figure 4.60 **Specimen A-14 damage details post-strength drift ratio levels: (a) top of middle interior of wall view at +6% drift (south and middle panels); middle interior of wall view at + 6% drift (north and middle panels); (c) bottom of south-end interior of wall view at -8% drift; (d) top of interior of wall view at -8% drift (north and middle panels); (d) bottom of north-end interior of wall view at +9% drift; and (f) bottom of south-end interior of wall view at +10% drift.**



(e)

(f)

Figure 4.60 (continued).

(a)

(b)

Figure 4.61 Specimen A-14 post-test photographs with lateral load = 0 kips at residual displacement = +11.42 in. @ +14.9% drift after monotonic push to +13.0 in. @ +18.1% drift: (a) exterior elevation; (b) interior elevation; (c) south-end exterior of wall corner view; and (d) south-end interior of wall corner view.

(c)

(d)

Figure 4.61 (continued).

4.4.9 Specimen A-23 Post-Strength to Performance

The lateral strength occurred at 3% drift in the push direction and 2% drift in the pull direction. From 3% drift to 10% drift, the loss in strength was nearly constant in each subsequent drift cycle group. After the 6% drift ratio level, less than 40% of the capacity of the wall remained. Figure 4.62 shows details of the damage accumulated post peak strength. All drift ratio levels are at 10% unless otherwise noted. Initially, the panels detached from the sill plate and then continued to detach up the studs. Eventually the detachment of the panels on the studs was far enough that the underlying panel had almost no contact with the overlying panel; see Figure 4.62(b). Most of the nails at the sill plate and near the base of the studs pulled through or were torn through the edges of the T1-11 panel. Higher up on the wall, more nails remained attached to the panels and had pulled out from the framing, as seen in Figure 4.62(c). The rotation of the studs at the interface with both the top plates and the sill plates had rotated heavily, both in and out of plane; see Figure 4.62(d). The trim boards at the finish face of the corner had almost fully detached from the framing and only remained partially fastened to the T1-11 panels; see Figure 4.62(e). There was no uplift of the sill plate due to the lack of connection between the sill plate and the panels; see Figure 4.62(f).

Figure 4.63 shows photographs of the cripple wall in the residual state after a monotonic push to +15.0” (+20.8% drift ratio). The monotonic push was initiated after the 10% drift ratio cycle group, which is where an 80% drop in strength occurred. The residual state of the cripple wall had a displacement of +14.33 in. (+19.9% drift ratio).

Figure 4.62 **Specimen A-23 damage details for post-strength drift ratio levels: (a) bottom interior of wall view at +10% drift (south and middle panels); (b) exterior isometric view of wall at -10% drift; (c) top exterior of wall view at -10% drift (south panel); (d) top of middle interior of wall view at -10% drift; (e) bottom of south-end of wall corner view at +10% drift; and (f) bottom of north-end interior of wall at +10% drift.**

(a)

(b)

Figure 4.63 Specimen A-23 post-test photographs with lateral load = 0 kips at residual displacement = +14.33 in. @ +19.9% drift ratio after monotonic push to +15.0 in. @ +20.8% drift ratio: (a) exterior elevation; (b) interior elevation; (c) north-end exterior of wall corner view; and (d) south-end of wall corner view.

(c)

(d)

Figure 4.63 (continued).

4.4.10 Specimen A-24 Post-Strength to Performance

By 10% drift, the panels were heavily rotated and had lost most of their attachment to the framing. On the interior face, the building paper had large tears. The lateral strength occurred at 3% global drift in the push direction and 2% global drift in the pull direction. The response was fairly symmetric in subsequent drift cycles. After the 4% drift ratio level, there were large drops in capacity until 6% drift, where only 40% of the capacity of the cripple wall remained. By 8% drift, less than 15% of capacity remained in the push direction and less than 10% in the pull direction. Figure 4.64 shows close-up photographs of the cripple wall at various post-strength drift ratio levels. All drift ratio levels are at 8% unless otherwise noted. Cracking of the sill plate extended to nearly two studs bays on both ends of the wall, as shown in Figure 4.64(a) and (b), where the separation of T1-11 panels from the sill plate is evident. The nails attached through the 5/8-in. sections of the panels generally pulled out from the framing while the nails attached through the 5/16-in. sections generally pulled through the panels or tore through the edges of the panels. At the panel overlaps, the T1-11 has thin sections where the fasteners had pulled through or torn through the panels in most instances, especially near the bottom of the wall; see Figure 4.64(f). Splitting of one of the studs occurred at 6% drift; see Figure 4.64(d). Previously, there had been visible bending of the studs, which is what led to the splitting of the stud.

Figure 4.65 shows photographs of the cripple wall in the residual state reaching the target drift in the pull direction of the 8% drift ratio cycle group. The decision was made to stop the test at this point as an 80% drop in strength had already occurred and sufficient post-strength behavior had been recorded. The residual state of the cripple wall had a displacement of -5.63 in. (-7.8% drift ratio).

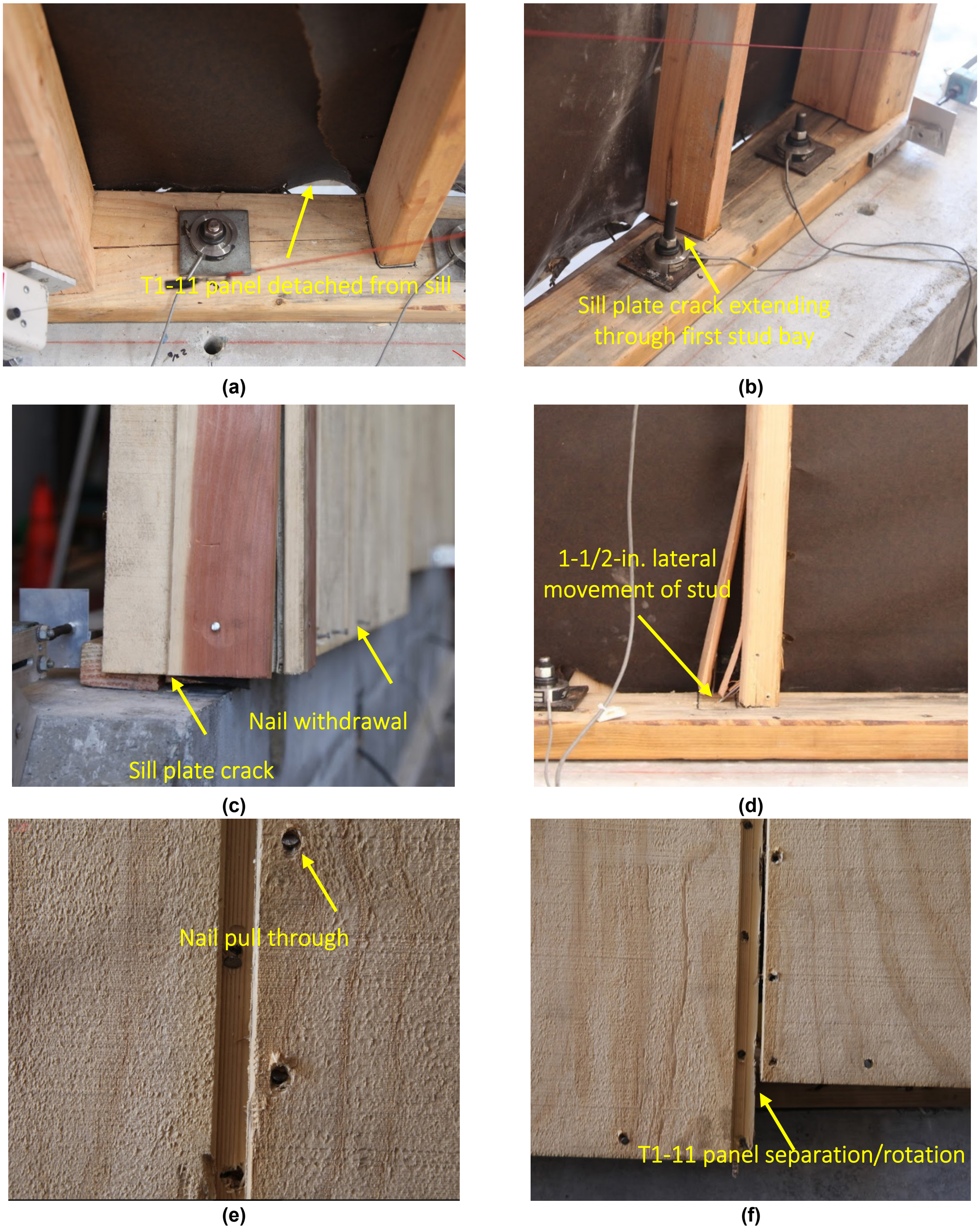


Figure 4.64 Specimen A-24 damage details for post-strength drift ratio levels: (a) bottom of south-end interior of wall view at +6% drift; (b) bottom of north-end interior of wall view at +6% drift; (c) bottom of south-end exterior of wall corner view at -6% drift; (d) bottom of middle interior of wall view at -6% drift; (e) middle exterior of wall view at -6% drift (south and middle panels); and (f) bottom exterior of wall view at -6% drift (north and middle panels).

(a)

(b)

Figure 4.65 **Specimen A-24 post-test photographs with lateral load = 0 kips at residual displacement = -5.63 in. @ -7.8% drift ratio after finishing the first cycle of the -8% drift ratio cycle group: (a) exterior elevation; (b) interior elevation; (c) north-end exterior of wall corner view; and (d) south-end of wall corner view.**

(c) (d)

Figure 4.65 (continued).

4.4.11 Specimen A-28 Post-Strength to Performance

Figure 4.66 shows close-up photographs of the cripple wall at various post-strength drift ratio levels. After achieving strength in the push direction (5% global drift ratio), the cripple wall consistently had small drops in capacity. During these cycles, the strength of the wall continued to increase in the pull direction, until it peaked at 7% drift. After 7% drift, drops in capacity began to occur due to damage accumulating in the sill plate. Figure 4.66(a) shows major uplift at the corner and cracking of the sill plate. More damage to the sill plate can be seen in Figures 4.66(e) and (f). At 10% drift, the entire sill plate was nearly split in half, resulting in major drops in capacity that eventually ended the test. Like all other cripple walls with horizontal siding over diagonal sheathing, the key factors contributing to loss of strength was not attributed the finish materials detaching from the framing. On the finish face of the specimen, a large gap formed between the top two siding boards, as shown in Figure 4.66(b).

Figure 4.67 shows photographs of the cripple wall in the residual state after a monotonic push to +3.12 in. (+13.0% drift ratio). At this point an 80% drop in strength had occurred. From the exterior face, the sheathing boards were visible at the top of cripple wall. From the interior face, the gaps between the sheathing boards had all closed, and there was visible bending of the sill plate. The residual state of the cripple wall had a displacement of +2.49 in. (+10.4% drift ratio).

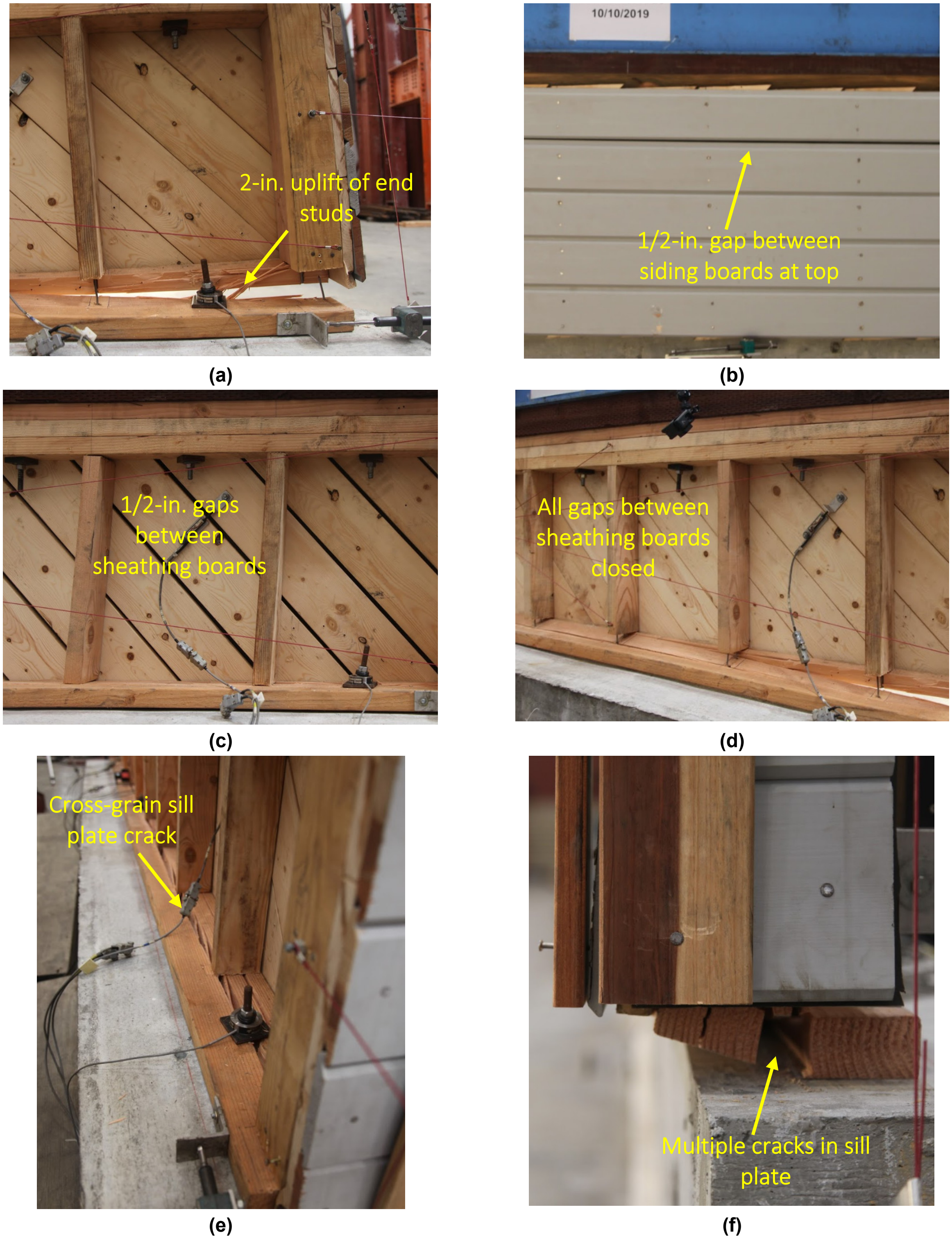


Figure 4.66 **Specimen A-28 damage state details post-strength drift ratio levels: (a) bottom of south-end interior of wall view at -9% drift; (b) top of middle exterior of wall view at +10% drift; (c) middle interior of wall view at +8% drift; (d) middle interior of wall view at -8% drift; (e) bottom of south-end interior of wall view at -10% drift; and (f) bottom of north-end of wall corner view at -10% drift.**

(a)

(b)

Figure 4.67 Specimen A-28 post-test photographs with lateral load = 0 kips at residual displacement = +2.49 in. @ +10.4% drift ratio after a monotonic push to +3.12 in. @ +13.0% drift ratio: (a) exterior elevation; (b) interior elevation; (c) north-end exterior of wall corner view; and (d) south-end of interior of wall view.

(c) (d)

Figure 4.67 (continued).

5 CONCLUSIONS

Quantifying the performance of retrofitted and existing single-family wood-frame houses has become increasingly important in California due to the high seismicity of the state and the often-poor seismic resiliency of the housing stock. From field observations of past earthquakes, it has been found 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 improvement in performance with retrofit have observed only limited experimental quantification. In addition, little knowledge is available to characterize the performance of houses with existing cripple walls and sill anchorages.

To this end, this report presents results from an experimental investigation of the seismic performance of retrofitted and existing cripple walls with sill anchorage, with particular focus on dry (non-stucco) finished specimens. Paralleled by a large-component test program conducted at UC Berkeley [Cobeen et al. 2020], the present report involves a portion of a multi-phased small component test suite conducted at UC San Diego. In the entire small-component test program, parameters examined are cripple wall height, finish style, gravity load, boundary conditions, and anchorage condition. This report specifically addresses the second and half of the fourth phases of testing, which consisted of eleven specimens, all finished with dry (non-stucco) materials. In addition to varying the type of dry finish materials, parameters examined in this report are the effectiveness of the *FEMA P-1100* prescriptive retrofit. The exterior finishes used were horizontal shiplap siding, horizontal shiplap siding over diagonal sheathing, and T1-11 plywood structural panels. The two cripple wall heights used were 2 ft and 6 ft. The anchorage and boundary conditions as well as cripple wall length were all held constant for each specimen. All specimens had the same heavy vertical load besides one, which was lightly loaded. Lastly, the same loading protocol was used for all tests discussed herein. In what follows, conclusions specific to the parameters varied within the present investigations are summarized.

5.1 IMPACT OF EXTERIOR FINISH

Horizontal Siding

- Horizontal siding was the weakest exterior finish material tested. Comparing the existing, 2-ft-tall specimens, the horizontal siding finished cripple wall had 30% of the lateral load capacity of the T1-11 plywood finished cripple wall (172 plf on average compared with 557 plf on average) and around 10% of the capacity of the horizontal siding over diagonal sheathing finished cripple wall (172 plf on average compared with 1435 plf on average). For the 6-ft-tall specimens, the horizontal siding finished specimen had 25% of the capacity of the T1-11 finished specimen (93 plf on average compared with 378 plf on average);

- Horizontal siding also exhibited the largest drift capacity of any exterior finish, with little to no lateral strength degradation. The 2-ft-tall specimen observed a reduction in load of 25% from peak, which occurred at 4% drift ratio, to 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 are nominally symmetric.

T1-11 Wood Structural Panels

- Cripple walls finished with T1-11 plywood panels were the second strongest exterior finish material tested; however, for a wood structural panel, the measured lateral strengths are relatively low. This can be attributed to the wide spacing used for the nailing of the T1-11 panels to framing, which was 8 in. on center compared with 3–4-in. on center typically used with plywood;
- 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 does not maintain its strength post-peak. This was a result of fasteners losing attachment via tearing and pull through; and
- The hysteretic response of cripple walls finished with T1-11 plywood panels are nominally symmetric.

Horizontal Siding over Diagonal Sheathing

- Horizontal siding over diagonal sheathing was 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 pull 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 of the anchor bolts;
- The horizontal siding over diagonal sheathing cripple wall has a secant stiffness associated with relative drift at 80% pre-lateral strength 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. The 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 difference in global and relative drift are due to the large amounts of sliding of the sill plate over 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 causing 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, a response 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 diagonal sheathing provides similar capacity as plywood.

5.2 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, but demonstrated that taller cripple walls are more flexible, shown by peak strength being achieved 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 6-ft-tall cripple walls when compared to the 2-ft-tall specimens. For existing cripple walls with horizontal siding exterior finishes, due to the low strength of the finish, a more appreciable contribution to strength is due to the framing. This led to the capacity of the 2-ft-tall cripple wall being almost 90% greater than that of the 6-ft-tall cripple wall. In the latter, the vertical studs of the taller cripple wall specimens were subjected to combined bending shear, compared with the short cripple wall specimens, whose behavior was more dominated by shear response. For the 6-ft-tall cripple wall finished with T1-11 panels, 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 (which was three times higher) caused the plywood to detach at a lower drift amplitude than the 2-ft-tall cripple wall. The same was the case for the T1-11 cripple walls, both existing and retrofitted.

5.3 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 horizontal siding cripple wall, accounting for more than 17 times increase in lateral strength. For the 2-ft-tall counterparts, there was more than 9.5 times increase. For T1-11 specimens, the strength increase was nearly 100% for the 2-ft-tall specimens, and 125% for the 6-ft-tall specimens;
- The lowest increase in secant stiffness associated with the relative drift at 80% pre-peak strength occurred with the T1-11 finished cripple walls. For the 2-ft-tall specimens, the increase was over 90% and over 50% for the 6-ft-tall specimens. The largest increase in secant stiffness occurred with the 6-ft-tall horizontal siding cripple wall, accounting for 2000% increase. For the 2-ft-tall specimens, the increase was drastically reduced to a 160% increase. There was a 125% increase for the horizontal siding over diagonal sheathing finished specimens;
- The drift at lateral strength was around 2.5% and was nearly identical for all retrofitted T1-11 finished specimens. For the horizontal siding finished specimens, the drift ratio at strength increased for the 2-ft-tall cripple walls and decreased 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 of the sill plate. Each retrofitted cripple wall experienced all of these phenomena, with the exception of the T1-11 cripple wall, which had no blocking.

5.4 VERTICAL LOAD

- Two different amplitudes of vertical loads were imposed on the existing 2-ft-tall 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;

- The peak uplift at the ends of the specimen was 3.5 times higher with the implementation of the light vertical load. It is noted, however, that the heavy vertical load cripple wall test concluded 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 modification in the dominant failure mode can be attributed to the reduced vertical load decreasing the uplift resistance.

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APPENDIX A MATERIAL PROPERTIES

Appendix A includes three sections: lumber moisture content readings (A.1), Specimen A-8 retrofit design calculations (A.2) and loading protocols (A.3). Discussion of these sections is provided in Chapter 2.

A.1 LUMBER MOISTURE CONTENT

The moisture content of the wood was recorded for all constructed cripple walls using an MD912 digital moisture meter, which is a pin meter with a resolution of 0.5% and accuracy of +/- 0.5%. A picture of the moisture content reader used for the project can be seen in Figure A.1. Understanding the moisture content of wood is important as drier wood has higher strength properties than fresh or moist wood. For all cripple walls, the moisture content was from 4–12% immediately prior to testing. The moisture content was read on five various places on a piece of lumber—top, bottom, middle, and sides—and then repeated on four additional pieces of the same type of lumber. The results were recorded and are displayed in Table A.1 for Phase 2 specimens and Table A.2 for Phase 4 specimens, along with the date of recording and averages for each type of lumber.

Figure A.1 **MD912 digital moisture content reader in use.**

Table A 1 Moisture content readings of Phase 2 lumber used in construction.

Construction Phase 2							
Lumber section	**Moisture contentreadings (%)**					**Average (%)**	**Date**
2×6 #2 Douglas Fir	16.2	18.3	20.2	15.4	22	18.42	3/9/2018
2×4 #2 Douglas Fir	19.6	21.4	23.2	19.4	19.9	20.70	3/9/2018
1×6 redwood shiplap siding	11.2	9.8	10.1	9.8	12	10.58	3/9/2018
1×6 construction Douglas Fir	16.5	22.4	16.2	17.7	22.4	19.04	3/9/2018

Test 7 – Specimen A-7							
Lumber section	**Moisture content readings (%)**					**Average (%)**	**Date**
2×6 #2 Douglas Fir	13.2	10.3	9.6	9.7	12.4	11.04	5/11/2018
2×4 #2 Douglas Fir	12.1	13.5	13.2	13	14.3	13.22	5/11/2018
1×6 redwood shiplap siding	5.6	8.4	8.4	6.8	7.4	7.32	5/11/2018
1×6 construction Douglas Fir	14.5	13.4	14.2	16.8	13.2	14.42	511/2018

Test 8 – Specimen A-8							
Lumber section	**Moisture content readings (%)**					**Average (%)**	**Date**
2×6 #2 Douglas Fir	13.5	11.5	11.9	11.1	13.5	13.50	5/22/2018
2×4 #2 Douglas Fir	13.5	13	12.8	12.8	14	13.22	5/22/2018
1×6 redwood shiplap siding	5.6	5.8	5.3	6.8	6.9	6.08	5/22/2018
1×6 construction Douglas Fir	15.6	17.2	14.5	16.2	16	15.90	5/22/2018

Test 9 – Specimen A-11							
Lumber section	**Moisture content readings (%)**					**Average (%)**	**Date**
2×6 construction grade redwood	6.7	5.8	6.9	4.3	5.5	5.84	10/31/2018
2×4 #2 Douglas Fir	8.6	8.4	9.7	9.9	9.5	9.22	10/31/2018
1×6 construction Douglas Fir	8.7	10.2	9.5	9.6	10.3	9.66	10/31/2018

Test 10 – Specimen A-12							
Lumber section	**Moisture content readings (%)**					**Average (%)**	**Date**
2×6 #2 Douglas Fir	10.8	12.1	10.7	9.9	11	10.80	12/18/2017
2×4 #2 Douglas Fir	12.2	8.6	10.1	9.8	10.2	10.18	12/18/2017
5/8 in. T-1-11 wood structural panel	16	15.2	14.9	14.7	14.5	15.06	12/18/2017

Test 11 – Specimen A-9							
Lumber Section	**Moisture Content Readings (%)**					**Average (%)**	**Date**
2×6 #2 Douglas Fir	13.6	10.9	11.4	11.7	12.6	12.04	6/28/2018
2×4 #2 Douglas Fir	7.8	8.6	10.1	5.5	6.6	7.72	6/28/2018
1×6 redwood shiplap siding	7.2	6.4	6.5	6.1	7	6.64	6/28/2018
1×6 construction Douglas Fir	12.4	4.5	4.9	7.1	6.2	7.02	6/28/2018

Test 12 – Specimen A-10							
Lumber section	**Moisture content readings (%)**					**Average (%)**	**Date**
2×6 #2 Douglas Fir	8.7	10.2	9.8	9.3	9.6	9.52	7/19/2018
2×4 #2 Douglas Fir	9.9	9.2	11.7	10.7	11.6	10.62	7/19/2018
1×6 redwood shiplap siding	5.5	5.8	5.7	6	5.7	5.74	7/19/2018
1×6 construction Douglas Fir	15.2	13.7	13.4	14.5	12.9	13.94	7/19/2018

Test 13 – Specimen A-13							
Lumber section	**Moisture content readings (%)**					**Average (%)**	**Date**
2×6 #2 Douglas Fir	10.7	11.2	10.8	10.8	12.1	11.12	8/6/2018
2×4 #2 Douglas Fir	13.2	13.8	13.4	12.8	14.2	13.48	8/6/2018
1×6 redwood shiplap siding	6.4	6.9	7.2	6.2	7	6.74	8/6/2018
1×6 construction Douglas Fir	16.2	16.8	14.5	15.7	15.4	15.72	8/6/2018

Test 14 – Specimen A-14							
Lumber section	**Moisture content readings (%)**					**Average (%)**	**Date**
2×6 #2 Douglas Fir	9.8	9.7	10.6	10.4	10.1	10.12	8/30/2018
2×4 #2 Douglas Fir	12.2	11.7	11.6	12.3	10.8	11.72	8/30/2018
1×6 redwood shiplap siding	5.4	5	6.1	5.8	4.7	5.40	8/30/2018
1×6 construction Douglas Fir	14.2	13.8	13.9	14	14.5	14.08	8/30/2018

Table A.2 Moisture content readings of Phase 4 lumber used in construction.

Construction Phase 4							
Lumber section	**Moisture content readings (%)**					**Average (%)**	**Date**
2×6 #2 Douglas Fir	12.6	14.2	13.5	13	12.7	13.20	8/2/2019
2×4 #2 Douglas Fir	12.6	13.5	13.4	12.6	12.8	12.98	8/2/2019
1×6 redwood shiplap siding	10.4	9.8	7.9	9.6	8.8	9.30	8/2/2019
1×6 construction Douglas Fir	14.3	12.8	12.7	13.6	13.6	13.40	8/2/2019

Test 23 – Specimen A-23							
Lumber section	**Moisture content readings (%)**					**Average (%)**	**Date**
2×6 #2 Douglas Fir	9.7	10.2	9.8	9.6	10.3	9.92	9/16/2019
2×4 #2 Douglas Fir	9.5	9.6	10.2	8.9	9.2	9.48	9/16/2019
5/8 in. T-1-11 wood structural panel	6.5	5.8	7.3	6.1	6	6.34	9/16/2019

Test 24 – Specimen A-24							
Lumber section	**Moisture content readings (%)**					**Average (%)**	**Date**
2×6 #2 Douglas Fir	8.9	10.1	8.5	9.3	9.4	8.90	10/3/2019
2×4 #2 Douglas Fir	8.7	9.6	9.4	9.6	8.9	9.24	10/3/2019
5/8 in. T-1-11 wood structural panel	4.6	5.6	5.1	6.2	4.8	5.26	10/3/2019

Test 25 – Specimen A-28							
Lumber section	**Moisture content readings (%)**					**Average (%)**	**Date**
2×6 #2 Douglas Fir	10.1	10.6	10.3	9.7	10.1	10.16	10/10/2019
2×4 #2 Douglas Fir	8.6	9.5	9.4	9.6	9.1	9.24	10/10/2019
1×6 redwood shiplap siding	8.9	9.7	9.6	9.3	8.6	9.22	10/10/2019
1×6 construction Douglas Fir	10.6	11.2	10.4	9.7	10.2	10.42	10/10/2019

A.2 SPECIMEN A-8 RETROFIT DESIGN CALCULATIONS

The following section describes how the retrofit design for Specimen A-8 was determined.

Test Description

- 12 ft × 2 ft cripple wall with horizontal siding exterior finish, heavy vertical load (450 plf)

Retrofit Design Parameters

- ATC-110 median home (two-story house with 30 ft × 40 ft floor plan), 1200 square feet per floor, 2400 square feet total
- Heavy building materials
  - Horizontal siding exterior finish
  - Plaster on wood lath interior finish
  - Concrete tile and plaster on wood roof
- ASCE 7 design parameter:
  - $R = 3$
  - $\Omega_0 = 2$
  - $S_{DS} = 1.0$g

Seismic weight

Note: seismic weight values are derived from the ATC-110 unit-weight spread sheets.

Table A.3 Seismic weight calculation for Specimen A-8.

House component	Mateial description	Material weight (psf)	Material area (sq. ft.)	Seismic weight (kips)
Exterior finish	Wood siding + plaster on lath	13	(2 stories)(8 ft)(30 ft + 40 ft)(2)(0.85) = 1904	24.8
Interior finish	Plaster on wood lath	18	(2 stories)(8 ft)(30 ft + 40 ft)= 1120	20.2
Roofing	Concrete tiles	29	(30 ft + 2 ft)(40 ft + 2 ft) = 1344	39.0
1st Floor	Heavy materials	19	(30 ft x 40 ft) = 1200	12.0
2nd Floor	Heavy materials	10	(30 ft x 40 ft) = 1200	12.8
Seismic Weight = 118.8 kips				

Seismic design – ASD

$$E = \frac{S_{DS}}{R}W = \frac{1.0}{3}(118.8\ kips) = 39.6\ kips$$
$$\Rightarrow\ 0.7E = 27.7\ kips\ or\ 13.86\ \frac{kips}{cripple\ wall}$$

Wood structural panel sheathing

Using 15/32 in. plywood w/ 8d common nails at 3 in. o.c. E.N. and 12 in. o.c. F.N which has retrofit design capacity of 980 plf as per ATC-110:

$$\frac{V_n}{\Omega_0} = \frac{980\,plf}{2} = 490\,plf \;\Rightarrow\; Length\ Required = \frac{13860\,lbs}{490\,plf} = 28.3\,ft\ for\ 30\,ft\ wall$$

$$\therefore \boldsymbol{Fully\ sheath\ 12\ ft\ length}$$

Anchor bolts

$$V = 13.86\,kip * \Omega_0 = 27.7\,kips$$

$$with \quad \tfrac{1}{2}"\ \text{Ø}\ AB\ in\ D.F. \;\Rightarrow\; Z'_{II} = (860)(1.6) = 1.38\ {}^{kips}/_{AB} \;\Rightarrow\; 20\ AB\ for\ 40\ ft\ wall$$

$$\therefore \boldsymbol{Use\ 5\ anchor\ bolts\ for\ 12\ ft\ wall\ length\ to\ accommodate\ foundation\ geometry}$$

A.3 LOADING PROTOCOLS

The following section presents a graph and table of the loading protocol for the specimens considered in this report. Overall, the loading protocol started the same for each specimen. Variations occurred at later drift ratio levels depending on the rate of post-peak strength degradation of the individual specimen. As noted in Section 2.4, all cripple walls underwent the same loading protocol up until the specimen realized a loss greater than 60% of its measured lateral strength. At this point in the protocol, the following and each subsequent drift ratio level was increased by 2%, rather than 1%. If the 60% loss in strength did not occur, each drift ratio level would remain at an increase of 1% per cycle grouping. The loading protocol would progress until an 80% loss in strength was realized. At this point, a monotonic push would be conducted, typically to a global drift of 20%. The amplitude of the monotonic push might vary slightly depending on instrumentation constraints.

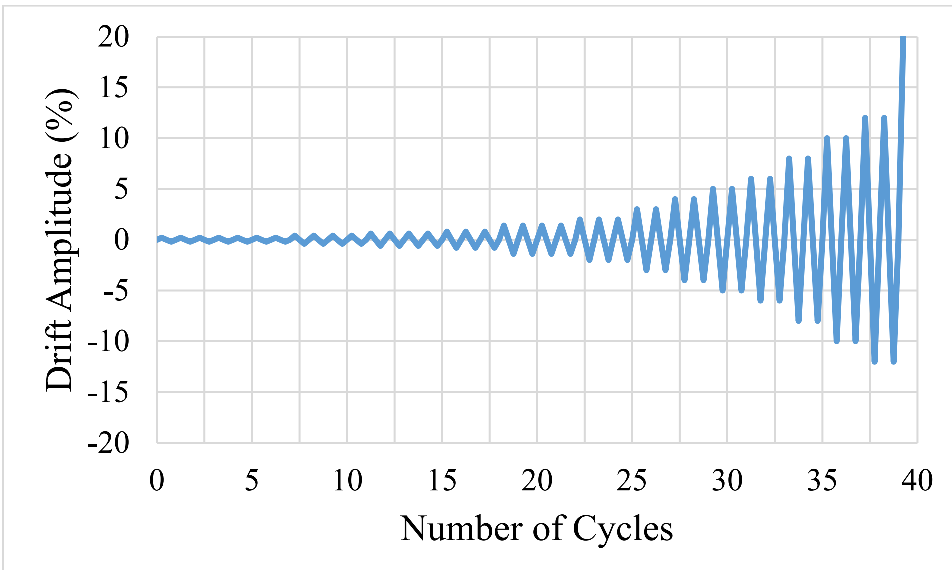


Figure A.2 Specimen A-7 loading protocol.

Table A.4 Specimen A-7 loading protocol.

Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loadig rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.048	7	0.0064	30	210
2	0.4	0.096	4	0.0128	30	120
3	0.6	0.144	4	0.0192	30	120
4	0.8	0.192	3	0.0256	30	90
5	1.4	0.336	3	0.0448	30	90
6	2	0.48	3	0.064	30	90
7	3	0.72	2	0.096	30	60
8	4	0.96	2	0.128	30	60
9	5	1.2	2	0.16	30	60
10	6	1.44	2	0.192	30	60
11	8	1.92	2	0.256	30	60
12	10	2.4	2	0.16	60	120
17	Mono	5.0	--	0.333	60	60

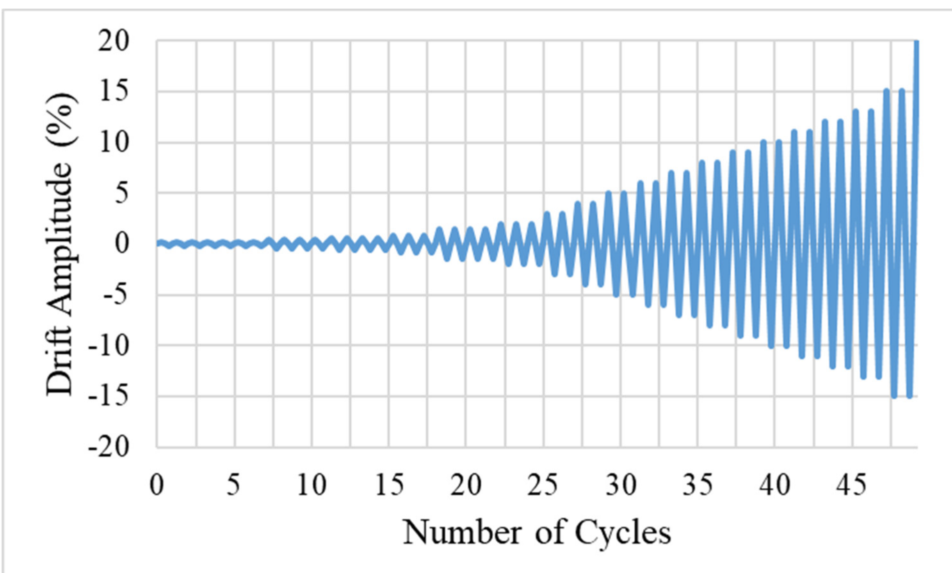


Figure A.3 Specimen A-8 loading protocol.

Table A.5 Specimen A-8 loading protocol.

Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loadig rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.048	7	0.0064	30	210
2	0.4	0.096	4	0.0128	30	120
3	0.6	0.144	4	0.0192	30	120
4	0.8	0.192	3	0.0256	30	90
5	1.4	0.336	3	0.0448	30	90
6	2	0.48	3	0.064	30	90
7	3	0.72	2	0.096	30	60
8	4	0.96	2	0.128	30	60
9	5	1.2	2	0.16	30	60
10	6	1.44	2	0.192	30	60
11	7	1.68	2	0.224	30	60
12	8	1.92	2	0.256	30	60
13	9	2.16	2	0.288	30	60
14	10	2.4	2	0.16	60	120
15	11	2.64	2	0.176	60	120
16	12	2.88	2	0.192	60	120
17	13	3.12	2	0.208	60	120
18	15	3.6	2	0.24	60	120
19	Mono	5.0	--	0.333	60	60

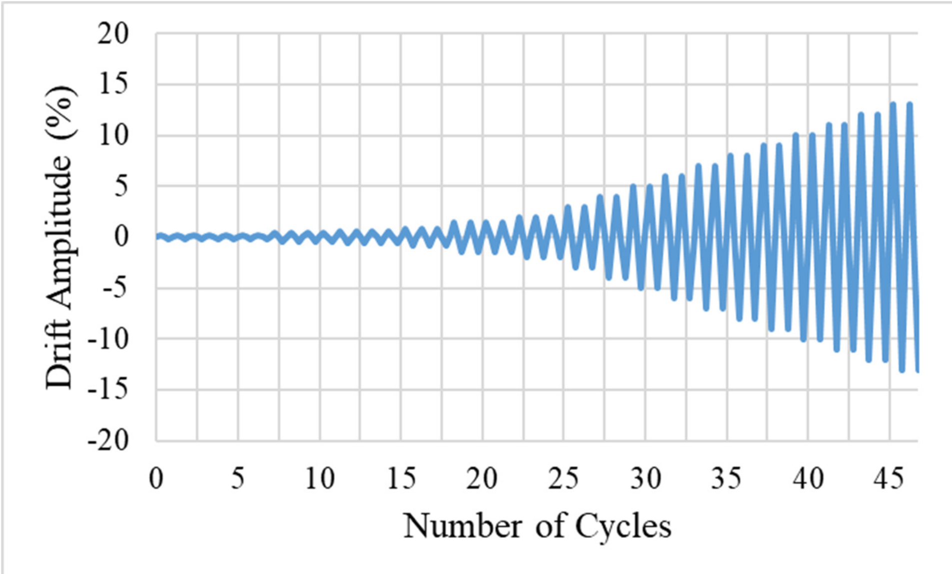


Figure A.4 Specimen A-9 loading protocol.

Table A.6 Specimen A-9 loading protocol.

Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loadig rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.048	7	0.0064	30	210
2	0.4	0.096	4	0.0128	30	120
3	0.6	0.144	4	0.0192	30	120
4	0.8	0.192	3	0.0256	30	90
5	1.4	0.336	3	0.0448	30	90
6	2	0.48	3	0.064	30	90
7	3	0.72	2	0.096	30	60
8	4	0.96	2	0.128	30	60
9	5	1.2	2	0.16	30	60
10	6	1.44	2	0.192	30	60
11	7	1.68	2	0.224	30	60
12	8	1.92	2	0.256	30	60
13	9	2.16	2	0.288	30	60
14	10	2.4	2	0.16	60	120
15	11	2.64	2	0.176	60	120

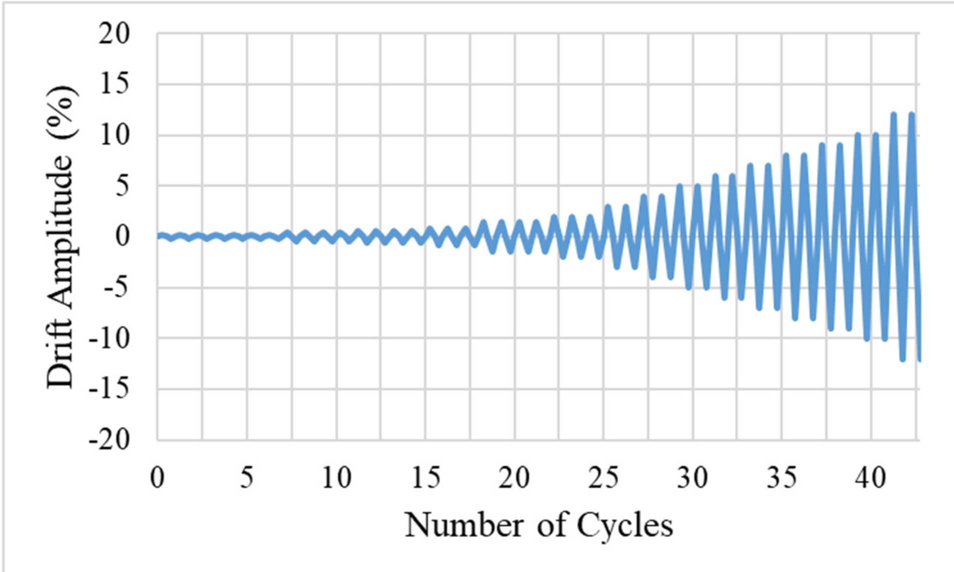


Figure A.5 Specimen A-10 loading protocol.

Table A.7 Specimen A-10 loading protocol.

Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loadig rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.048	7	0.0064	30	210
2	0.4	0.096	4	0.0128	30	120
3	0.6	0.144	4	0.0192	30	120
4	0.8	0.192	3	0.0256	30	90
5	1.4	0.336	3	0.0448	30	90
6	2	0.48	3	0.064	30	90
7	3	0.72	2	0.096	30	60
8	4	0.96	2	0.128	30	60
9	5	1.2	2	0.16	30	60
10	6	1.44	2	0.192	30	60
11	7	1.68	2	0.224	30	60
12	8	1.92	2	0.256	30	60
13	9	2.16	2	0.288	30	60
14	10	2.4	2	0.16	60	120
15	12	2.88	2	0.192	60	120

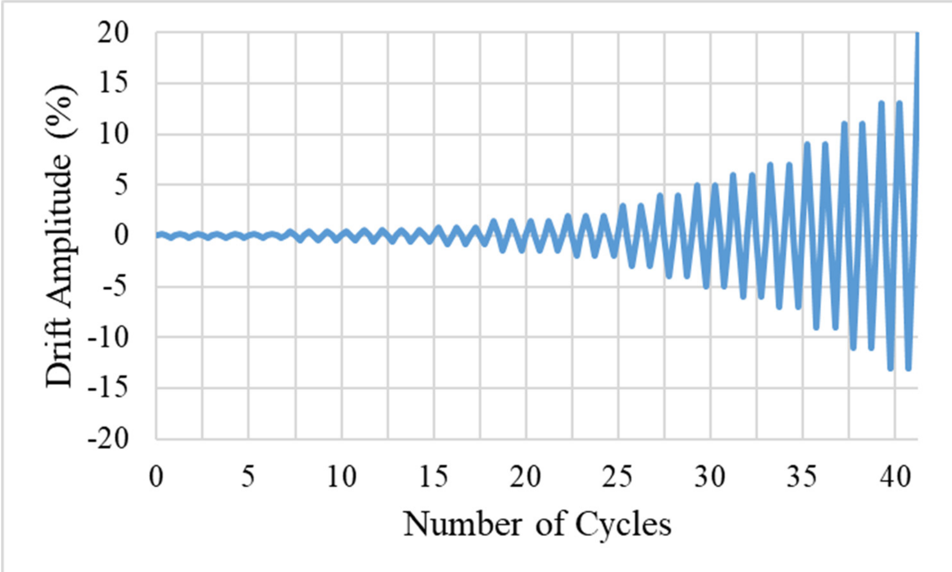


Figure A.6 Specimen A-11 loading protocol.

Table A.8 Specimen A-11 loading protocol.

Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loadig rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.048	7	0.0064	30	210
2	0.4	0.096	4	0.0128	30	120
3	0.6	0.144	4	0.0192	30	120
4	0.8	0.192	3	0.0256	30	90
5	1.4	0.336	3	0.0448	30	90
6	2	0.48	3	0.064	30	90
7	3	0.72	2	0.096	30	60
8	4	0.96	2	0.128	30	60
9	5	1.2	2	0.16	30	60
10	6	1.44	2	0.192	30	60
11	7	1.68	2	0.224	30	60
12	9	2.16	2	0.288	30	60
13	11	2.64	2	0.176	60	120
14	13	3.12	2	0.208	60	120
15	Mono	5.0	--	0.333	60	60

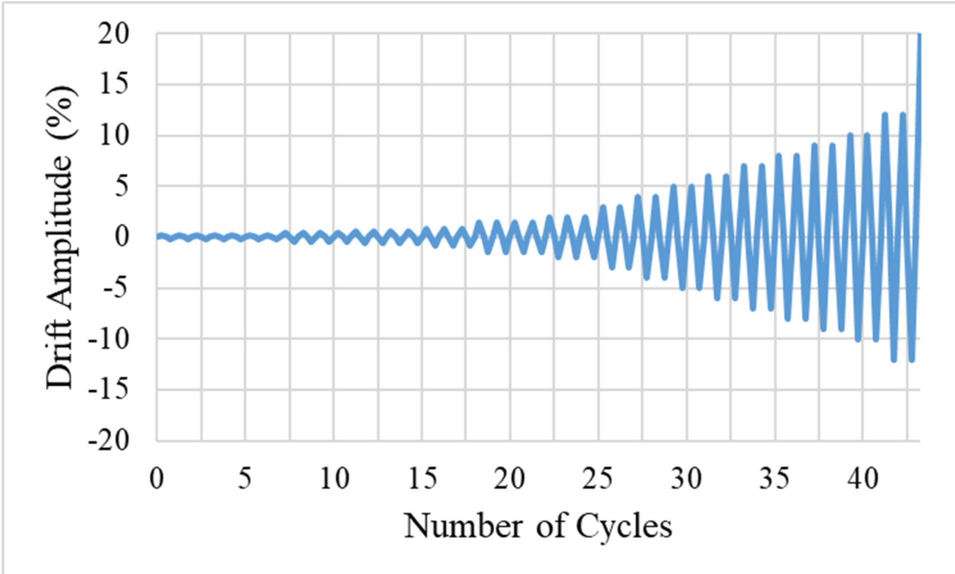


Figure A.7 Specimen A-12 loading protocol.

Table A.9 Specimen A-12 loading protocol.

Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loadig rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.048	7	0.0064	30	210
2	0.4	0.096	4	0.0128	30	120
3	0.6	0.144	4	0.0192	30	120
4	0.8	0.192	3	0.0256	30	90
5	1.4	0.336	3	0.0448	30	90
6	2	0.48	3	0.064	30	90
7	3	0.72	2	0.096	30	60
8	4	0.96	2	0.128	30	60
9	5	1.2	2	0.16	30	60
10	6	1.44	2	0.192	30	60
11	7	1.68	2	0.224	30	60
12	8	1.92	2	0.256	30	60
13	9	2.16	2	0.288	30	60
14	10	2.4	2	0.16	60	120
15	12	2.88	2	0.192	60	120
19	Mono	5.0	--	0.333	60	60

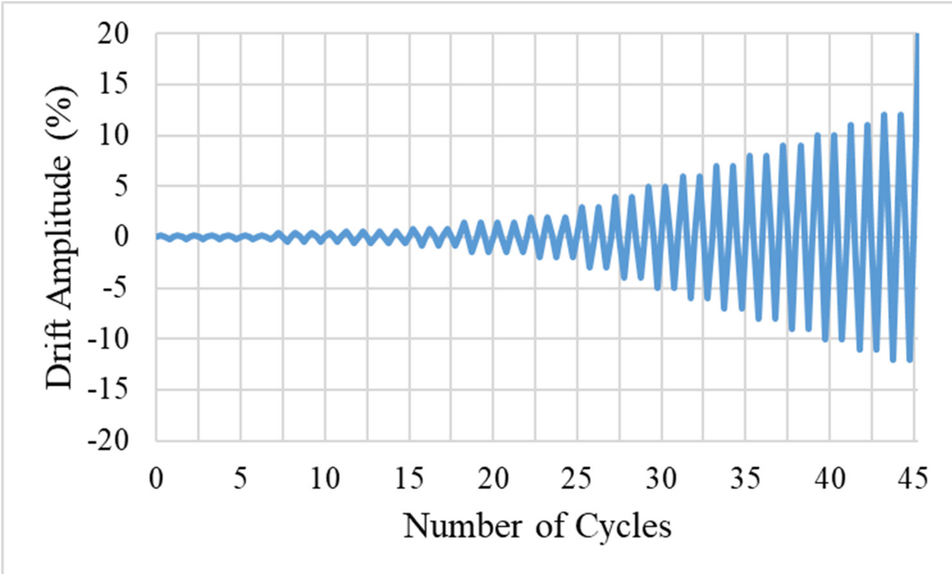


Figure A.8 Specimen A-13 loading protocol.

Table A.10 Specimen A-13 loading protocol.

Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loadig rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.144	7	0.0192	30	210
2	0.4	0.288	4	0.0384	30	120
3	0.6	0.432	4	0.0576	30	120
4	0.8	0.576	3	0.0768	30	90
5	1.4	1.008	3	0.1344	30	90
6	2	1.44	3	0.096	60	180
7	3	2.16	2	0.144	60	120
8	4	2.88	2	0.192	60	120
9	5	3.6	2	0.24	60	120
10	6	4.32	2	0.288	60	120
11	7	5.04	2	0.336	60	120
12	8	5.76	2	0.192	120	240
13	9	6.48	2	0.216	120	240
14	10	7.20	2	0.24	120	240
15	11	7.92	2	0.264	120	240
16	12	8.64	2	0.288	120	240
17	Mono	15.0	--	0.333	180	180

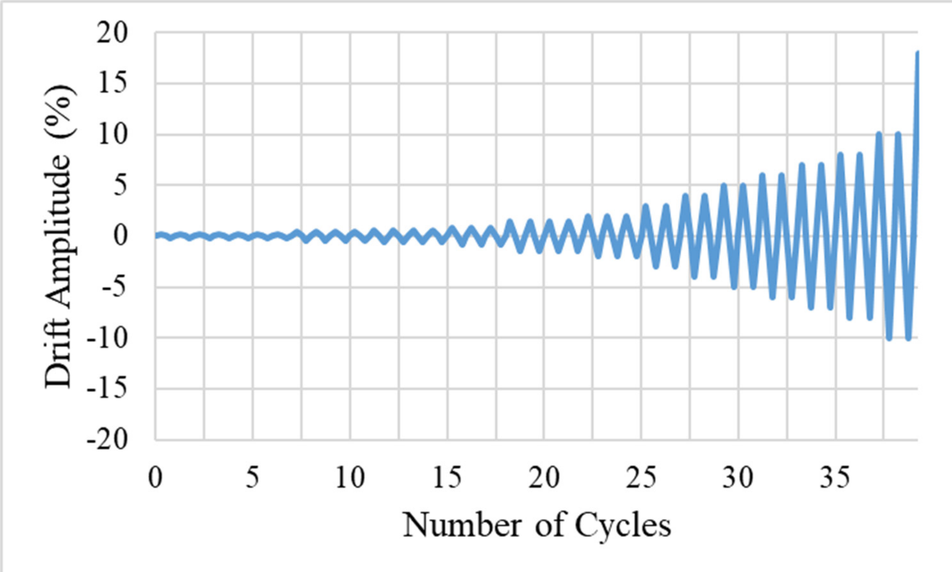


Figure A.9 Specimen A-14 loading protocol.

Table A.11 Specimen A-14 loading protocol.

Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loadig rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.144	7	0.0192	30	210
2	0.4	0.288	4	0.0384	30	120
3	0.6	0.432	4	0.0576	30	120
4	0.8	0.576	3	0.0768	30	90
5	1.4	1.008	3	0.1344	30	90
6	2	1.44	3	0.096	60	180
7	3	2.16	2	0.144	60	120
8	4	2.88	2	0.192	60	120
9	5	3.6	2	0.24	60	120
10	6	4.32	2	0.288	60	120
11	7	5.04	2	0.336	60	120
12	8	5.76	2	0.192	120	240
13	10	7.20	2	0.24	120	240
14	Mono	15.0	--	0.333	180	180

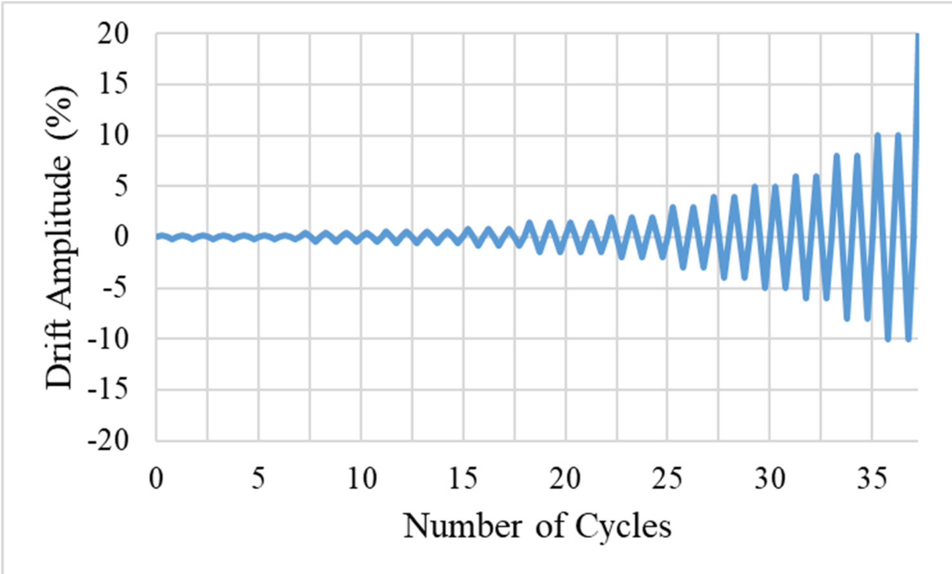


Figure A.10 Specimen A-23 loading protocol.

Table A.12 Specimen A-23 loading protocol.

Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loadig rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.144	7	0.0192	30	210
2	0.4	0.288	4	0.0384	30	120
3	0.6	0.432	4	0.0576	30	120
4	0.8	0.576	3	0.0768	30	90
5	1.4	1.008	3	0.1344	30	90
6	2	1.44	3	0.096	60	180
7	3	2.16	2	0.144	60	120
8	4	2.88	2	0.192	60	120
9	5	3.6	2	0.24	60	120
10	6	4.32	2	0.288	60	120
11	8	5.76	2	0.192	120	240
12	10	7.20	2	0.24	120	240
13	Mono	15.0	--	0.333	180	180

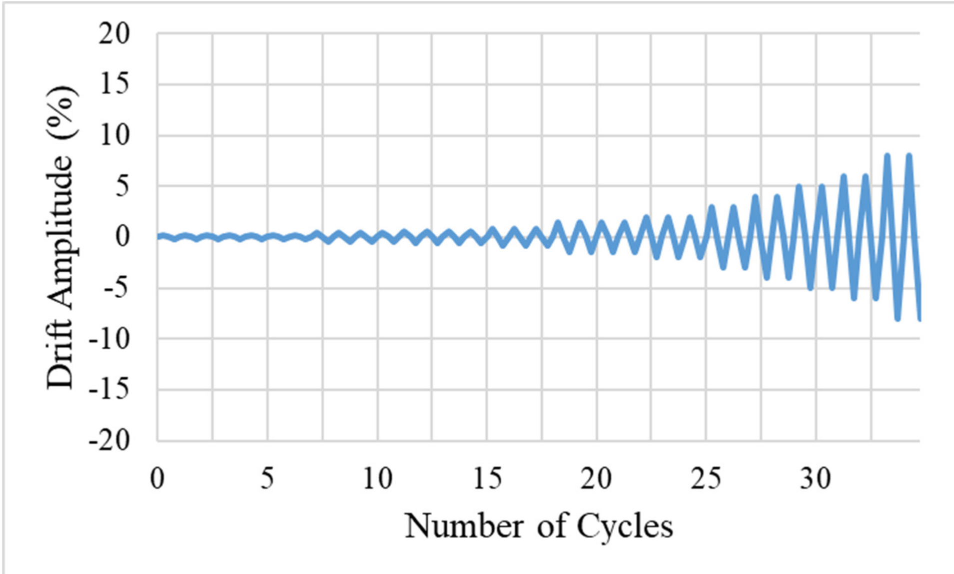


Figure A.11 **Specimen A-24 loading protocol.**

Table A.13 **Specimen A-24 loading protocol.**

Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loadig rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.144	7	0.0192	30	210
2	0.4	0.288	4	0.0384	30	120
3	0.6	0.432	4	0.0576	30	120
4	0.8	0.576	3	0.0768	30	90
5	1.4	1.008	3	0.1344	30	90
6	2	1.44	3	0.096	60	180
7	3	2.16	2	0.144	60	120
8	4	2.88	2	0.192	60	120
9	5	3.6	2	0.24	60	120
10	6	4.32	2	0.288	60	120
11	8	5.76	2	0.192	120	240

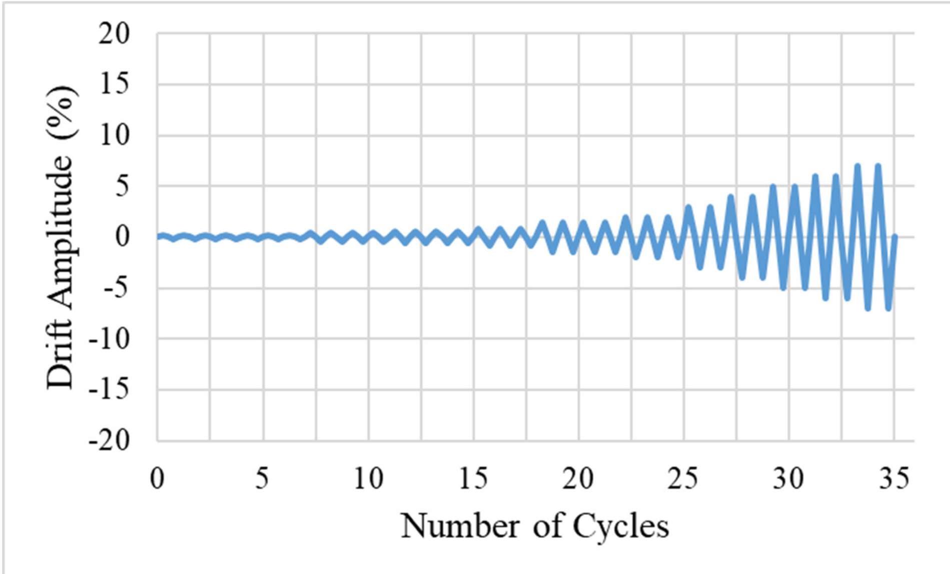


Figure A.12 Specimen A-28 loading protocol.

Table A.14 Specimen A-28 loading protocol.

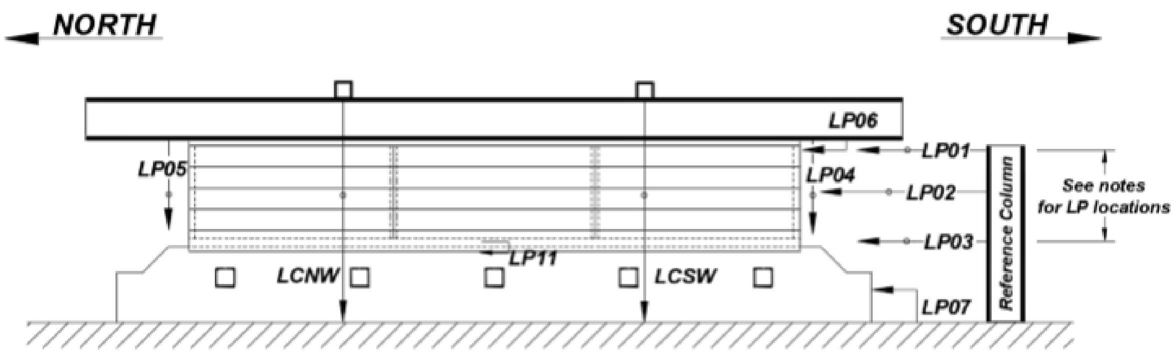
Cycle group no.	Drift (%)	Amplitude (in.)	No. of cycles per group	Loadig rate (in./sec)	Time per cycle (sec)	Total time per cycle group (sec)
1	0.2	0.048	7	0.0064	30	210
2	0.4	0.096	4	0.0128	30	120
3	0.6	0.144	4	0.0192	30	120
4	0.8	0.192	3	0.0256	30	90
5	1.4	0.336	3	0.0448	30	90
6	2	0.48	3	0.064	30	90
7	3	0.72	2	0.096	30	60
8	4	0.96	2	0.128	30	60
9	5	1.2	2	0.16	30	60
10	6	1.44	2	0.192	30	60
11	7	1.68	2	0.224	30	60
12	8	1.92	2	0.256	30	60
13	9	2.16	2	0.288	30	60
14	10	2.4	2	0.16	60	120
15	Mono	5.0	--	0.333	60	60

APPENDIX B TEST SETUP

Appendix B includes one section: instrumentation plans for testing (B.1). Discussion of this section is provided in Chapter 2.

B.1 INSTRUMENTATION DRAWINGS

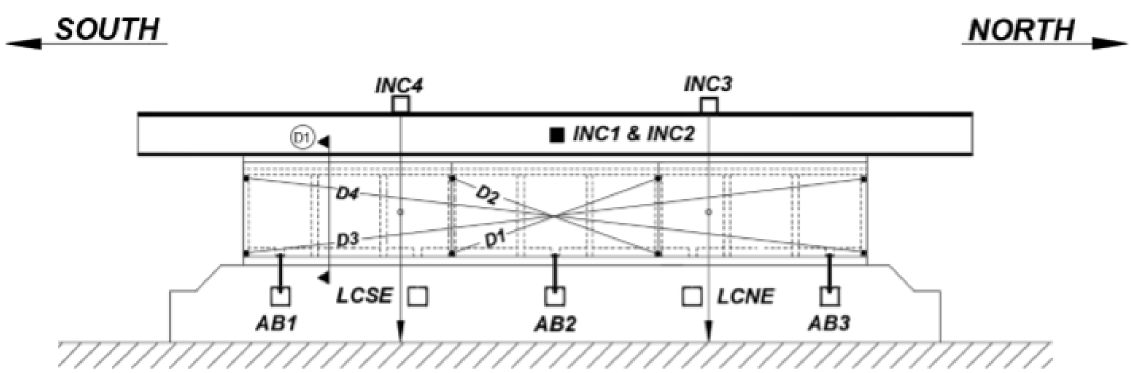
B.1.1 Specimen A-7 Instrumentation Drawings



Notes:

1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of upper top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LCNW and LCSW monitor the axial load on the East Side of the wall.
6. LP11 monitors displacement of siding relative to the footing.

Instrumentation Elevation (E1)
Siding Face
2' Tall Cripple Wall

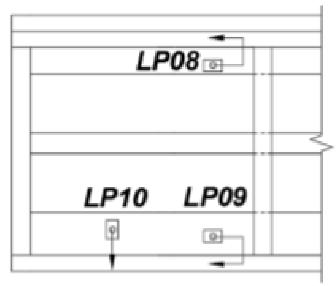


Notes:

1. AB1-AB5 are instrumented with 2" Ø Donut Load Cells measuring uplift loads.
2. Pairs of strength potentiometers are mounted on framing studs or plywood panels for retrofit cases.
3. INC1 measures longitudinal rotation of load transfer beam. INC2 measures the transverse rotation of the load transfer beam.
4. INC3 and INC4 measure the rotation of the transverse beams where axial load is applied.
5. LCNE and LCSE monitor the axial load on the West Side of the wall.

Instrumentation Elevation (E2)
Framing Face
2' Tall Cripple Wall

Figure B.1 Specimen A-7 instrumentation.

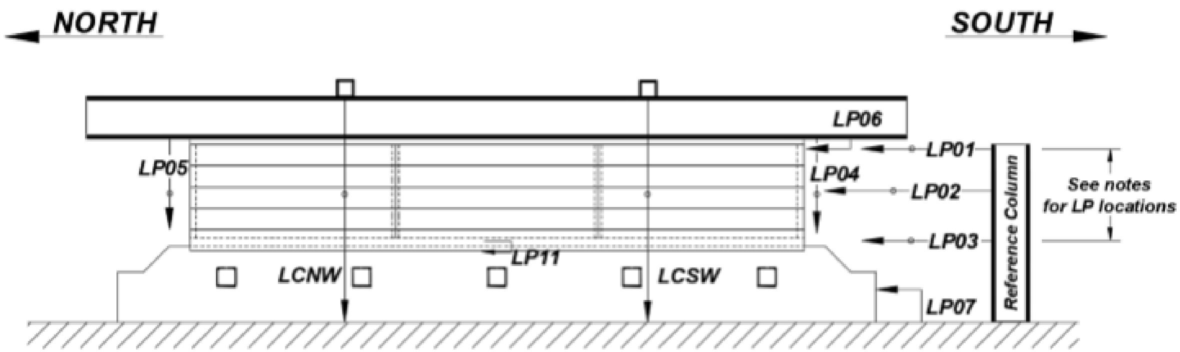


Notes:
1. LP08 and LP09 monitor the siding slip.
2. LP10 monitors siding uplift.

Instrumentation Detail D1
Siding

Figure B.1 (continued).

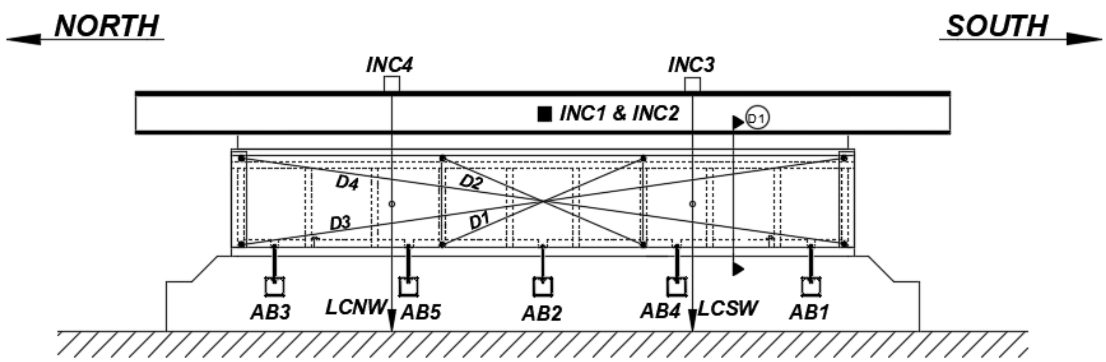
B.1.2 Specimen A-8 Instrumentation Drawings



Notes:

1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of upper top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LCNW and LCSW monitor the axial load on the East Side of the wall.
6. LP11 monitors displacement of siding relative to the footing.

Instrumentation Elevation E1
Siding Face
2' Tall Cripple Wall

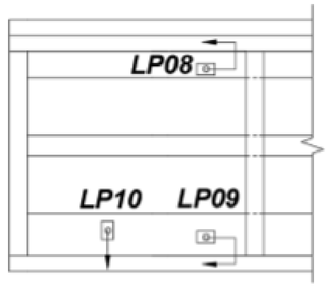


Notes:

1. AB1-AB5 are instrumented with 2" Ø Donut Load Cells measuring uplift loads.
2. Pairs of strength potentiometers are mounted on framing studs or plywood panels for retrofit cases.
3. INC1 measures longitudinal rotation of load transfer beam. INC2 measures the transverse rotation of the load transfer beam.
4. INC3 and INC4 measure the rotation of the transverse beams where axial load is applied.
5. LCNW and LCSW monitor the axial load on the West Side of the wall.

Instrumentation Elevation E2
Framing Face
2' Tall Cripple Wall

Figure B.2 Specimen A-8 instrumentation.

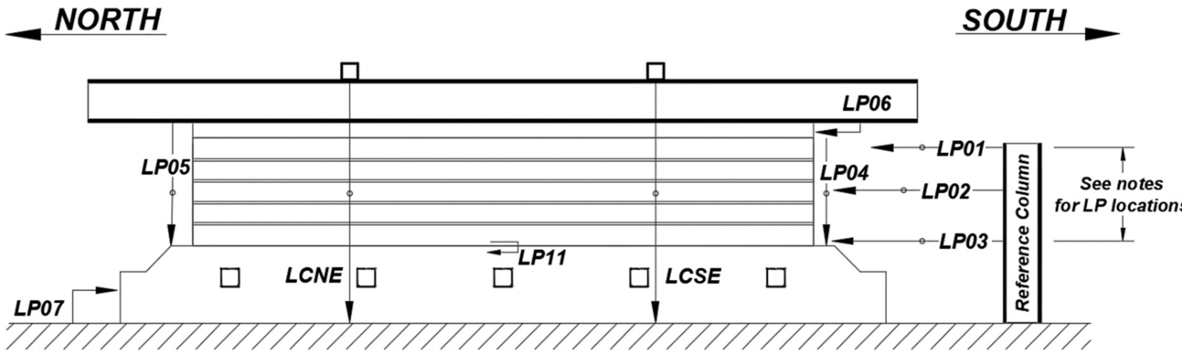


Notes:
1. LP08 and LP09 monitor the siding slip.
2. LP10 monitors siding uplift.

Instrumentation Detail D1
Siding

Figure B.2 (continued).

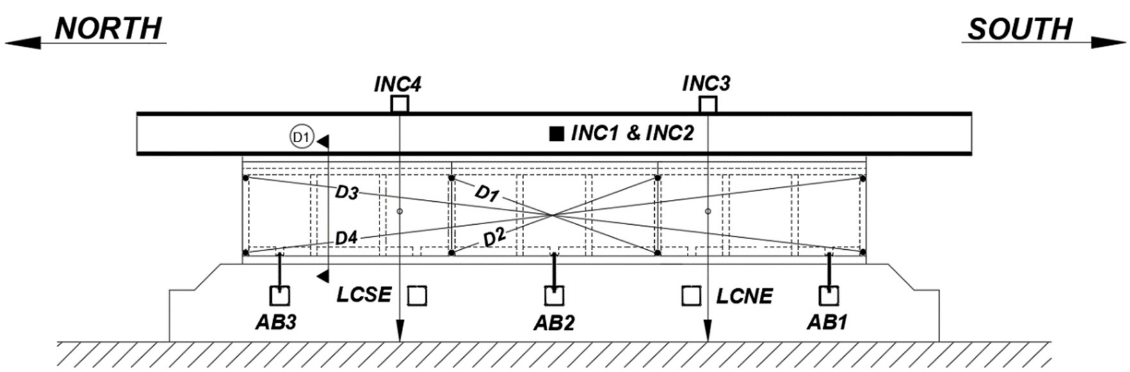
B.1.3 Specimen A-9 Instrumentation Drawings



Notes:

1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of second top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LCNE and LCSE monitor the axial load on the East Side of the wall.
6. LP11 monitors displacement of siding relative to the footing.

Instrumentation Elevation (E1)
Finish Face
2' Tall Cripple Wall

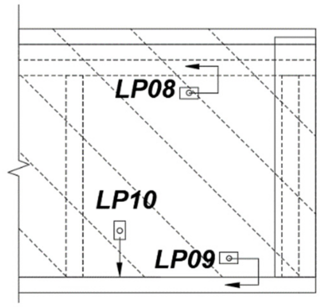


Notes:

1. AB1-AB5 are instrumented with 2" Ø Donut Load Cells measuring uplift loads.
2. Pairs of strength potentiometers are mounted on framing studs or plywood panels for retrofit cases.
3. INC1 measures longitudinal rotation of load transfer beam. INC2 measures the transverse rotation of the load transfer beam.
4. INC3 and INC4 measure the rotation of the transverse beams where axial load is applied.
5. LCNE and LCSE monitor the axial load on the East Side of the wall.

Instrumentation Elevation (E2)
Framing Face
2' Tall Cripple Wall

Figure B.3 Specimen A-9 instrumentation.

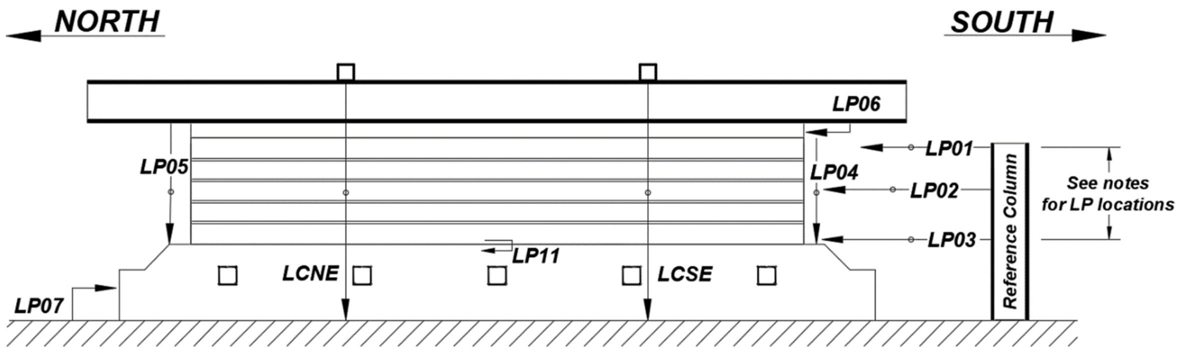


Notes:
1. LP08 and LP09 monitor the sheathing slip.
2. LP10 monitors the sheathing uplift

Instrumentation Detail D1
Sheathing

Figure B.3 (continued).

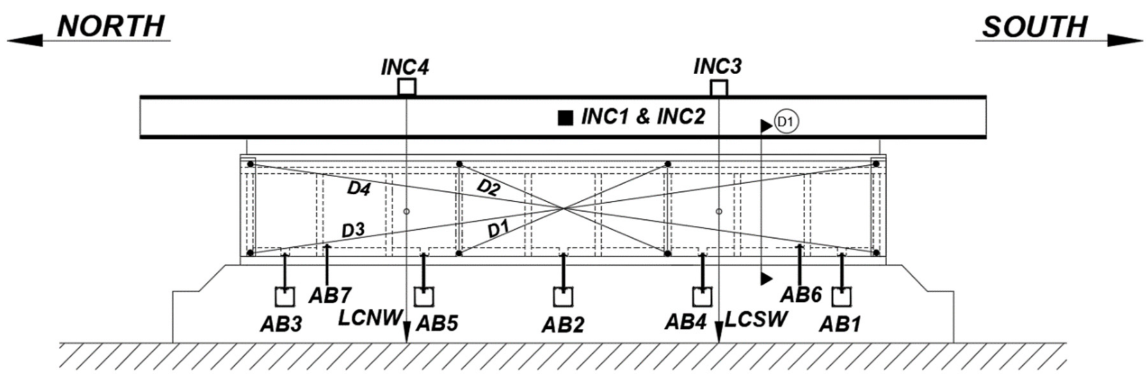
B.1.4 I Specimen A-10 Instrumentation Drawings



Notes:

1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of second top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LCNE and LCSE monitor the axial load on the East Side of the wall.
6. LP11 monitors displacement of siding relative to the footing.

Instrumentation Elevation (E1)
Finish Face
2' Tall Cripple Wall

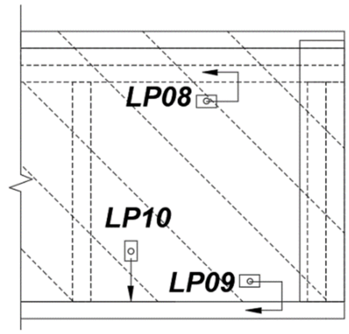


Notes:

1. AB1-AB7 are instrumented with 2" Ø Donut Load Cells measuring uplift loads.
2. Pairs of strength potentiometers are mounted on framing studs or plywood panels for retrofit cases.
3. INC1 measures longitudinal rotation of load transfer beam. INC2 measures the transverse rotation of the load transfer beam.
4. INC3 and INC4 measure the rotation of the transverse beams where axial load is applied.
5. LCNW and LCSW monitor the axial load on the West Side of the wall.

Instrumentation Elevation (E2)
Framing Face
2' Tall Cripple Wall

Figure B.4 Specimen A-10 instrumentation.



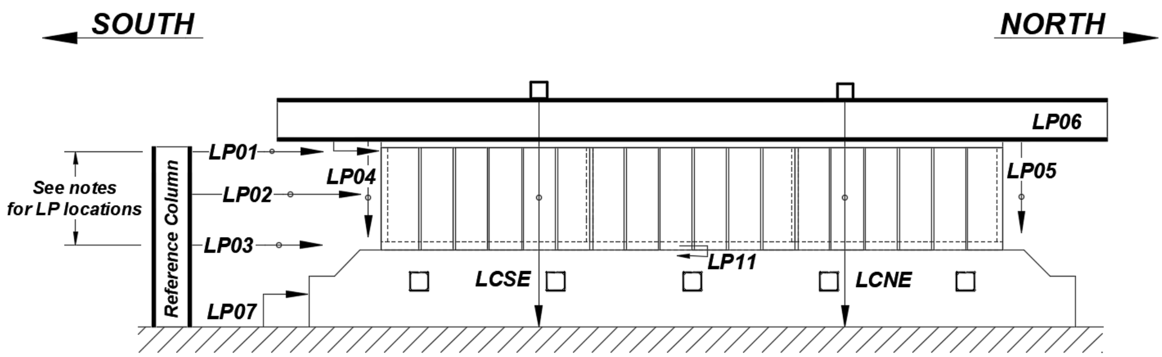
Notes:

1. LP08 and LP09 monitor the sheathing slip.
2. LP10 monitors the sheathing uplift

Instrumentation Detail D1
Sheathing

Figure B.5 (continued).

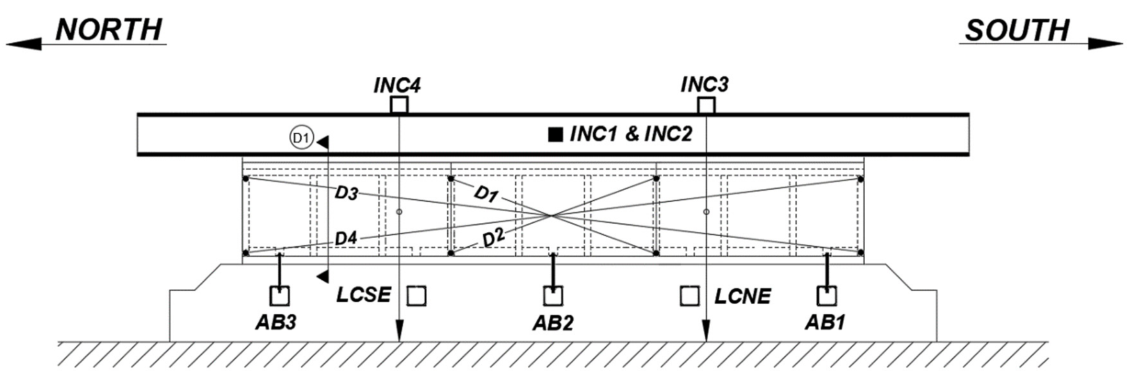
B.1.5 Specimen A-1 Instrumentation Drawings



Notes:

1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of upper top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LCNW and LCSW monitor the axial load on the West Side of the wall.
6. LP11 monitors displacement of siding relative to the footing.

Instrumentation Elevation (E1)
Panel Face
2' Tall Cripple Wall



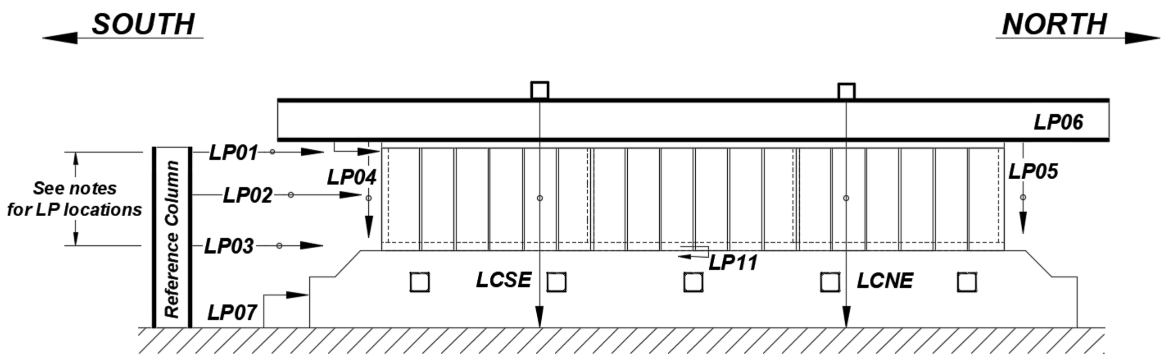
Notes:

1. AB1-AB5 are instrumented with 2" Ø Donut Load Cells measuring uplift loads.
2. Pairs of strength potentiometers are mounted on framing studs or plywood panels for retrofit cases.
3. INC1 measures longitudinal rotation of load transfer beam. INC2 measures the transverse rotation of the load transfer beam.
4. INC3 and INC4 measure the rotation of the transverse beams where axial load is applied.
5. LCNE and LCSE monitor the axial load on the East Side of the wall.

Instrumentation Elevation (E2)
Framing Face
2' Tall Cripple Wall

Figure B.6 Specimen A-11 instrumentation.

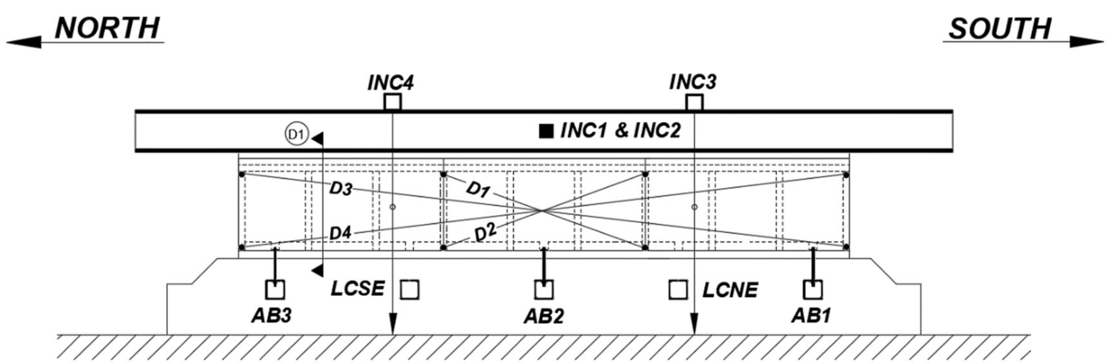
B.1.6 Specimen A-12 Instrumentation Drawings



Notes:

1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of upper top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LCNW and LCSW monitor the axial load on the West Side of the wall.
6. LP11 monitors displacement of siding relative to the footing.

Instrumentation Elevation (E1)
Panel Face
2' Tall Cripple Wall



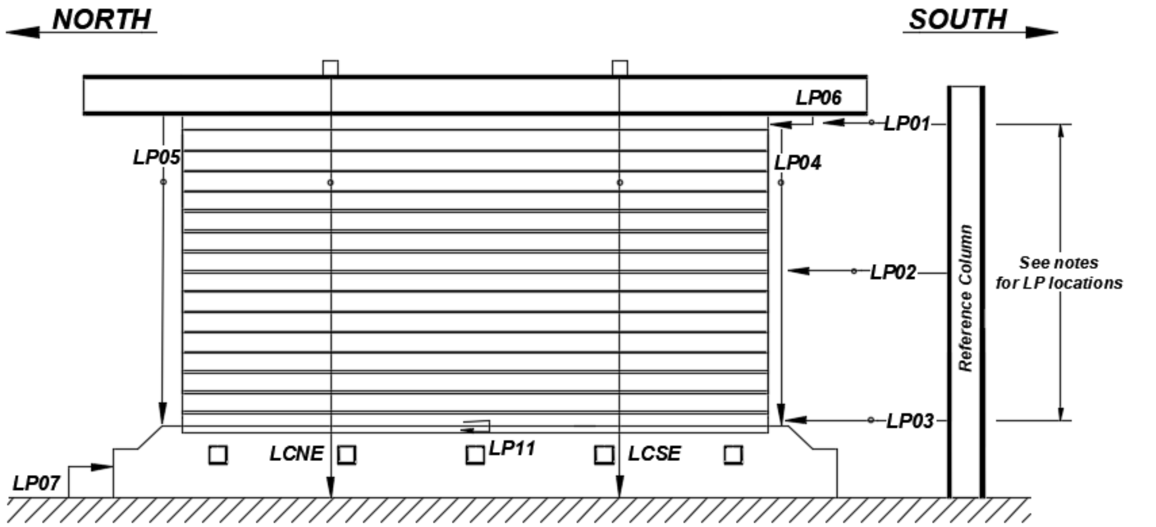
Notes:

1. AB1-AB5 are instrumented with 2" Ø Donut Load Cells measuring uplift loads.
2. Pairs of strength potentiometers are mounted on framing studs or plywood panels for retrofit cases.
3. INC1 measures longitudinal rotation of load transfer beam. INC2 measures the transverse rotation of the load transfer beam.
4. INC3 and INC4 measure the rotation of the transverse beams where axial load is applied.
5. LCNE and LCSE monitor the axial load on the East Side of the wall.

Instrumentation Elevation (E2)
Framing Face
2' Tall Cripple Wall

Figure B.7 Specimen A-12 instrumentation.

B.1.7 Specimen A-13 Instrumentation Drawings

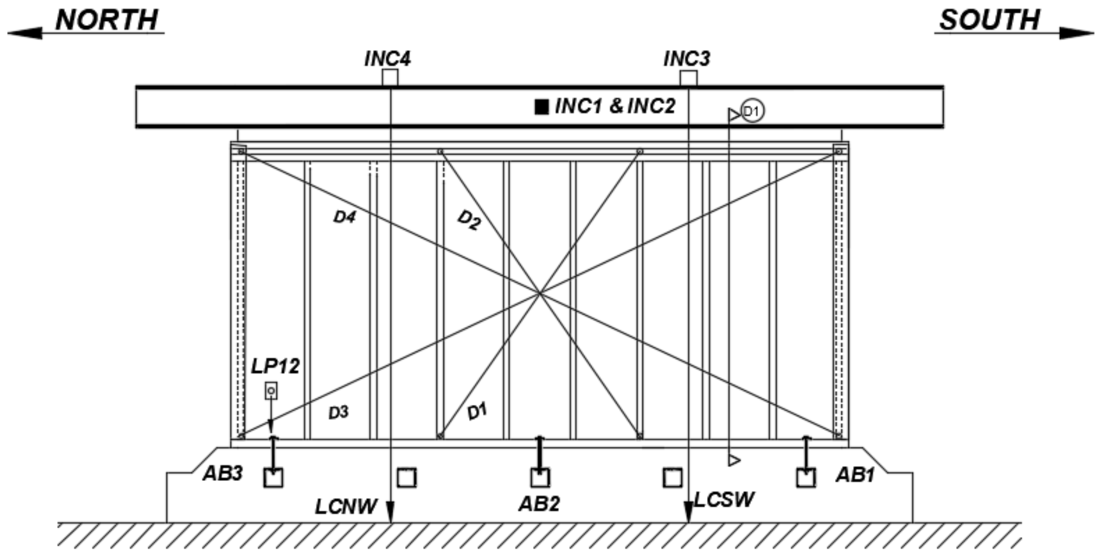


Notes:

1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of second top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LCNE and LCSE monitor the axial load on the East Side of the wall.
6. LP11 monitors displacement of siding relative to the footing.

Instrumentation Elevation (E1)
Finish Face
6' Tall Cripple Wall

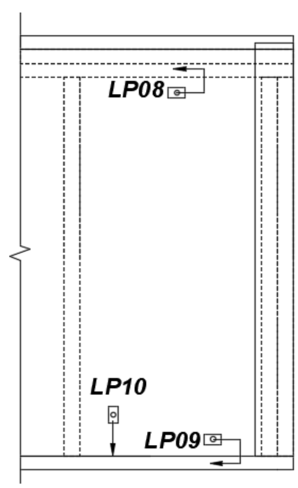
Figure B.8 Specimen A-13 instrumentation.



Notes:

1. AB1-AB3 are instrumented with 2" Ø Donut Load Cells measuring uplift loads.
2. Pairs of strength potentiometers are mounted on framing studs or plywood panels for retrofit cases.
3. INC1 measures longitudinal rotation of load transfer beam. INC2 measures the transverse rotation of the load transfer beam.
4. INC3 and INC4 measure the rotation of the transverse beams where axial load is applied.
5. LCNW and LCSW monitor the axial load on the West Side of the wall.
6. LP12 monitors the uplift of the bottom siding board relative to the sill plate.

Instrumentation Elevation (E2)
Framing Face
6' Tall Cripple Wall



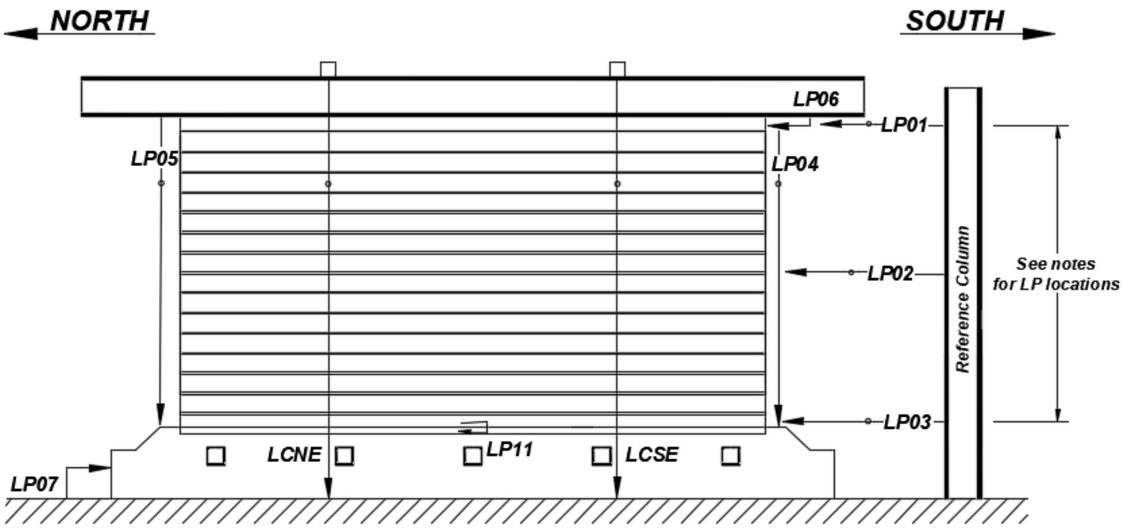
Notes:

1. LP08 and LP09 monitor the sheathing slip.
2. LP10 monitors the sheathing uplift

Instrumentation Detail (D1)
Sheathing

Figure B.7 (continued).

B.1.8 Specimen A-14 Instrumentation Drawings

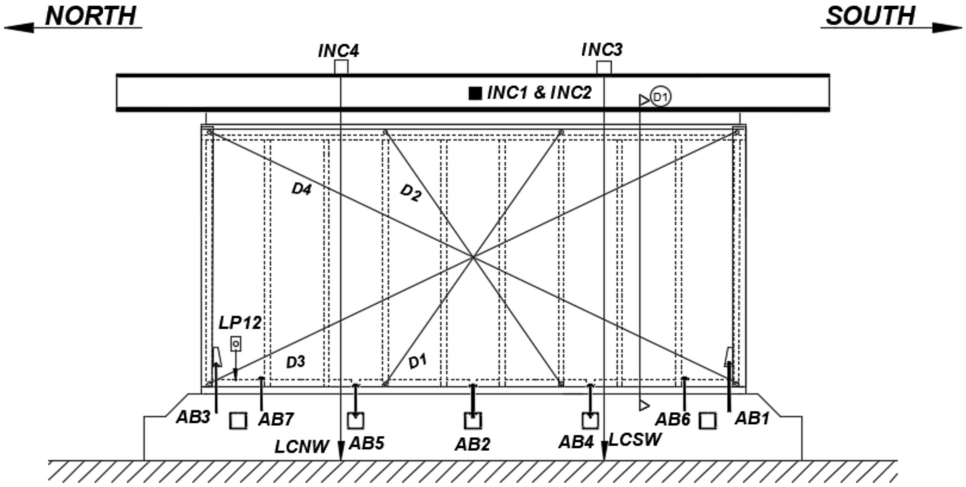


Notes:

1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of second top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LCNE and LCSE monitor the axial load on the East Side of the wall.
6. LP11 monitors displacement of siding relative to the footing.

Instrumentation Elevation (E1)
Finish Face
6' Tall Cripple Wall

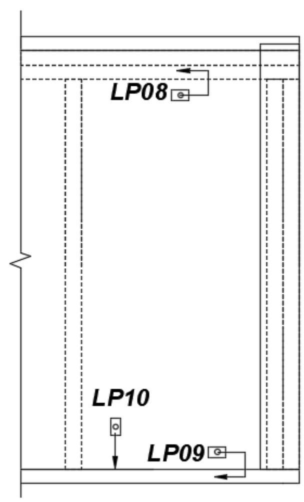
Figure B.9 Specimen A-14 instrumentation



Notes:

1. AB1-AB7 are instrumented with 2" Ø Donut Load Cells measuring uplift loads.
2. Pairs of strength potentiometers are mounted on framing studs or plywood panels for retrofit cases.
3. INC1 measures longitudinal rotation of load transfer beam. INC2 measures the transverse rotation of the load transfer beam.
4. INC3 and INC4 measure the rotation of the transverse beams where axial load is applied.
5. LCNW and LCSW monitor the axial load on the West Side of the wall.

Instrumentation Elevation (E2)
Framing Face
6' Tall Cripple Wall



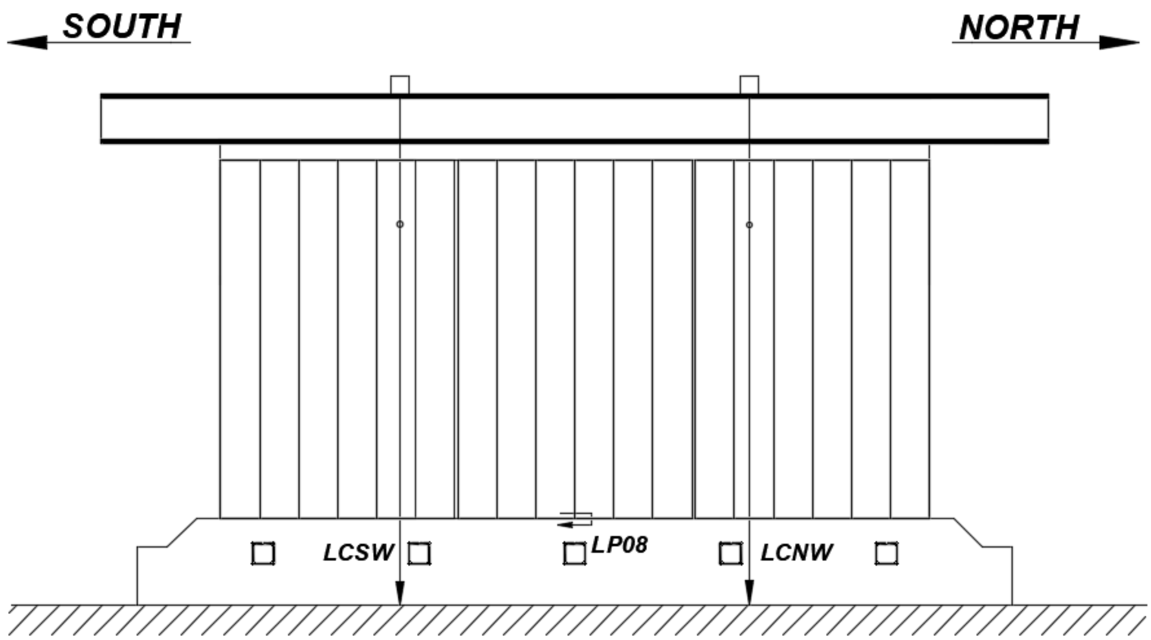
Notes:

1. LP08 and LP09 monitor the sheathing slip.
2. LP10 monitors the sheathing uplift

Instrumentation Detail (D1)
Sheathing

Figure B.8 (continued).

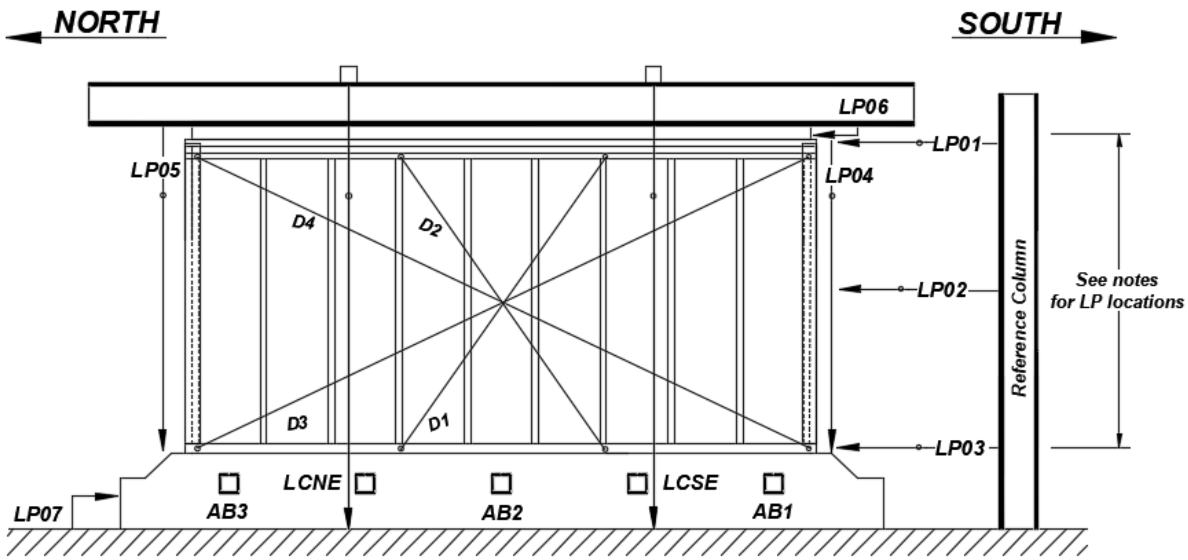
B.1.9 Specimen A-23 Instrumentation Drawings



Notes:
1. LP08 monitors the out-of-plane displacement of the middle T1-11 panel.
2. LCNE and LCSE monitor the axial load on the East Side of the wall.

Instrumentation Elevation E1
Finish Face
T1-11 6' Tall Cripple Wall

Figure B.9 Specimen A-23 instrumentation.



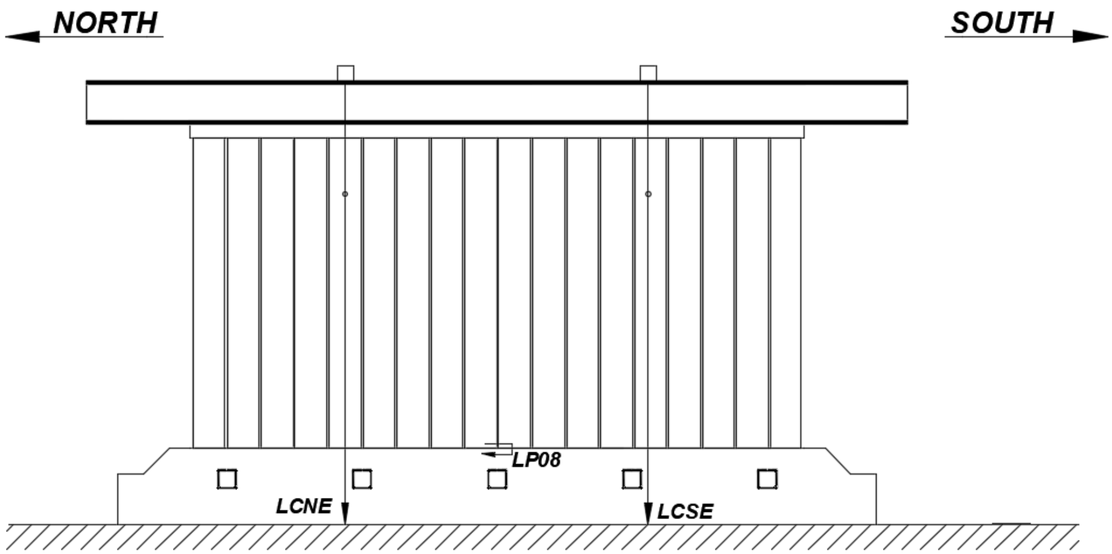
Notes:

1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of second top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LCNE and LCSE monitor the axial load on the East Side of the wall.

Instrumentation Elevation (E2)
Framing Face
6' Tall T1-11 Cripple Wall

Figure B.9 (continued)

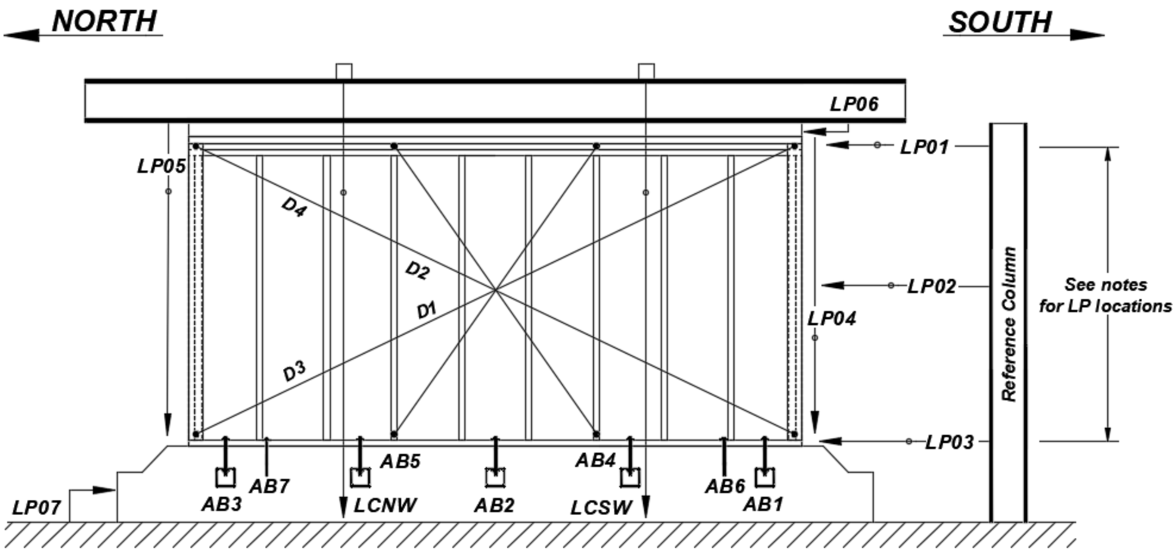
B.1.10 Specimen A-24 Instrumentation Drawings



Notes:
1. **LCNE and LCSE monitor the axial load on the East Side of the wall.**
2. **LP08 monitors displacement of siding relative to the footing.**

Instrumentation Elevation E1
Finish Face
6' T1-11 Retrofitted
Cripple Wall

Figure B.10 Specimen A-24 instrumentation.



Notes:

1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of second top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LCNW and LCSW monitor the axial load on the East Side of the wall.

Instrumentation Elevation (E2)
Framing Face
6' T1-11 Retrofitted
Cripple Wall

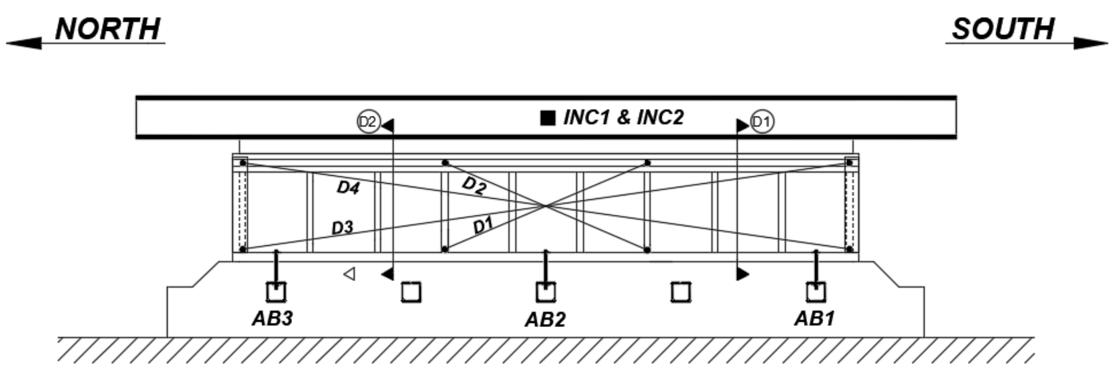
Figure B.9 (continued).

B.1.11 Specimen A-28 Instrumentation Drawings

Notes:

1. LP01, LP02, and LP03 measure lateral displacement with LP01 attached to middle of upper top plate, LP02 attached to middle of stud, and LP03 attached to middle of sill plate.
2. LP04 and LP05 measure the uplift at each end of the wall.
3. LP06 monitors the slip between the horizontal transfer beam and the upper top plate.
4. LP07 monitors the slip of the footing.
5. LP08 monitors displacement of bottom siding board relative to the footing.

Instrumentation Elevation (E1)
Finish Face
2' Tall Cripple Wall

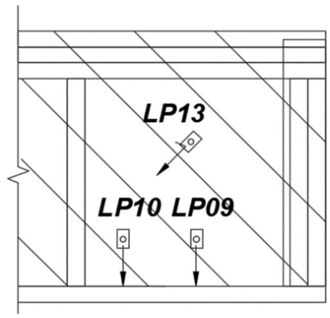


Notes:

1. AB1-AB3 are instrumented with 2" Ø Donut Load Cells measuring uplift loads.
2. Pairs of strength potentiometers are mounted on framing studs or plywood panels for retrofit cases.
3. INC1 measures longitudinal rotation of load transfer beam. INC2 measures the transverse rotation of the load transfer beam.

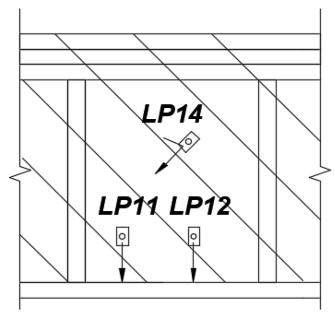
Instrumentation Elevation (E2)
Framing Face
2' Tall Cripple Wall

Figure B.11 Specimen A-28 instrumentation.



Notes:
1. LP09 and LP10 monitor the sheathing uplift.
2. LP13 monitors sheathing board gap.

Instrumentation Detail D1
Sheathing



Notes:
1. LP11 and LP12 monitor the sheathing uplift.
2. LP14 monitors sheathing board gap.

Instrumentation Detail D2
Sheathing

Figure B.10 (continued).

APPENDIX C ADDITIONAL TEST RESULTS

Appendix C includes three sections: anchor bolt load measurements (C.1), diagonal distortion measurements (C.2), and uplift measurements (C.3). Discussion of these sections is provided in Chapter 3.

C.1 ANCHOR BOLT MEASUREMENTS

Tension in anchor bolts were measured with 10-kip donut load cells placed on top of the square plate washers. A spherical washer was placed on top of the load cell and fastened with a nut. For unretrofitted cripple walls, three anchor bolts were used, spaced at 64 in. on center. For retrofitted cripple walls, additional anchor bolts were added as per the *FEMA P-1100* prescriptive retrofit guidelines. Anchor bolts were tensioned to around 0.2 kips, which is meant to mimic a hand-tightened amount of tension and representative of what would commonly be observed in the field for older homes.

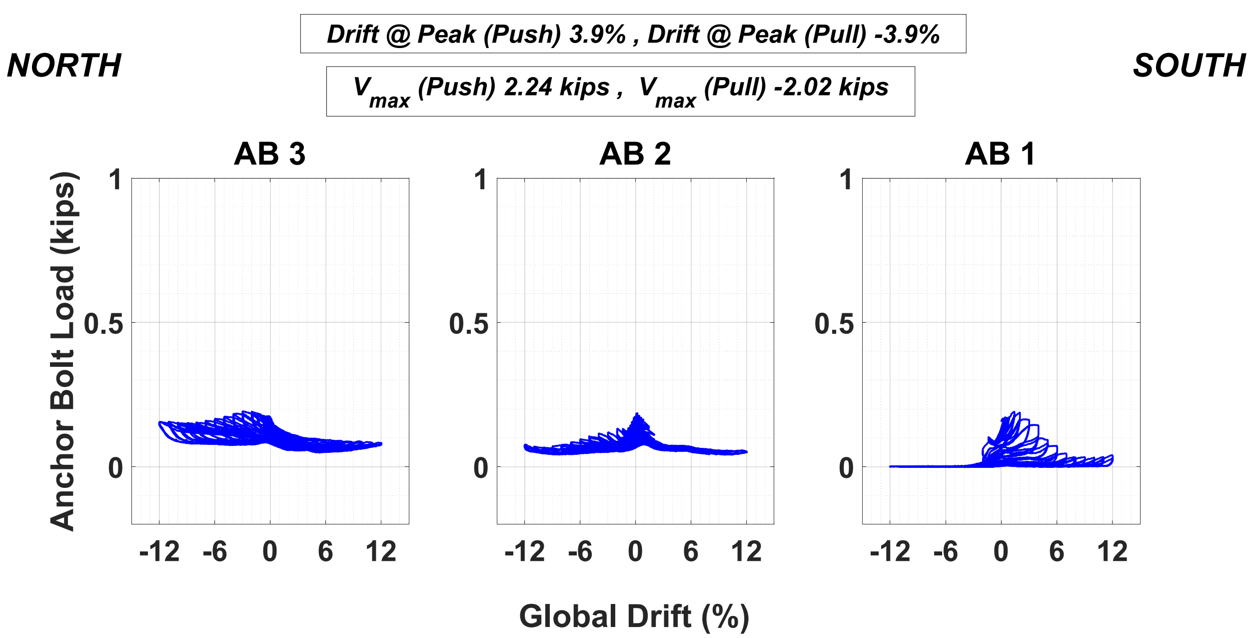


Figure C.1 **Specimen A-7 anchor bolt loads versus global drift for the existing 2-ft-tall cripple wall with horizontal siding.**

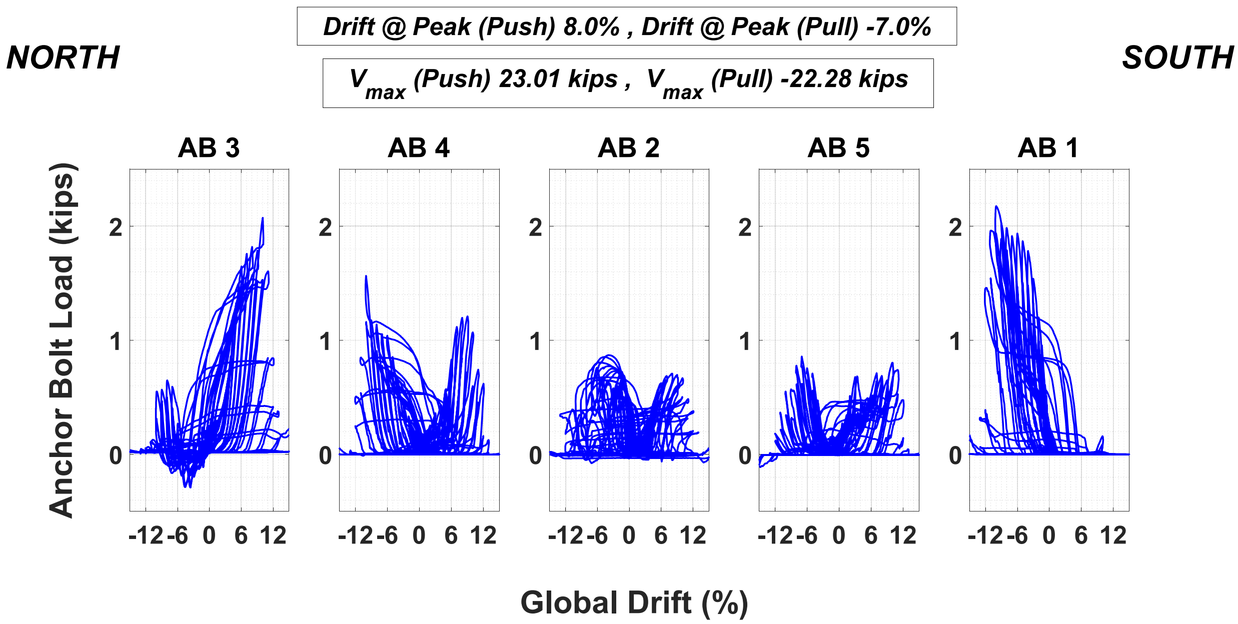


Figure C.2 Specimen A-8 anchor bolt loads versus global drift for the retrofitted 2-ft-tall cripple wall with horizontal siding.

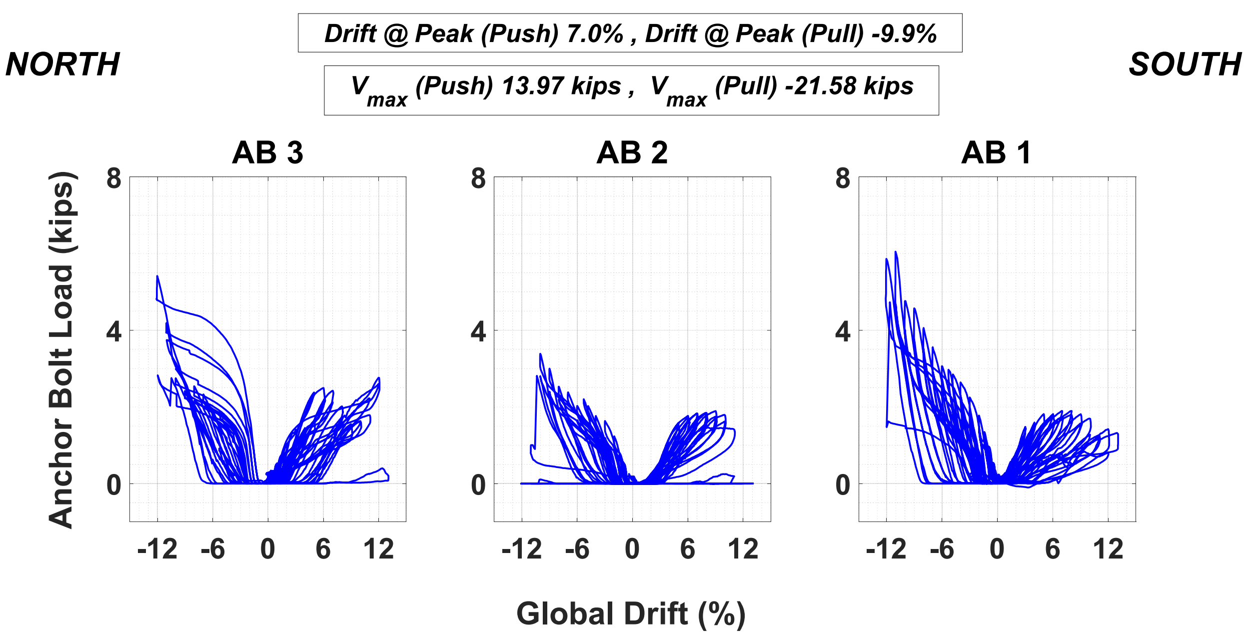


Figure C.3 Specimen A-9 anchor bolt loads versus global drift for the existing 2-ft-tall cripple wall with horizontal siding or diagonal sheathing.

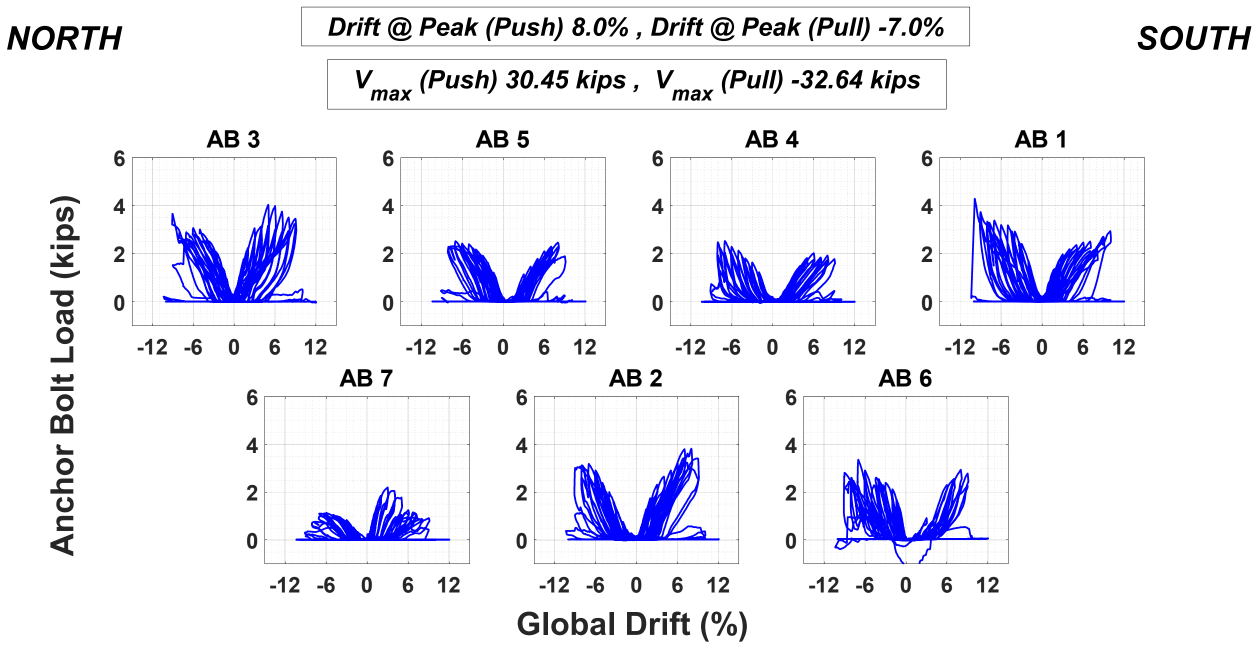


Figure C.4 Specimen A-10 anchor bolt loads versus global drift for the retrofitted 2-ft-tall cripple wall with horizontal siding or diagonal sheathing.

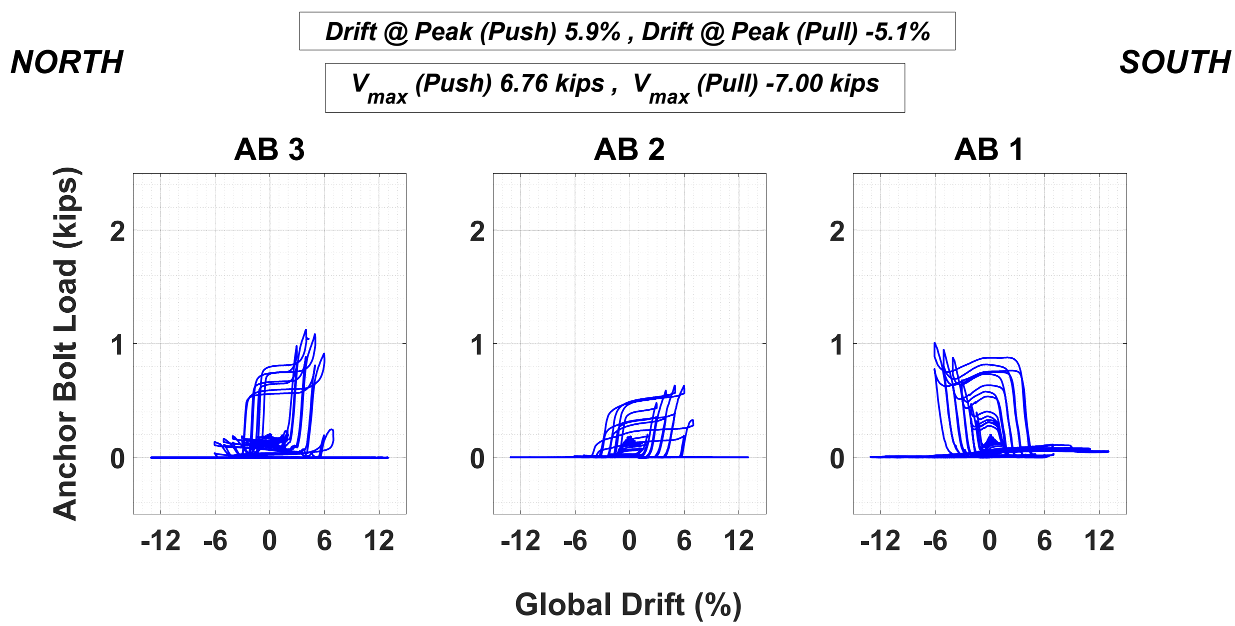


Figure C.5 Specimen A-11 anchor bolt loads versus global drift for the existing 2-ft-tall cripple wall with T1-11 wood structural panels.

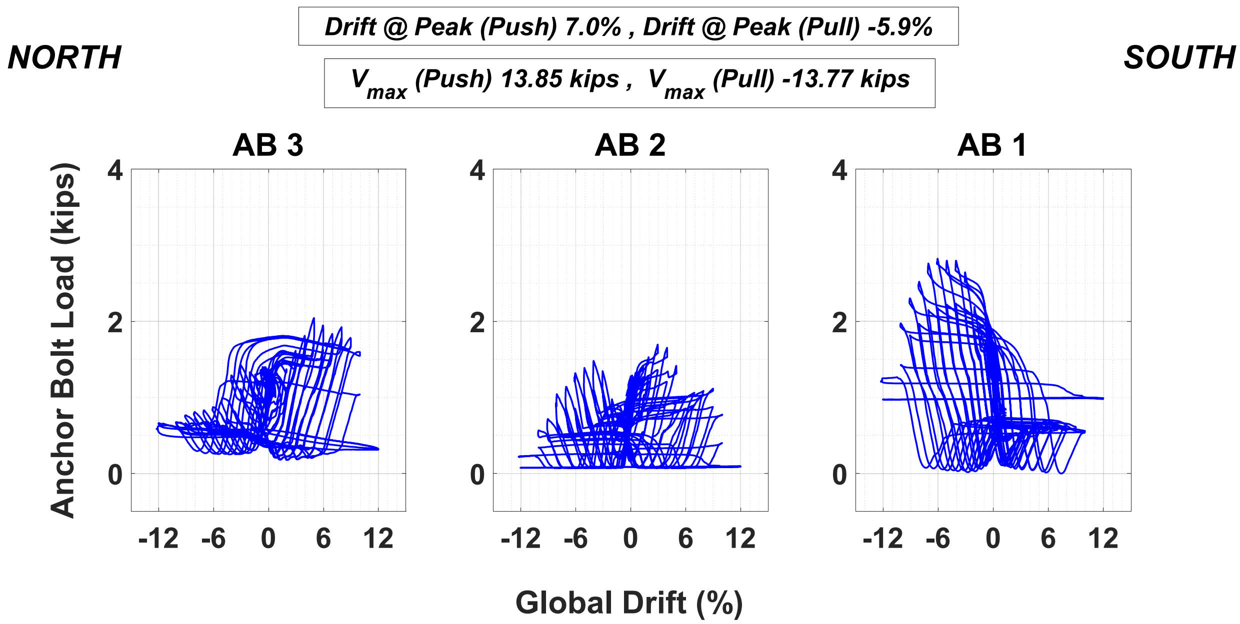


Figure C.6 Specimen A-12 anchor bolt loads versus global drift for the retrofitted 2-ft-tall cripple wall with T1-11 wood structural panels.

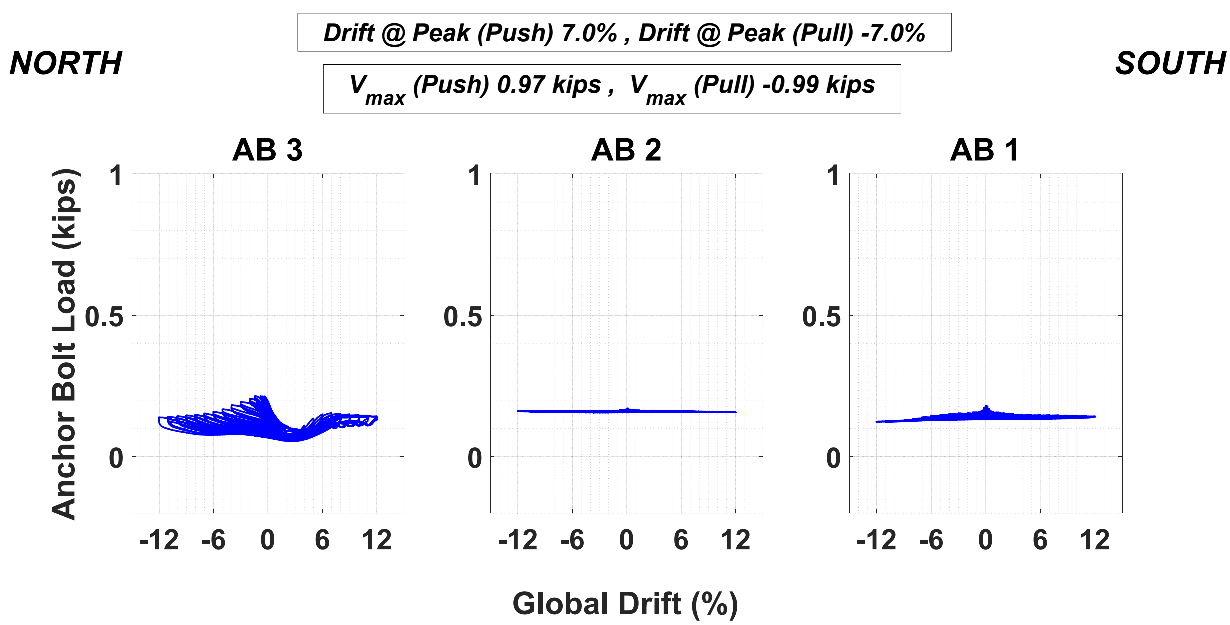


Figure C.7 Specimen A-13 anchor bolt loads versus global drift for the existing 6-ft-tall cripple wall with horizontal siding.

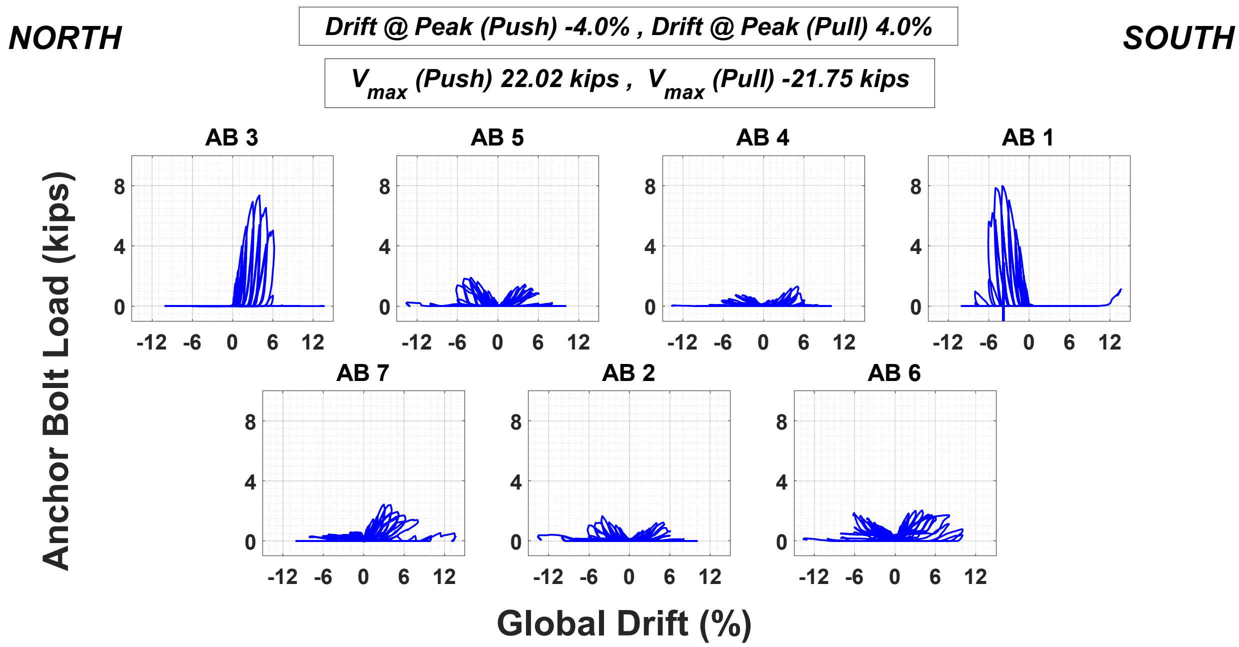


Figure C.8 Specimen A-14 anchor bolt loads versus global drift for existing 6-ft-tall cripple wall with horizontal siding.

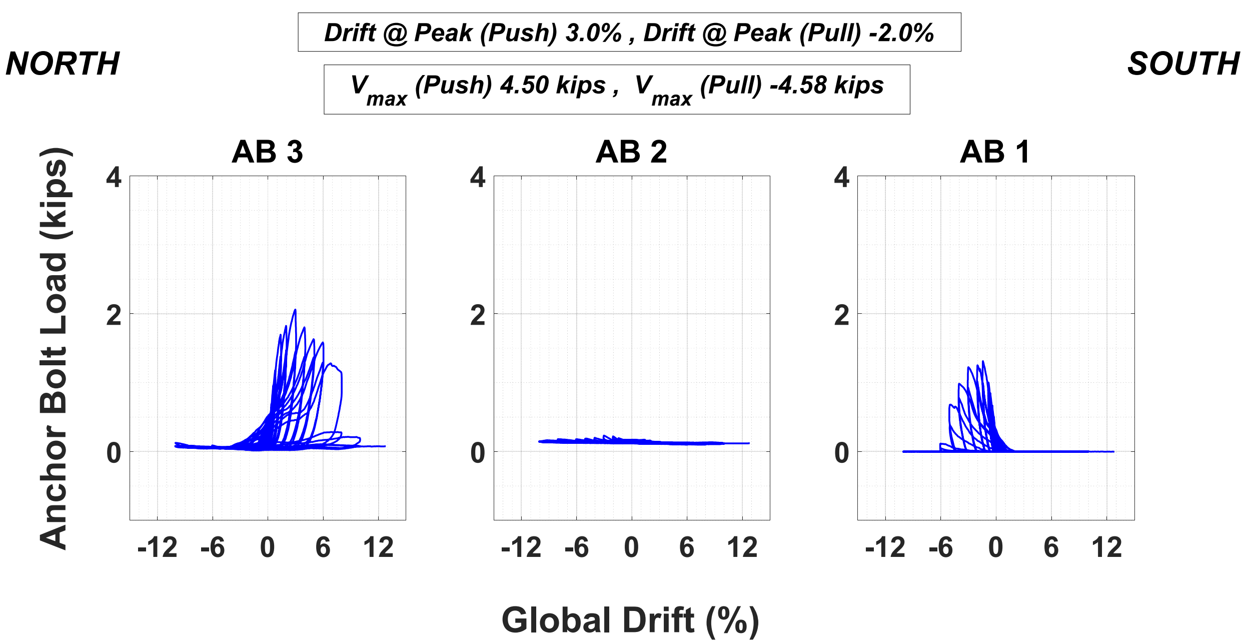


Figure C.9 Specimen A-23 anchor bolt loads versus global drift for existing 6-ft-tall cripple wall with T1-11 wood structural panels.

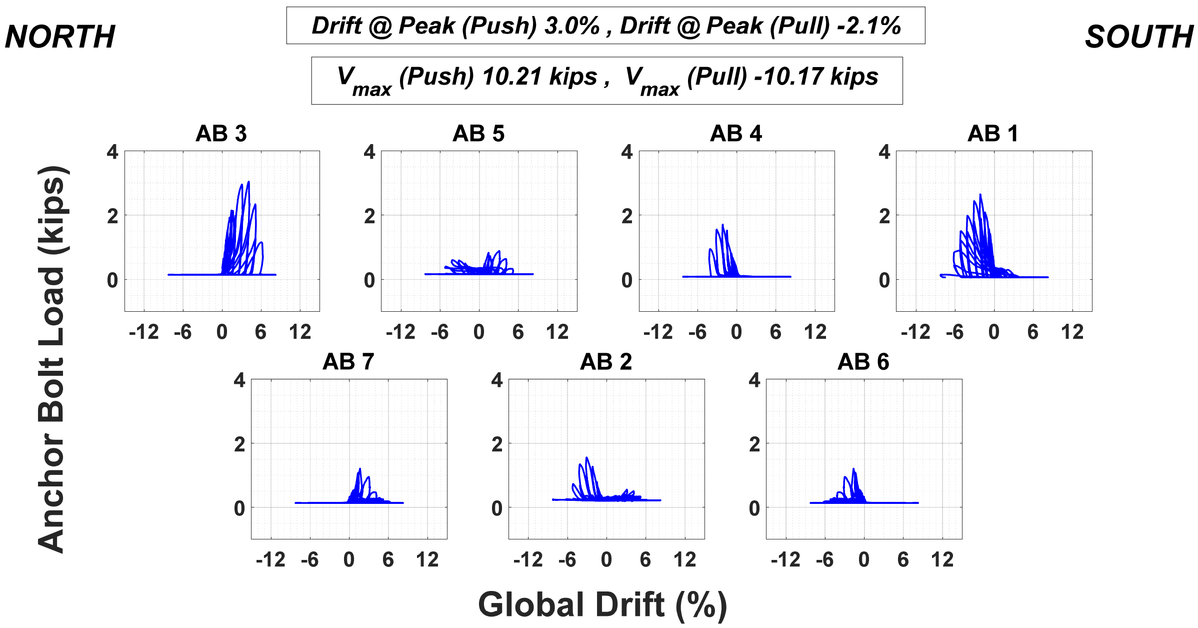


Figure C.10 Specimen A-24 anchor bolt loads versus global drift for the retrofitted 6-ft-tall cripple wall with T1-11 wood structural panels.

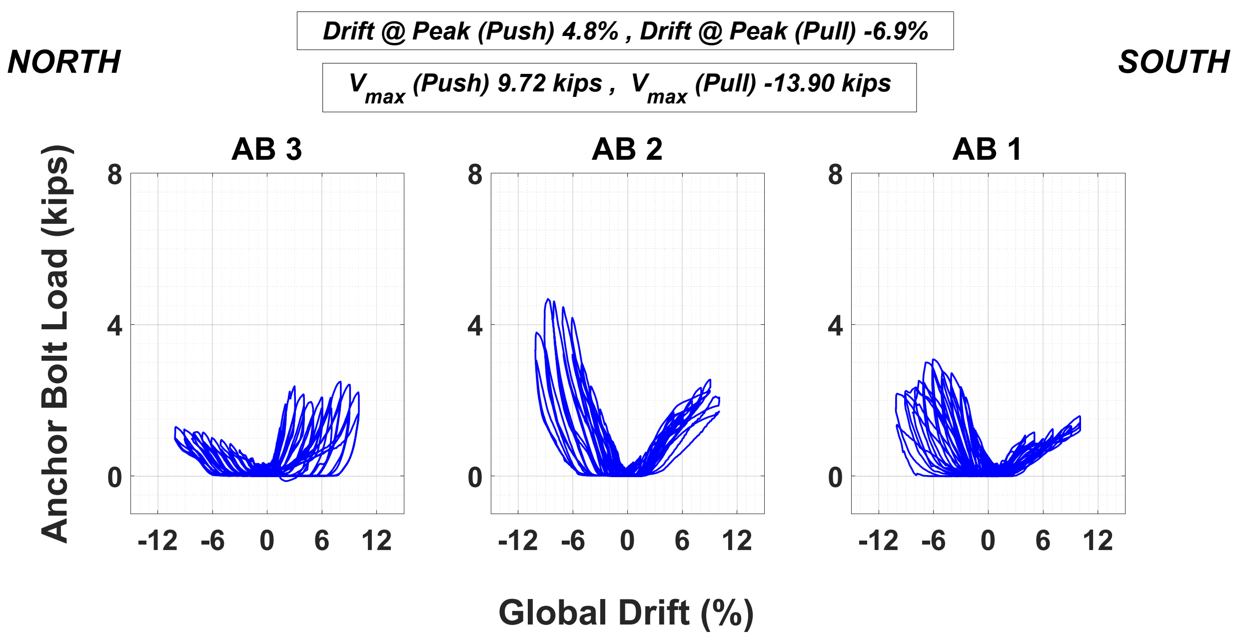


Figure C.11 Specimen A-28 anchor bolt loads versus global drift for the existing 2-ft-tall cripple wall with horizontal siding over diagonal sheathing and light vertical load.

C.2 DIAGONAL DISTORTION MEASUREMENTS

Two pairs of linear displacement potentiometers were used to measure the diagonal distortion of the cripple walls during testing. One pair, shown in Figure C.12 and denoted as D1 and D2, measured the distortion of the middle third of the cripple wall. These are referred to as the inner diagonal measurements. The other pair, denoted as D3 and D4, measured the distortion across the entire cripple wall. These are referred to as the outer diagonal measurements. The diagonal measurements are useful in determining the amount of shear distortion experienced by the cripple wall during testing. When coupled with the uplift measurements, LP04 and LP05, the amount of lateral displacement of the cripple wall can be resolved and compared to the measured lateral displacement. Figure C.13 gives a schematic for the how the resolved lateral displacements from diagonal and uplift measurements were derived.

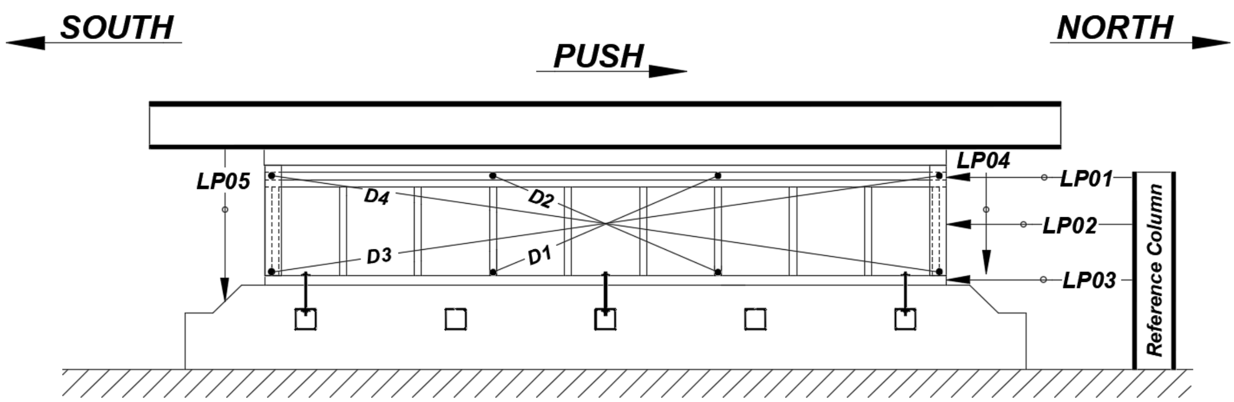


Figure C.12 Diagonal, end uplift, and lateral displacement potentiometer schematic.

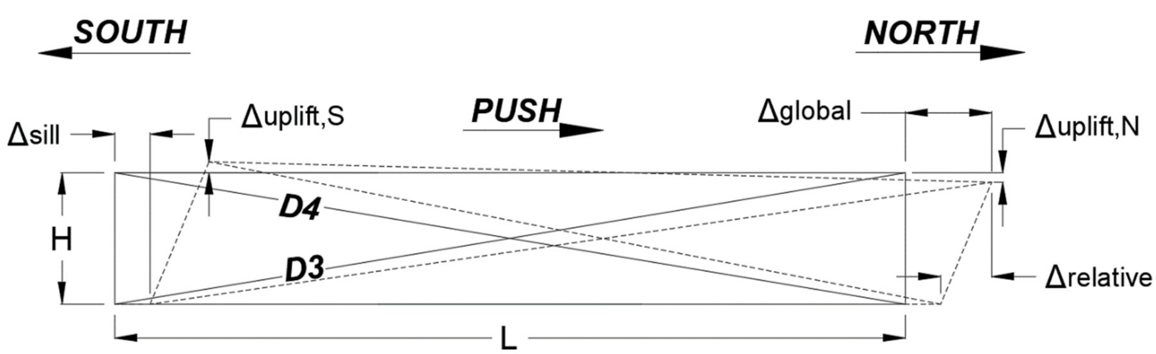


Figure C.13 Deformed cripple wall with measurements used for resolving lateral displacement from diagonal and uplift measurements.

The resolved lateral displacement from the diagonal and end uplift potentiometer measurements is as follows:

Undeformed diagonal lengths

$$L_{D30} = L_{D40} = \sqrt{L^2 + H^2}$$

$$L_{D10} = L_{D20} = \sqrt{\left(\frac{L}{3}\right)^2 + H^2}$$

$$where, \quad L = horizontal\ distance\ between\ D3\ and\ D4,$$
$$H = vertical\ distance\ between\ D3\ and\ D4$$

Diagonal measurement relationship

$$D1 = L_{D1} - L_{D10}$$
$$D2 = L_{D2} - L_{D20}$$
$$D3 = L_{D3} - L_{D30}$$
$$D4 = L_{D4} - L_{D40}$$

$$where, \quad D1, D2, D3, and\ D4\ are\ the\ diagonal\ measurements$$
$$and\ L_{D1}, L_{D2}, L_{D3}, and\ L_{D4}\ are\ the\ deformed\ lengths\ of\ the\ diagonals$$

Assume the uplift is linear across the entire wall. Therefore, the uplift at locations of D1, D2, D3, and D4 measurements can be linearly interpolated:

$$\Delta_{uplift}(x) = \Delta_{uplift,N} + \frac{(\Delta_{uplift,S} - \Delta_{uplift,N})}{L + 2L_{end}}(x)$$

$where\ L_{end}$
$= horiztonal\ distance\ from\ the\ uplift\ measurement\ to\ the\ outside\ diagonal\ measurement$

$$For\ D1: \quad x = \frac{2L}{3} + L_{end} \ \therefore \Delta_{uplift,D1} = \Delta_{uplift,N} + \frac{(\Delta_{uplift,S} - \Delta_{uplift,N})}{L + 2L_{end}} * (\frac{2L}{3} + L_{end})$$

$$For\ D2: \quad x = \frac{L}{3} + L_{end} \ \therefore \Delta_{uplift,D2} = \Delta_{uplift,N} + \frac{(\Delta_{uplift,S} - \Delta_{uplift,N})}{L + 2L_{end}} * (\frac{L}{3} + L_{end})$$

$$For\ D3: \quad x = L + L_{end} \ \therefore \Delta_{uplift,D3} = \Delta_{uplift,N} + \frac{(\Delta_{uplift,S} - \Delta_{uplift,N})}{L + 2L_{end}} * (L + L_{end})$$

$$For\ D4: \quad x = L_{end} \ \therefore \Delta_{uplift,D4} = \Delta_{uplift,N} + \frac{(\Delta_{uplift,S} - \Delta_{uplift,N})}{L + 2L_{end}} * (L_{end})$$

$$where\ \Delta_{uplift,N}\ is\ measured\ from\ LP04\ and\ \Delta_{uplift,S}\ are\ measured\ from\ LP05$$

Deformed diagonal lengths (sample calculation for D1)

$$L_{D1} = \sqrt{(\frac{L}{3} - \Delta_{relative})^2 + (H + \Delta_{uplift,D1})^2}$$

$$L_{D2} = \sqrt{(\frac{L}{3} + \Delta_{relative})^2 + (H + \Delta_{uplift,D2})^2}$$

$$L_{D3} = \sqrt{(L - \Delta_{relative})^2 + (H + \Delta_{uplift,D3})^2}$$

$$L_{D4} = \sqrt{(L + \Delta_{relative})^2 + (H + \Delta_{uplift,D4})^2}$$

$$where\ \Delta_{relative}\ is\ positive\ in\ the\ push\ direction\ and\ negative\ in\ the\ pull\ direction$$

Vertical component of uplift measurements

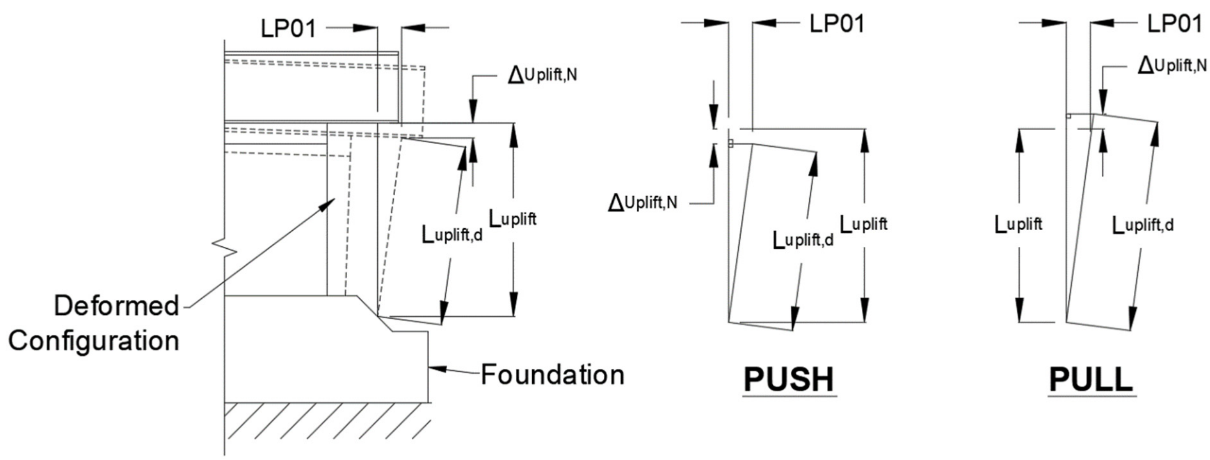


Figure C.14 Schematic for resolving end of wall uplift

$$Let,\quad L_{uplift} = length\ of\ uplift\ measurement\ string,\ L_{uplift,d} = deformed\ length\ of\ uplift\ measurement\ string$$

Push loading

$$L_{uplift,d} = L_{uplift} + LP04$$

$$L_{uplift,d}^2 = \left(L_{uplift} - \Delta_{uplift,S}\right)^2 + LP01^2$$

$$\Rightarrow \left(L_{uplift} + LP04\right)^2 = \left(L_{uplift} - \Delta_{uplift,S}\right)^2 + LP01^2$$

$$\Rightarrow \left(L_{uplift} - \Delta_{uplift,S}\right)^2 = \left(L_{uplift} + LP04\right)^2 - LP01^2$$

$$\therefore \Delta_{uplift,S} = L_{uplift} - \sqrt{\left(L_{uplift} + LP04\right)^2 - LP01^2}$$

$$\therefore \Delta_{uplift,N} = L_{uplift} - \sqrt{\left(L_{uplift} + LP05\right)^2 - LP01^2}$$

Pull loading

$$L_{uplift,d} = L_{uplift} + LP04$$

$$L_{uplift,d}^2 = \left(L_{uplift} + \Delta_{uplift,S}\right)^2 + LP01^2$$

$$\Rightarrow \left(L_{uplift} + LP04\right)^2 = \left(L_{uplift} + \Delta_{uplift,S}\right)^2 + LP01^2$$

$$\Rightarrow \left(L_{uplift} + \Delta_{uplift,S}\right)^2 = \left(L_{uplift} + LP04\right)^2 - LP01^2$$

$$\therefore \Delta_{uplift,S} = \sqrt{\left(L_{uplift} + LP04\right)^2 - LP01^2} - L_{uplift}$$

$$\therefore \Delta_{uplift,N} = \sqrt{\left(L_{uplift} + LP05\right)^2 - LP01^2} - L_{uplift}$$

Solving for relative displacements as a function of uplift and diagonal measurements

$$D1 = L_{D1} - L_{D10} \Rightarrow D1 = \sqrt{\left(\frac{L}{3} + \Delta_{relative}\right)^2 + \left(H + \Delta_{uplift,D1}\right)^2} - \sqrt{\left(\frac{L}{3}\right)^2 + H^2}$$

$$\Rightarrow D1 + \sqrt{\left(\frac{L}{3}\right)^2 + H^2} = \sqrt{\left(\frac{L}{3} + \Delta_{relative}\right)^2 + \left(H + \Delta_{uplift,D1}\right)^2}$$

$$\Rightarrow D1^2 + 2D1\sqrt{\left(\frac{L}{3}\right)^2 + H^2} + \left(\frac{L}{3}\right)^2 + H^2 = \left(\frac{L}{3} + \Delta_{relative}\right)^2 + \left(H + \Delta_{uplift,D1}\right)^2$$

$$\Rightarrow D1^2 + 2D1\sqrt{\left(\frac{L}{3}\right)^2 + H^2} + \left(\frac{L}{3}\right)^2 + H^2 - \left(H + \Delta_{uplift,D1}\right)^2 = \left(\frac{L}{3} + \Delta_{relative}\right)^2$$

$$\Rightarrow \sqrt{D1^2 + 2D1\sqrt{\left(\frac{L}{3}\right)^2 + H^2} + \left(\frac{L}{3}\right)^2 + H^2 - \left(H + \Delta_{uplift,D1}\right)^2} = \frac{L}{3} + \Delta_{relative}$$

Resolved lateral displacements as a function of the uplift and diagonal measurements

$$\therefore\ \Delta_{relative} = \sqrt{D1^2 + 2D1\sqrt{\left(\frac{L}{3}\right)^2 + H^2} + \left(\frac{L}{3}\right)^2 + H^2 - \left(H + \Delta_{uplift,D1}\right)^2} - \frac{L}{3} \quad for\ D1$$

$$\therefore\ \Delta_{relative} = \frac{L}{3} - \sqrt{D2^2 + 2D2\sqrt{\left(\frac{L}{3}\right)^2 + H^2} + \left(\frac{L}{3}\right)^2 + H^2 - \left(H + \Delta_{uplift,D2}\right)^2} \quad for\ D2$$

$$\therefore\ \Delta_{relative} = \sqrt{D3^2 + 2D3\sqrt{L^2 + H^2} + L^2 + H^2 - \left(H + \Delta_{uplift,D3}\right)^2} - L \quad for\ D3$$

$$\therefore\ \Delta_{relative} = L - \sqrt{D2^2 + 2D2\sqrt{L^2 + H^2} + L^2 + H^2 - \left(H + \Delta_{uplift,D4}\right)^2} \quad for\ D4$$

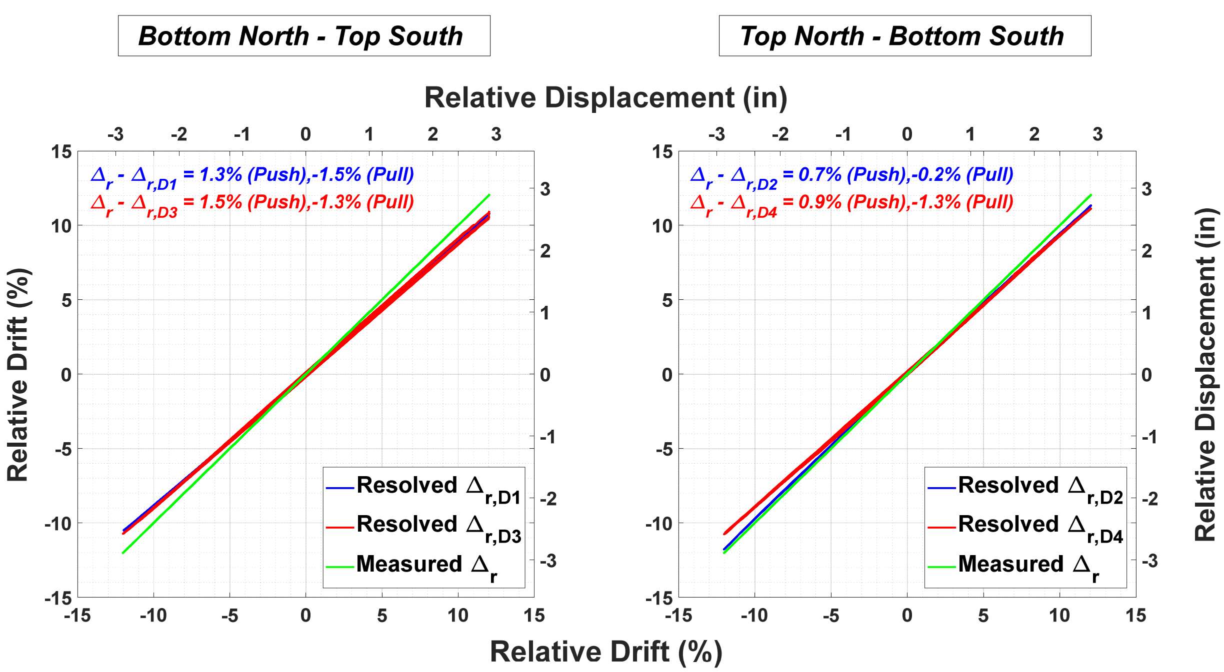


Figure C.15 **Specimen A-7 resolved relative drift from diagonal measurements in one direction versus measured relative drift for the existing 2-ft-tall cripple wall with horizontal siding.**

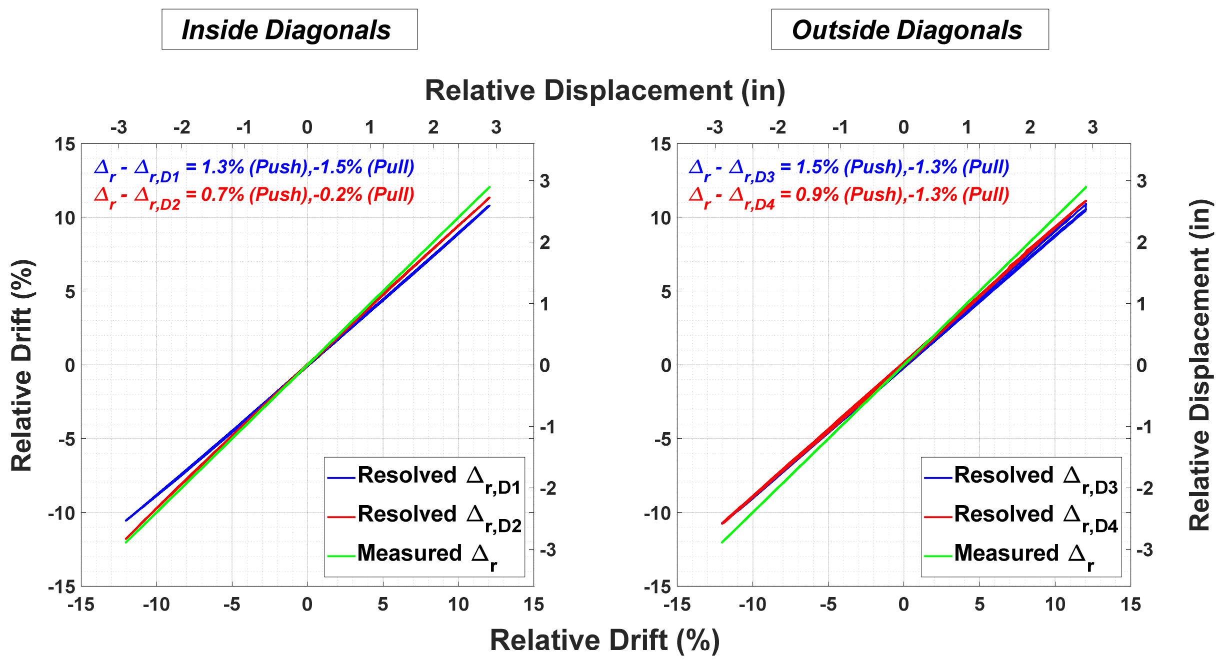


Figure C.16 Specimen A-7 resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for existing 2-ft-tall cripple wall with horizontal siding.

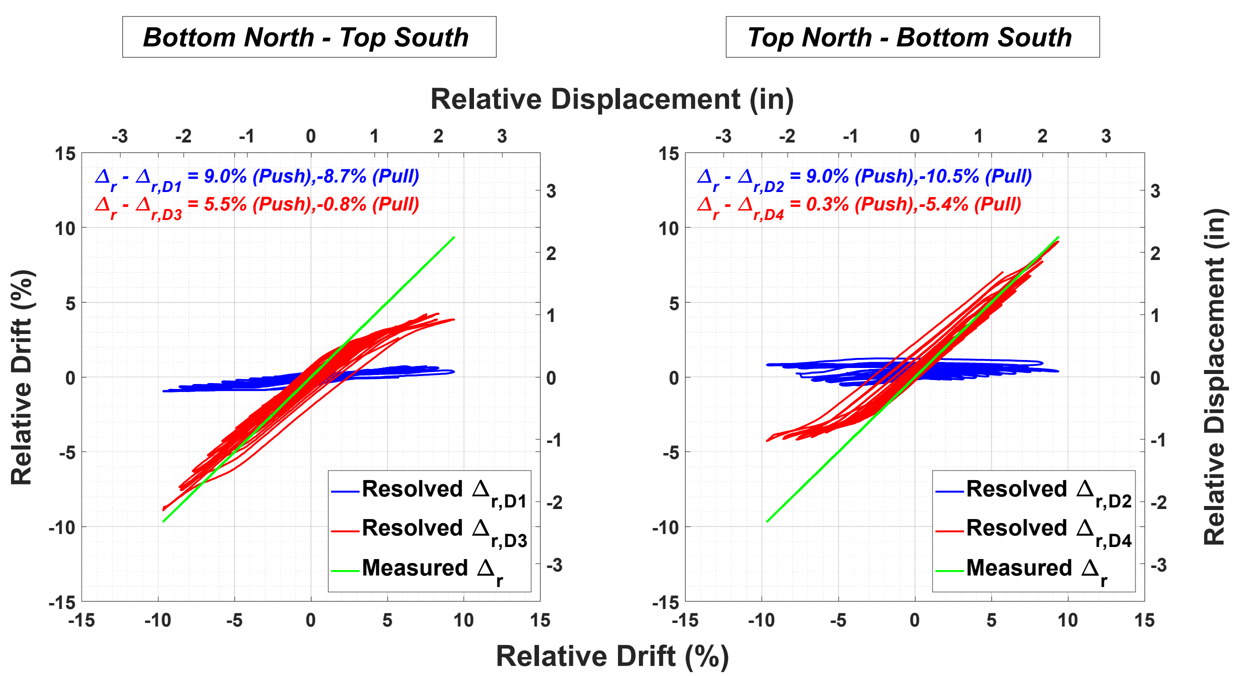


Figure C.17 Specimen A-8 resolved relative drift from diagonal measurements in one direction versus measured relative drift for 2-foot-tall, retrofitted cripple wall with horizontal siding.

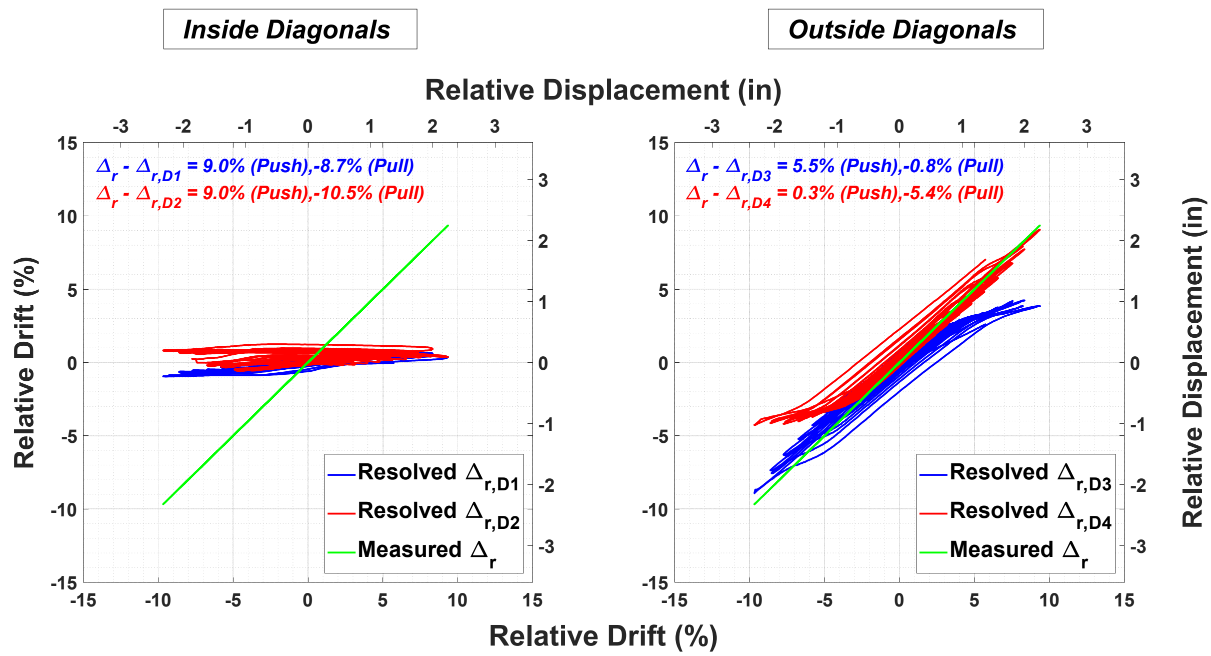


Figure C.18 Specimen A-8 resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for the retrofitted 2-ft-tall cripple wall with horizontal siding.

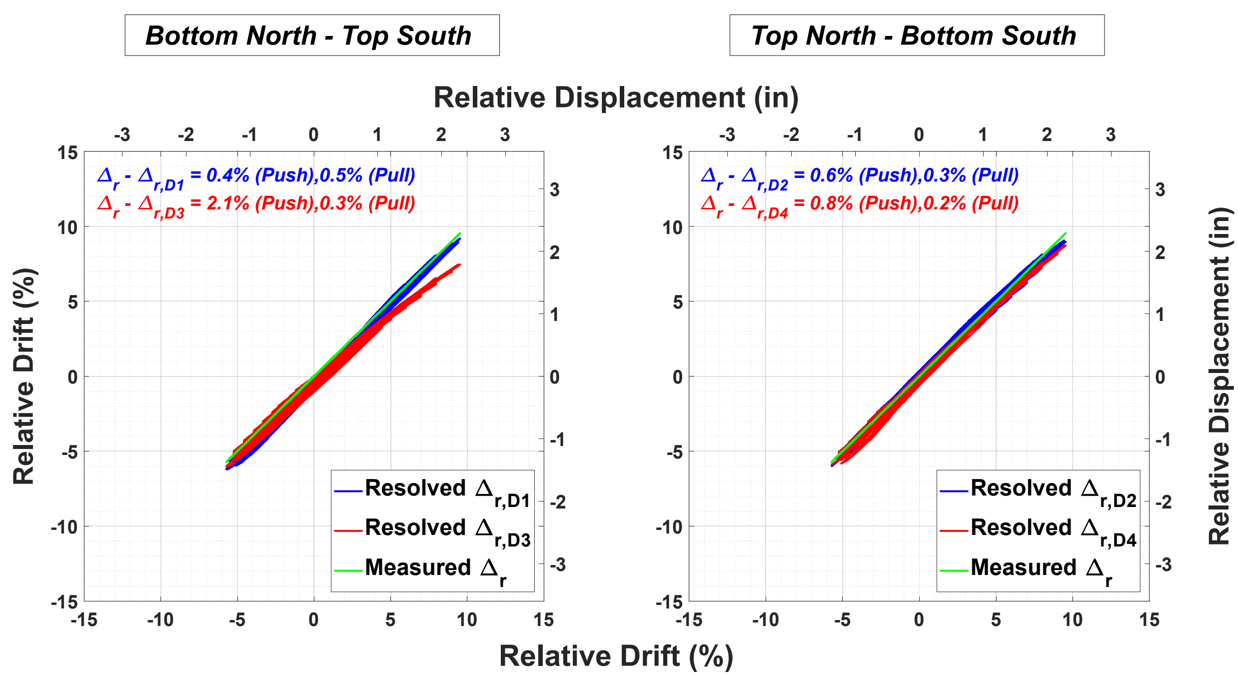


Figure C.19 Specimen A-9 resolved relative drift from diagonal measurements in one direction versus measured relative drift for the existing 2-ft-tall cripple wall with horizontal siding over diagonal sheathing.

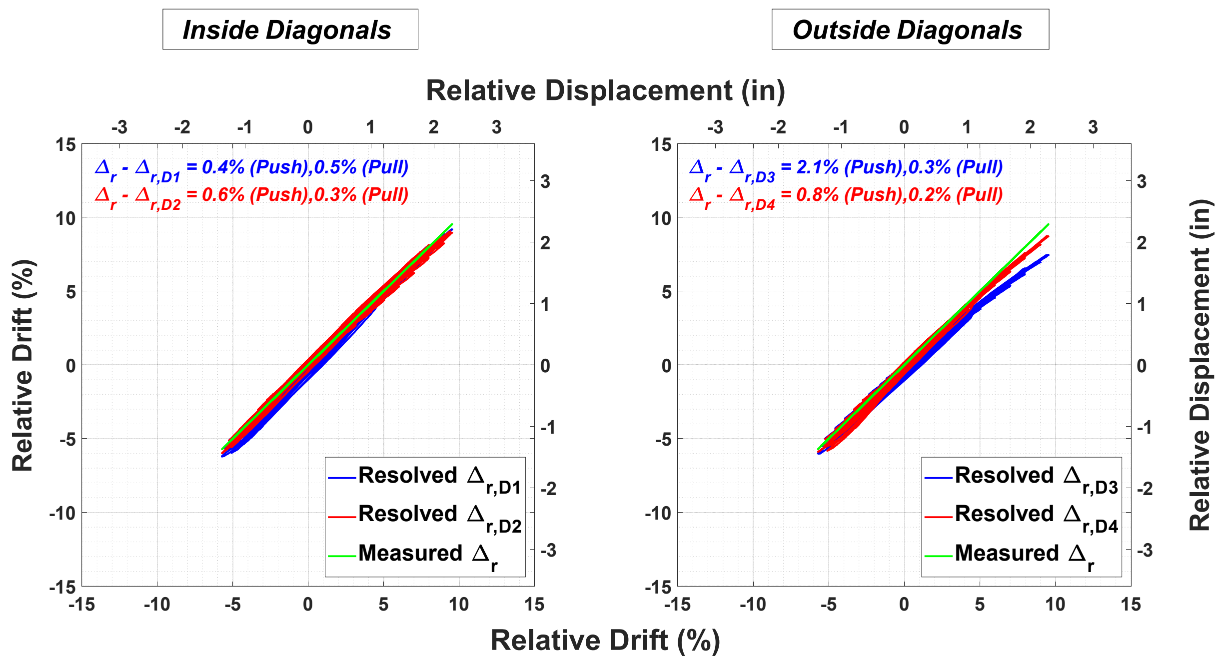


Figure C.20 Specimen A-9 resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for existing 2-ft-tall cripple wall with horizontal siding over diagonal sheathing.

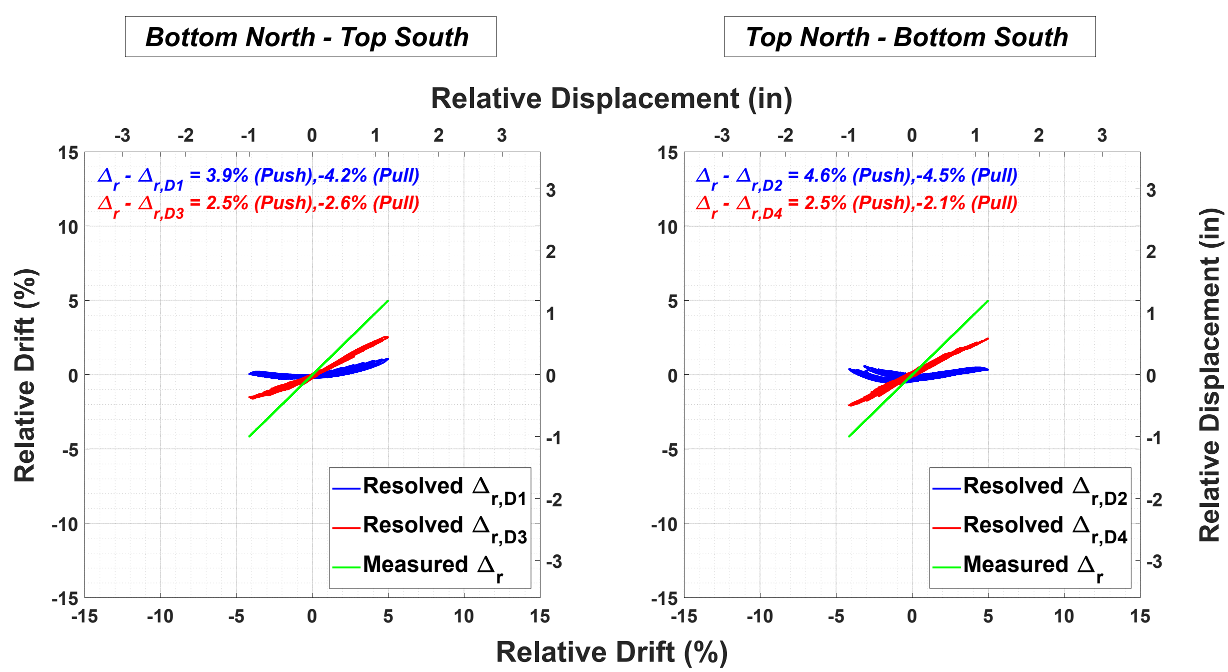


Figure C.21 Specimen A-10 resolved relative drift from diagonal measurements in one direction versus measured relative drift for the retrofitted 2-ft-tall cripple wall with horizontal siding over diagonal sheathing.

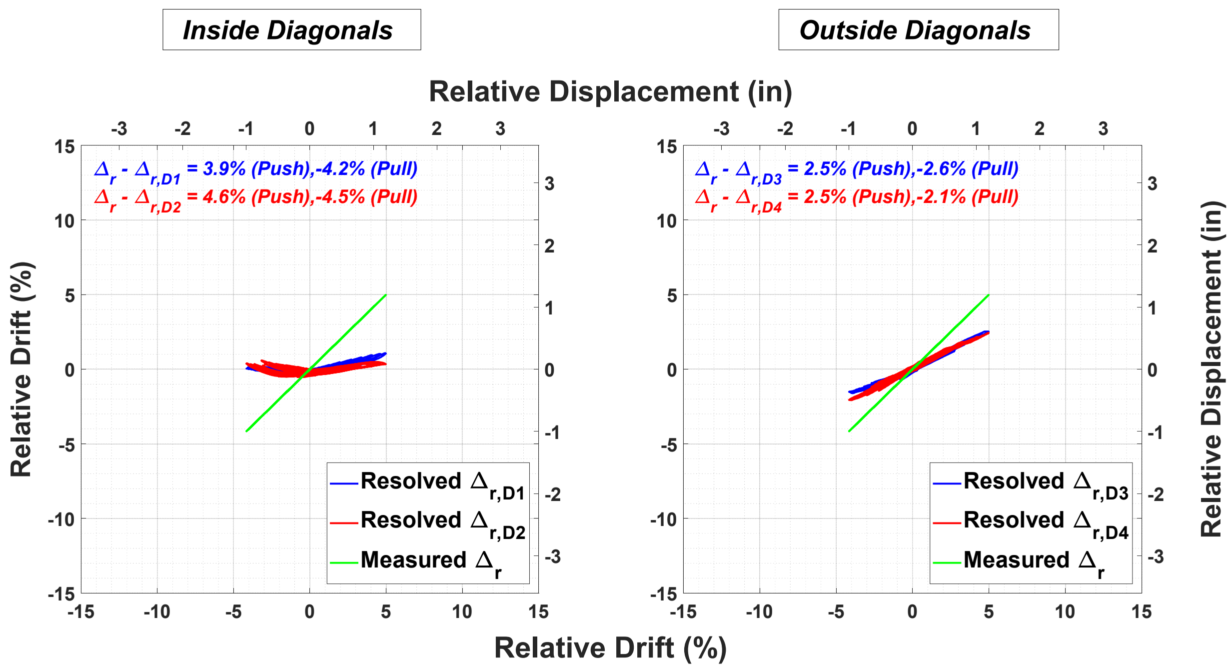


Figure C.22 Specimen A-10 resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for the retrofitted 2-ft-tall cripple wall with horizontal siding over diagonal sheathing.

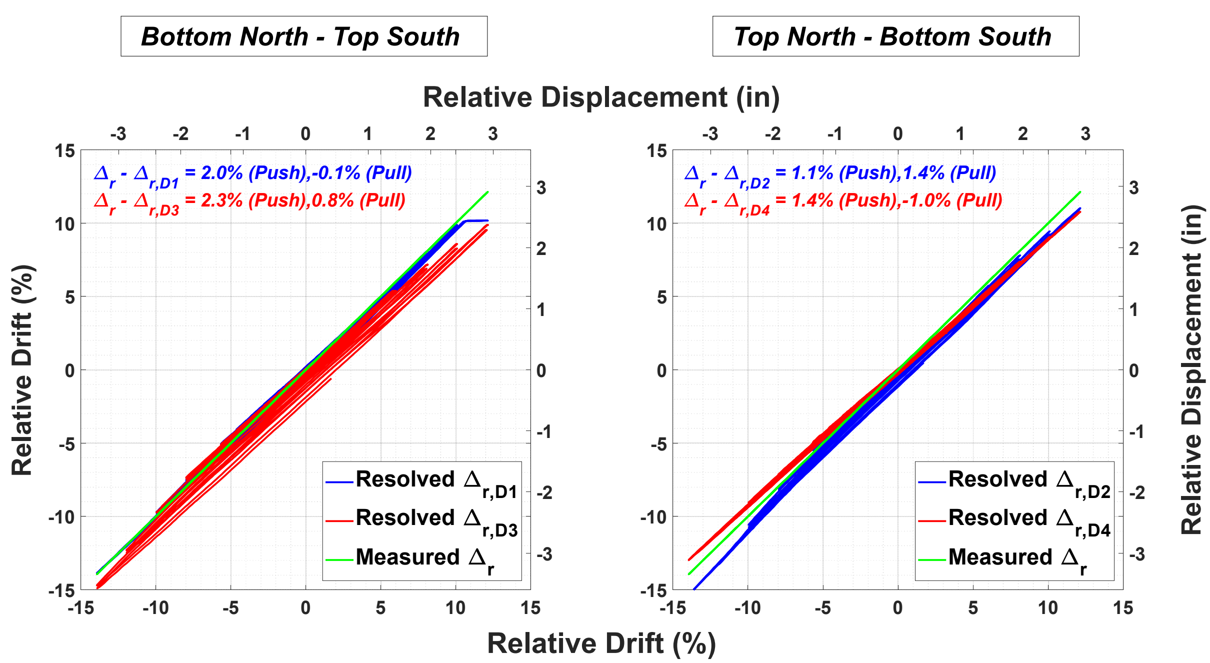


Figure C.23 Specimen A-11 resolved relative drift from diagonal measurements in one direction versus measured relative drift for the existing 2-ft-tall cripple wall with T1-11 plywood.

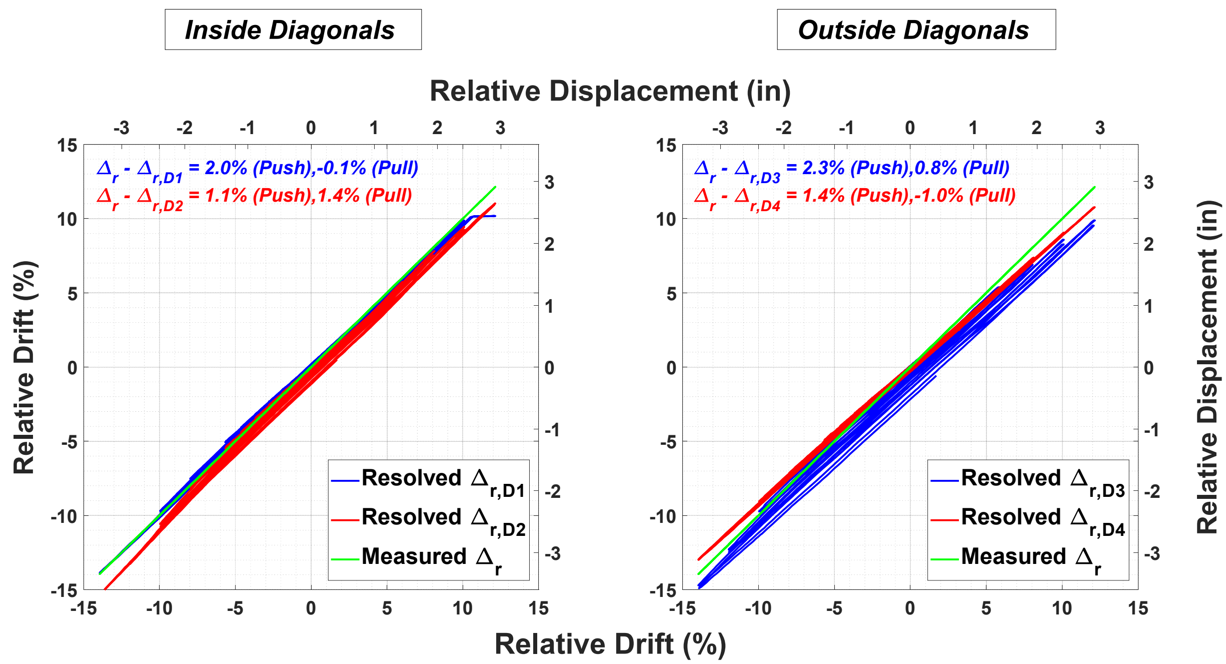


Figure C.24 Specimen A-11 resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for the existing 2-ft-tall cripple wall with T1-11 plywood.

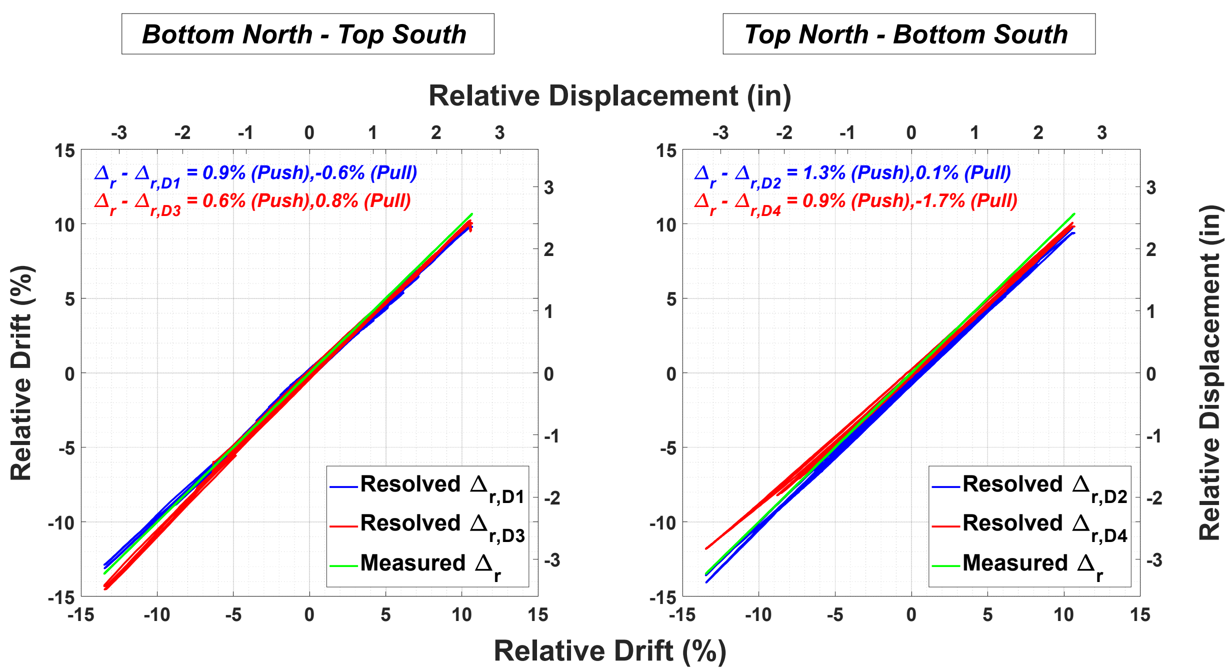


Figure C.25 Specimen A-12 resolved relative drift from diagonal measurements in one direction versus measured relative drift for the retrofitted 2-ft-tall cripple wall with T1-11 plywood.

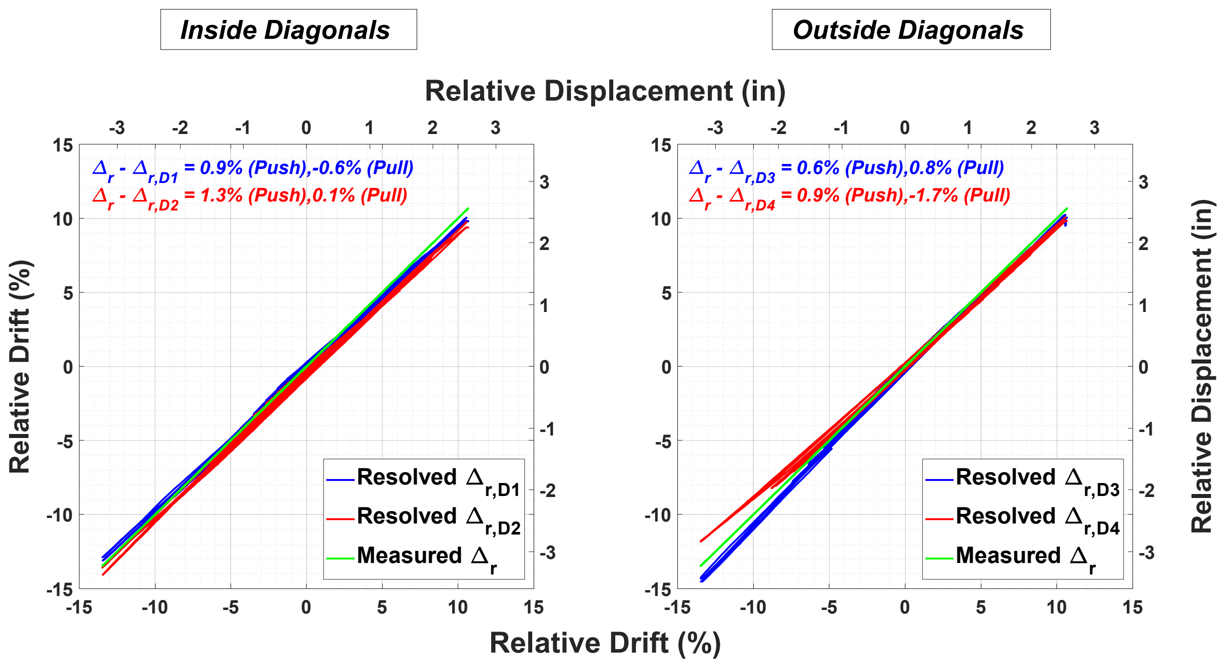


Figure C.26 **Specimen A-12 resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for the retrofitted 2-ft-tall cripple wall with T1-11 plywood.**

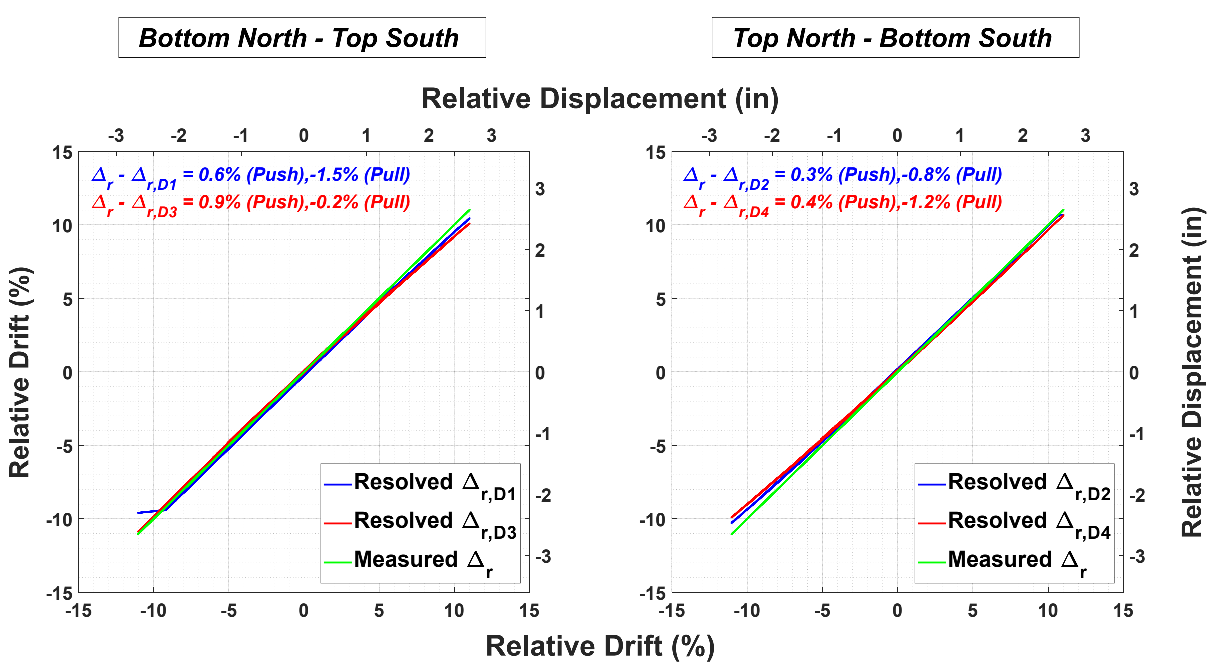


Figure C.27 **Specimen A-13 resolved relative drift from diagonal measurements in one direction versus measured relative drift for the existing 6-ft-tall cripple wall with horizontal siding.**

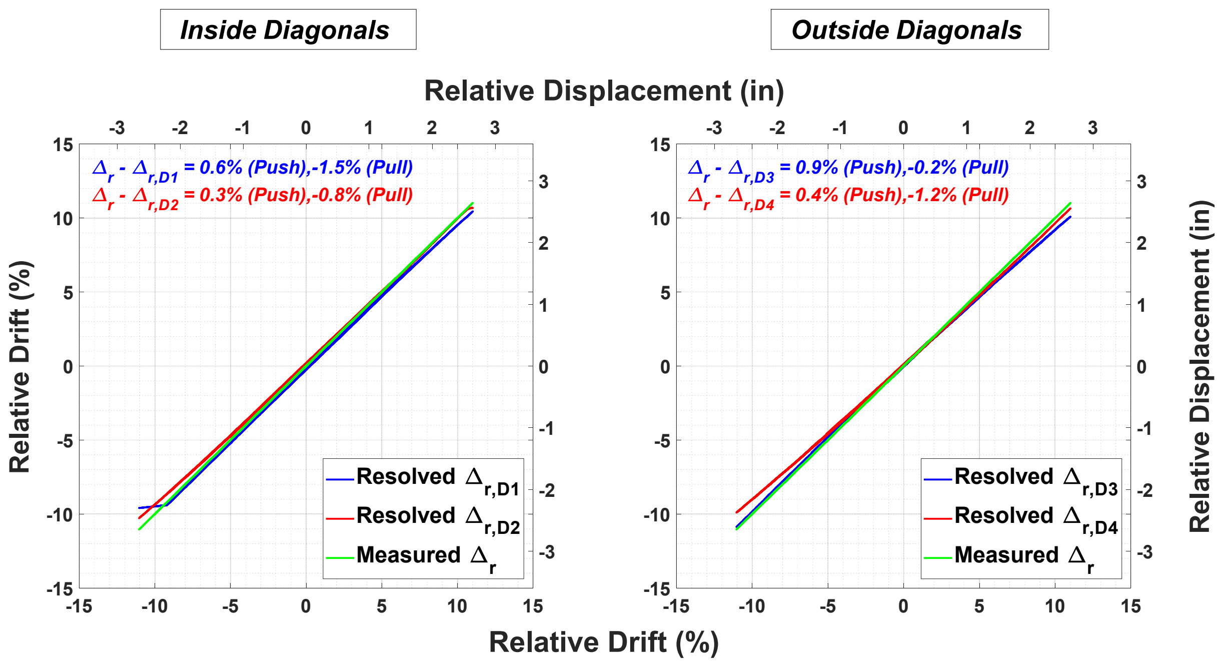


Figure C.28 Specimen A-13 resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for the existing 6-ft-tall cripple wall with horizontal siding.

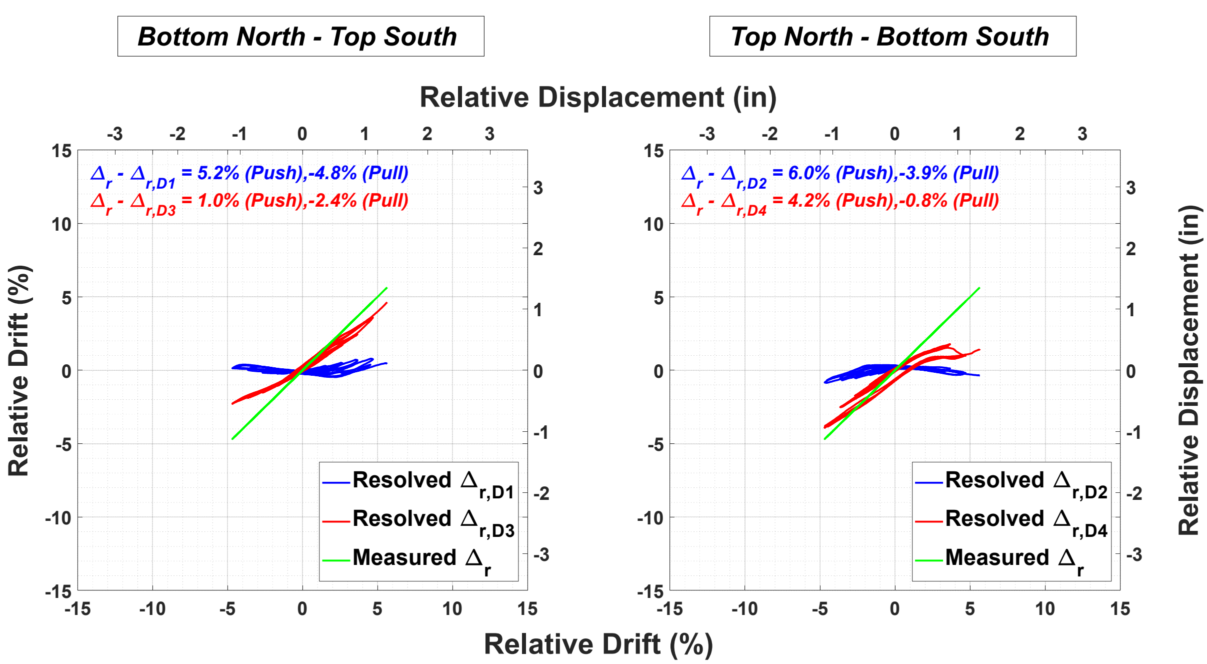


Figure C.29 Specimen A-14 resolved relative drift from diagonal measurements in one direction versus measured relative drift for the retrofitted 6-ft-tall cripple wall with horizontal siding.

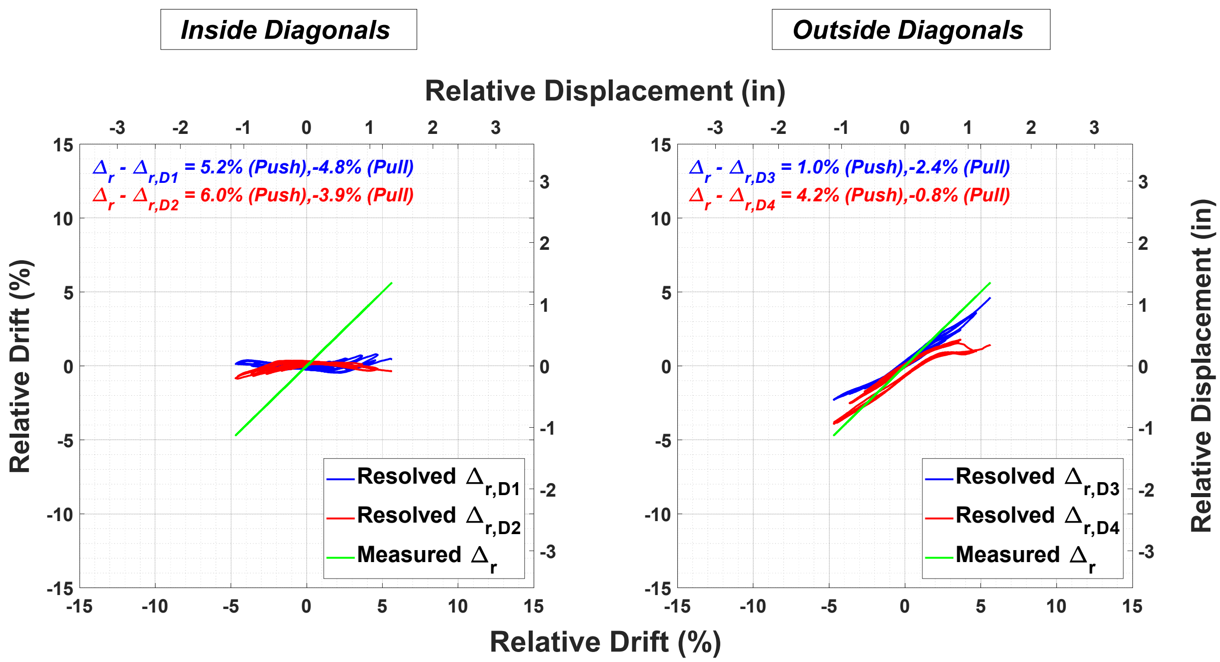


Figure C.30 Specimen A-14 resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for the retrofitted 6-ft-tall cripple wall with horizontal siding.

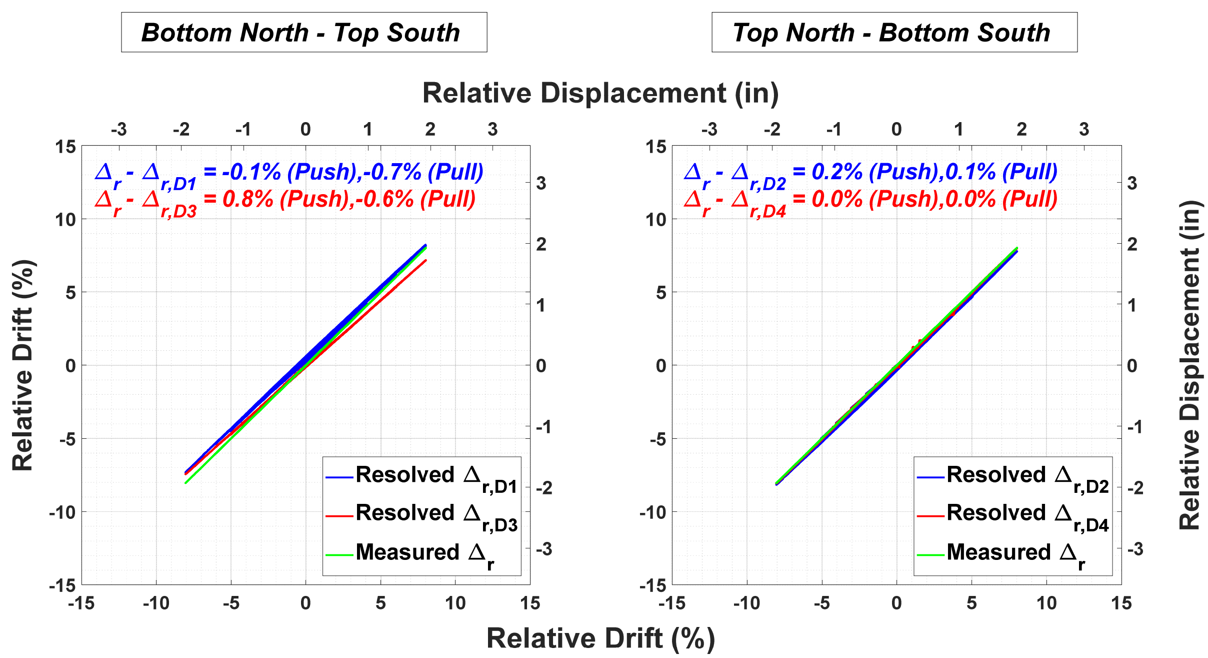


Figure C.31 Specimen A-23 resolved relative drift from diagonal measurements in one direction versus measured relative drift for the existing 6-ft-tall cripple wall with T1-11 plywood.

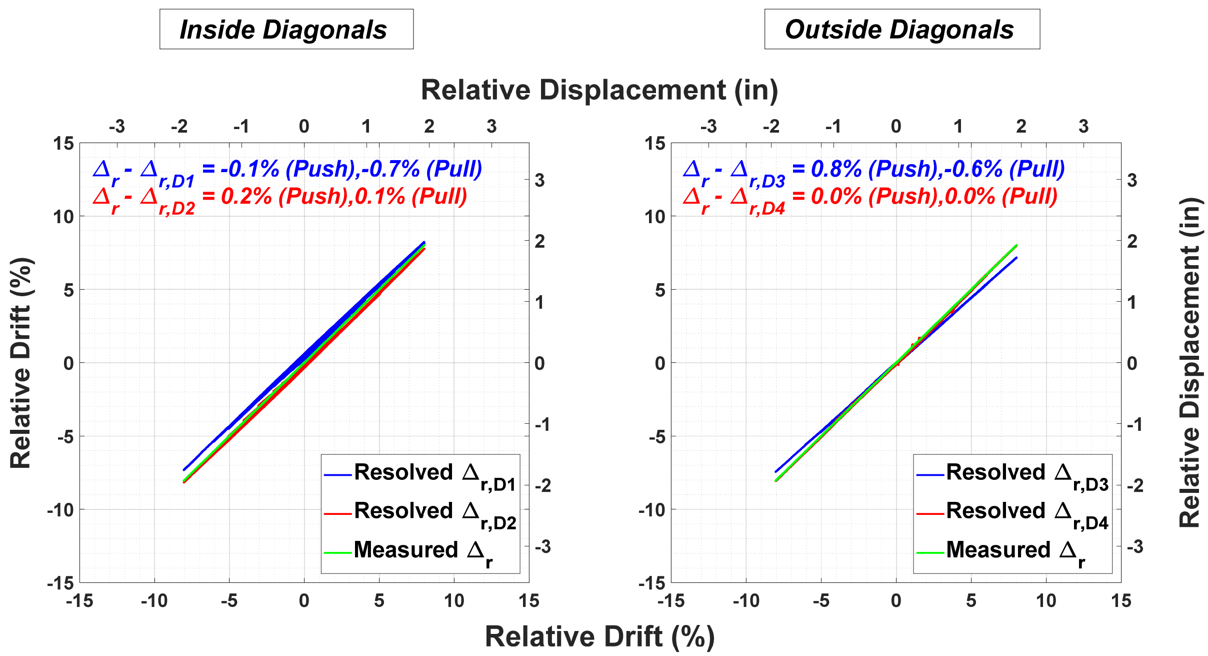


Figure C.32 Specimen A-23 resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for the existing 6-ft-tall cripple wall with T1-11 plywood (Specimen A-23)

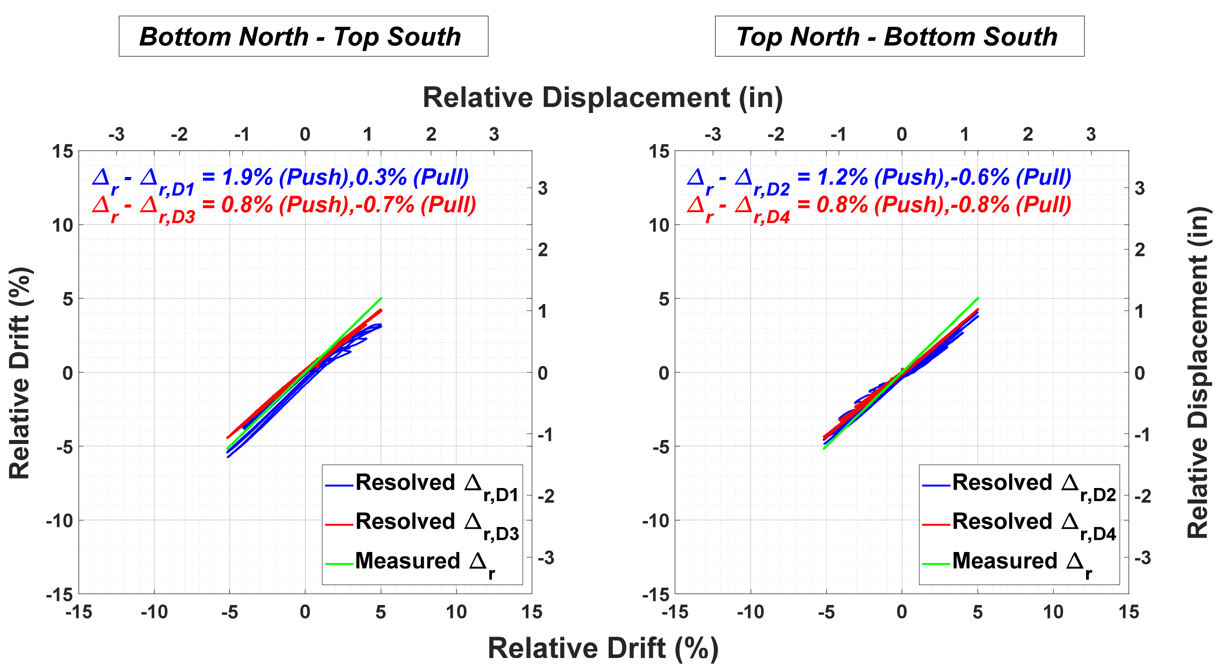


Figure C.33 Specimen A-24 resolved relative drift from diagonal measurements in one direction versus measured relative drift for 6-foot-tall, retrofitted cripple wall with T1-11 plywood.

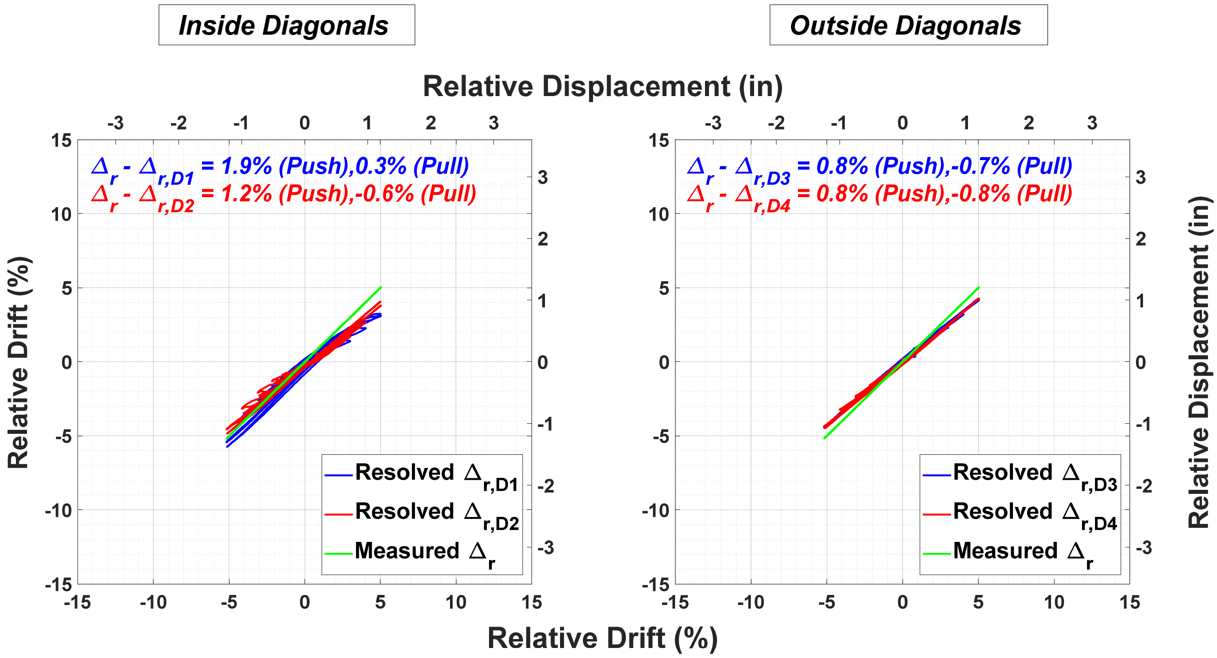


Figure C.34 Specimen A-24 resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for the retrofitted 6-ft-tall cripple wall with T1-11 plywood.

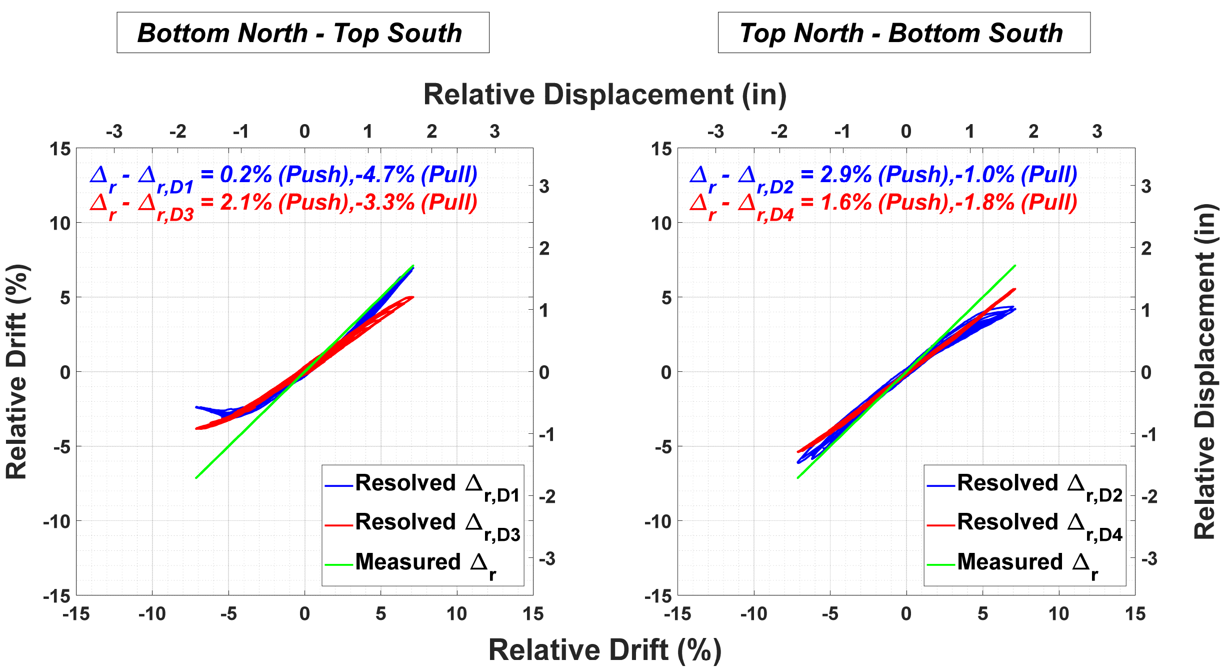


Figure C.35 Specimen A-28 resolved relative drift from diagonal measurements in one direction versus measured relative drift for the retrofitted 2-ft-tall cripple wall with horizontal siding over diagonal sheathing and low vertical load.

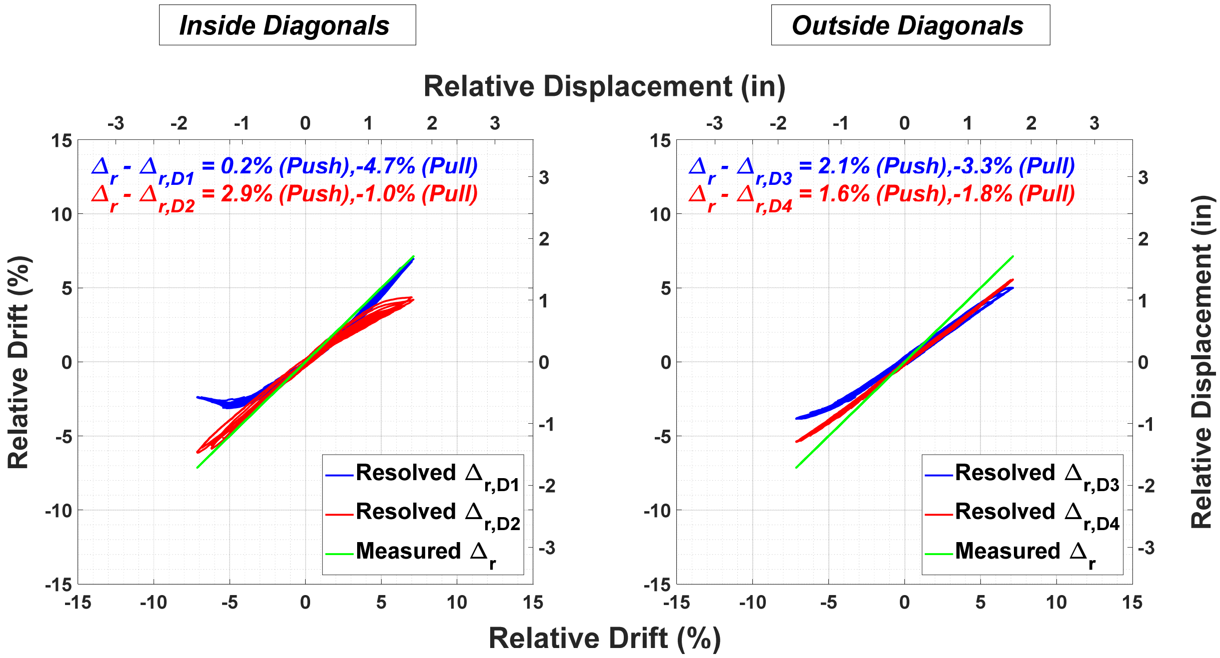


Figure C.36 **Specimen A-28 resolved relative drift from diagonal measurements (outside and inside diagonals) versus measured relative drift for the retrofitted 6-ft-tall cripple wall with horizontal siding over diagonal sheathing and low vertical load.**

C.3 UPLIFT MEASUREMENTS

Two linear potentiometers were used to measure the uplift at both ends of the cripple wall. These potentiometers were attached to the foundation and the steel load transfer beam. The calculations for determining the uplift of the cripple walls is shown in the previous section as the uplift measurements were factored into calculating the resolved relative displacement from the diagonal measurements.

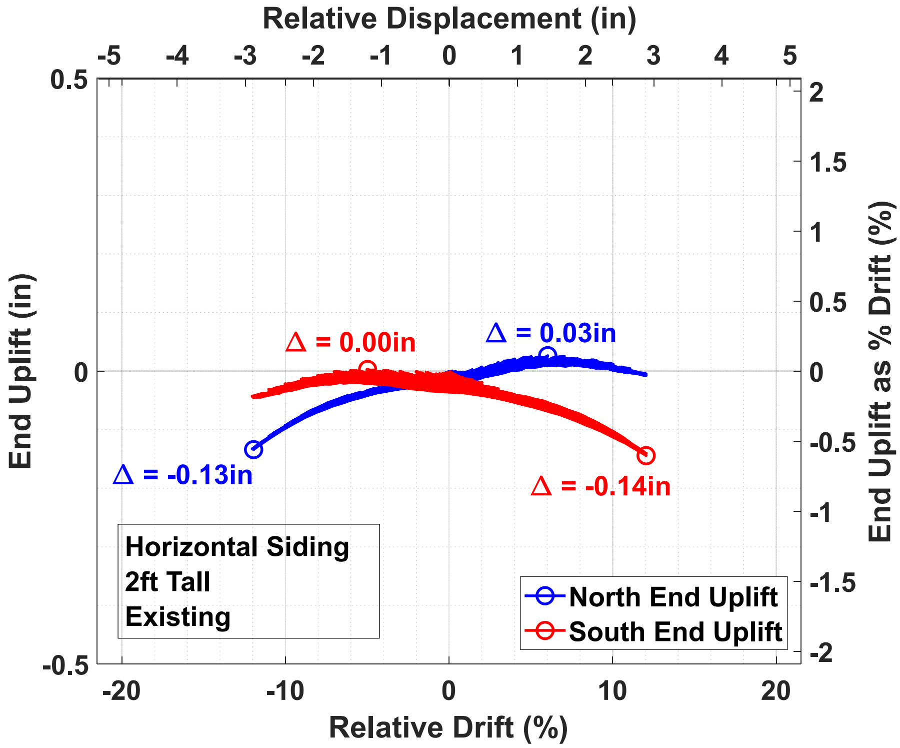


Figure C.37 Specimen A-7 end uplift versus relative drift for the existing 2-ft-tall cripple wall with horizontal siding.

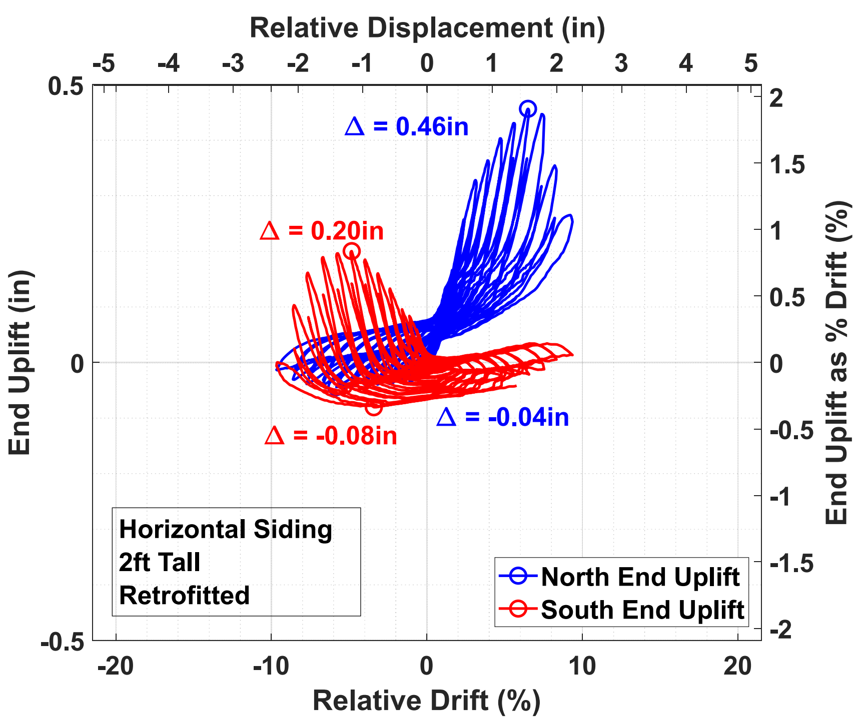


Figure C.38 Specimen A-8 end uplift versus relative drift for the retrofitted 2-ft-tall cripple wall with horizontal siding.

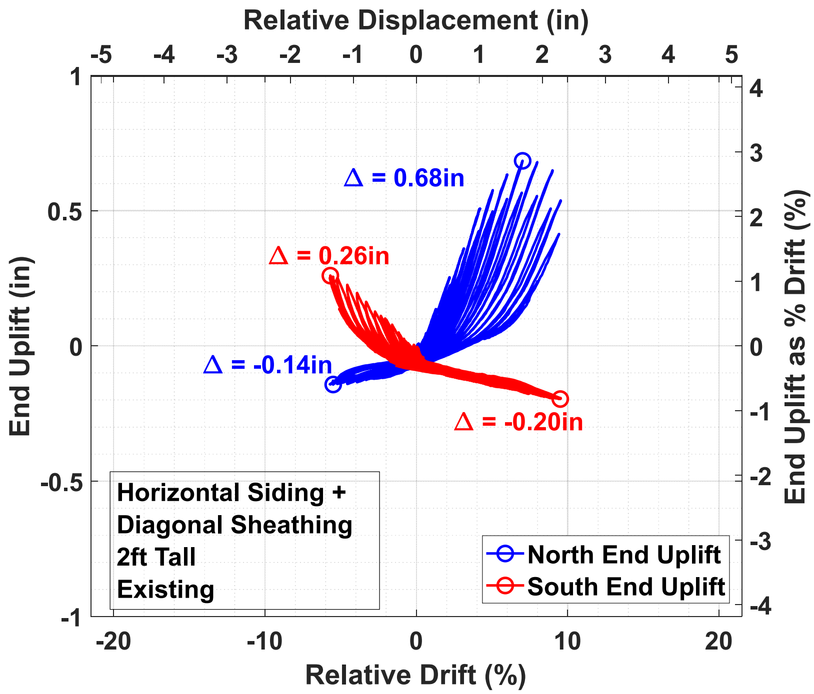


Figure C.39 Specimen A-9 end uplift versus relative drift for the existing 2-ft-tall cripple wall with horizontal siding over diagonal sheathing.

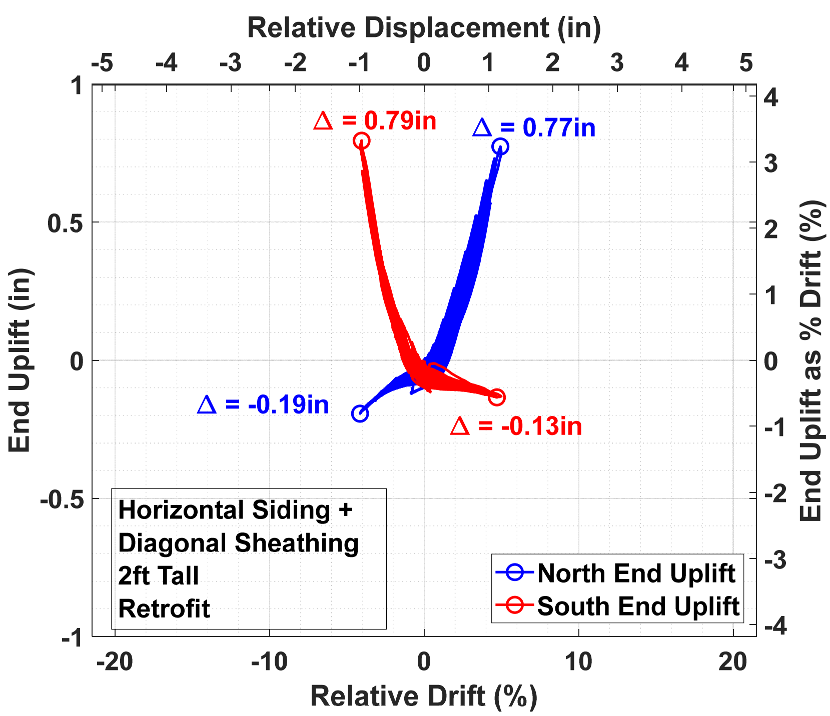


Figure C.40 Specimen A-10 end uplift versus relative drift for the retrofitted 2-ft-tall cripple wall with horizontal siding over diagonal sheathing.

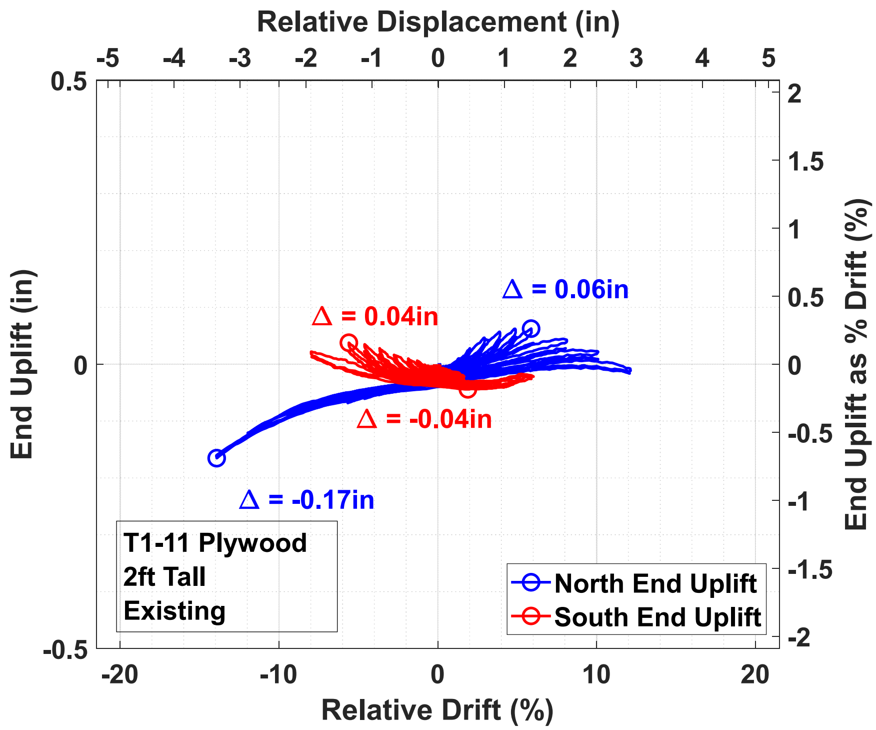


Figure C.41 Specimen A-11 end uplift versus relative drift for existing 2-ft-tall cripple wall with T1-11 plywood.

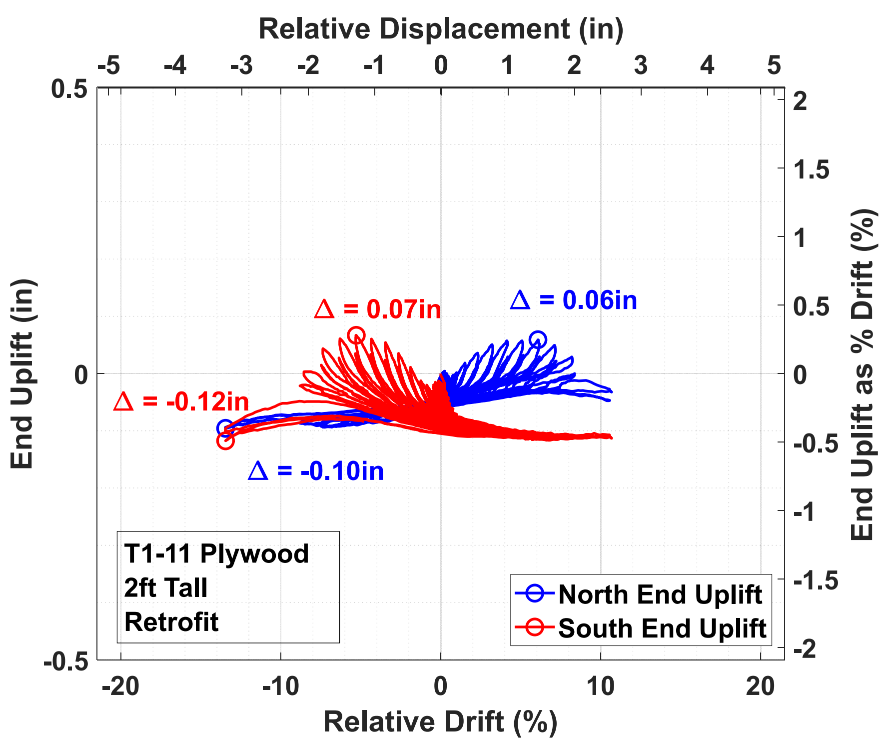


Figure C.42 Specimen A-12 end uplift versus relative drift for the retrofitted 2-ft-tall cripple wall with T1-11 plywood.

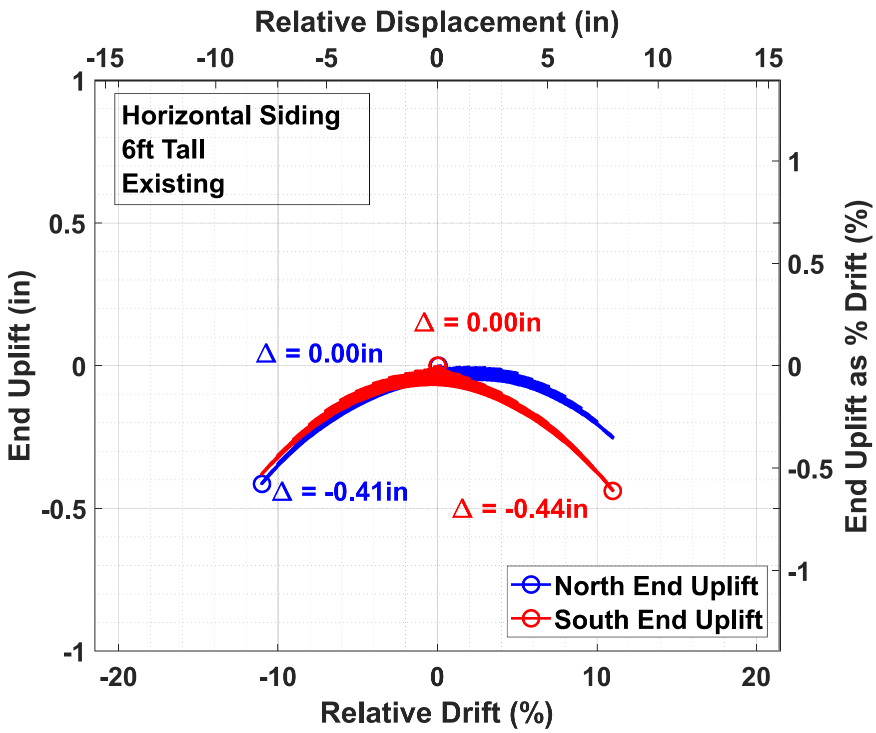


Figure C.43 Specimen A-13 end uplift versus relative drift for the existing 6-ft-tall cripple wall with horizontal siding.

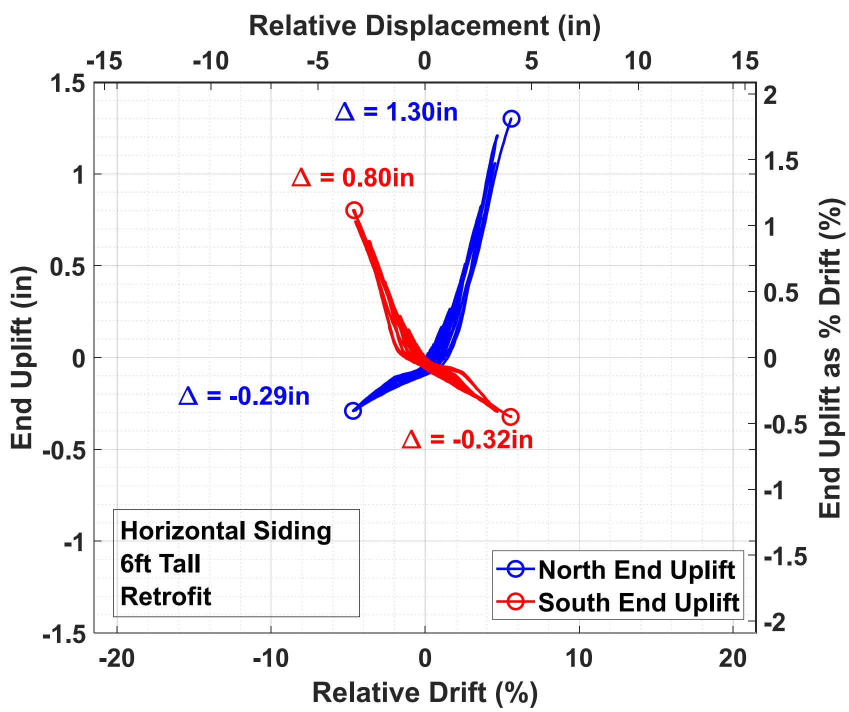


Figure C.44 Specimen A-14 end uplift versus relative drift for the retrofitted 6-ft-tall cripple wall with horizontal siding.

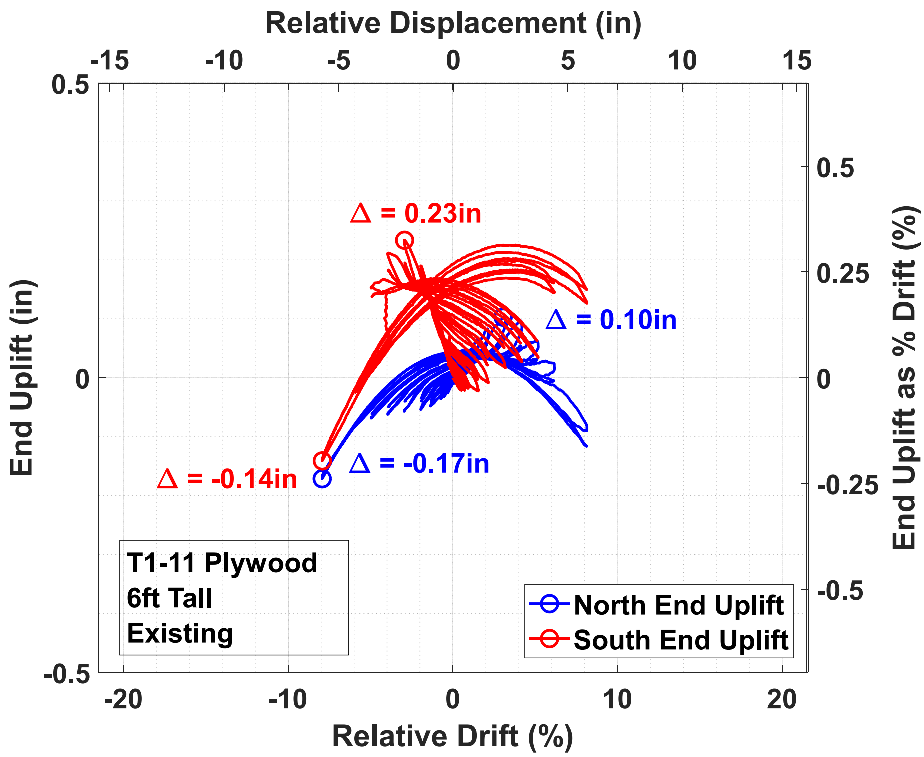


Figure C.45 Specimen A-23 end uplift versus relative drift for the existing 6-ft-tall cripple wall with T1-11 plywood.

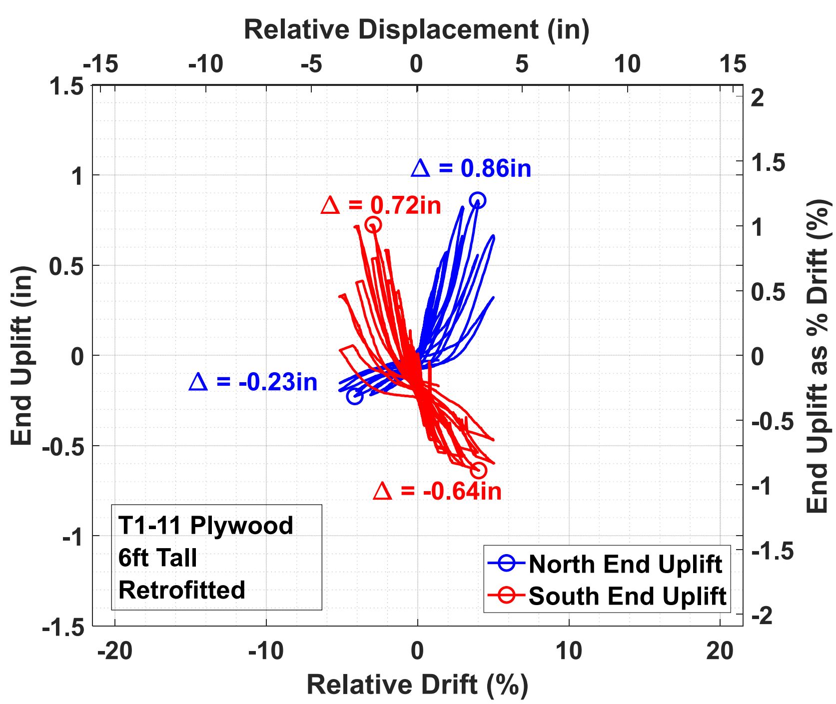


Figure C.46 Specimen A-24 end uplift versus relative drift for the retrofitted 6-ft-tall cripple wall with T1-11 plywood.

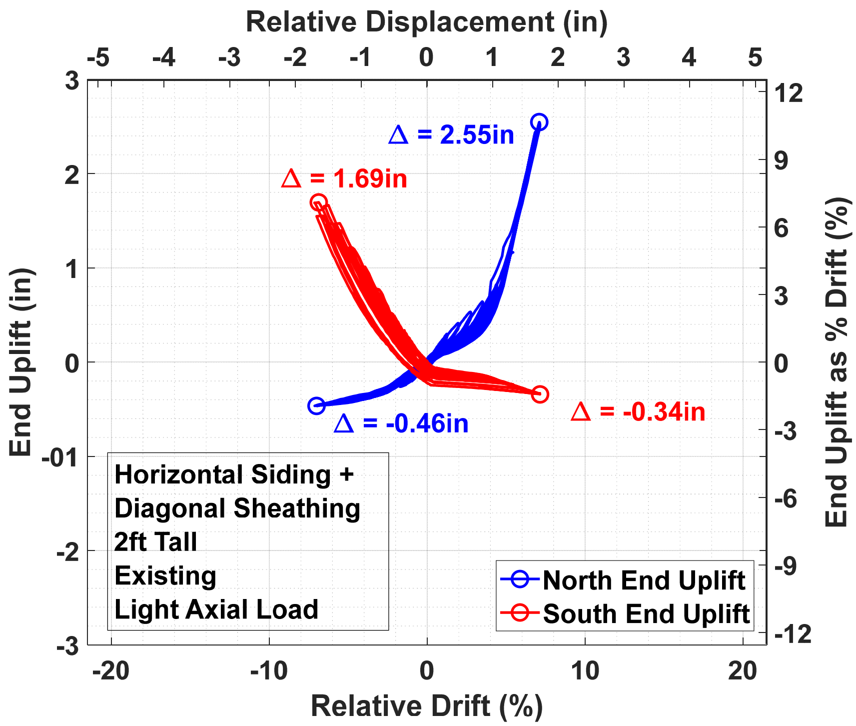


Figure C.47 Specimen A-28 end uplift versus relative drift for the existing 2-ft-tall cripple wall with horizontal siding over diagonal sheathing and light vertical load.

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