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QUASI-STATIC REVERSED CYCLIC RESPONSE OF WOOD SIDING FINISHED CRIPPLE WALLS: NUMERICAL MODELING

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Abstract

From field observations of past earthquakes, it has been found that inadequate lateral bracing of cripple walls and inadequate sill bolting in particular are often primary reasons for failure 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. To address this limitation, a multi-University-Industry collaborative experimental investigation was undertaken through the efforts of a Pacific Earthquake Engineering Research Center-California Earthquake Authority (PEER-CEA) program. The ultimate goal of the PEER-CEA effort was to develop improvements to seismic loss estimation models of older residential dwellings. The experimental component within the PEER-CEA effort consisted of both small and large component tests, the former conducted at UC San Diego, the later conducted at UC Berkeley. Experimental variables included cripple wall height, exterior and interior finishes, retrofit and unretrofitted conditions, to name a few.

In this paper, we focus on the development of numerical models of small component cripple wall assemblies based on the experimental results from the UC San Diego small component tests. These numerical investigations focus on specimens finished with horizontal shiplap siding both in their existing and retrofitted configurations, as well as the sensitivity of the modeling. Numerical models of the cripple walls utilize a two-dimensional modeling computer program called MATLAB toolbox for Cyclic Analysis of Shear Walls (M-CASHEW), a successor to CASHEW, developed within the Consortium of Universities for Research in Earthquake Engineering – Caltech Woodframe Research Project (CUREE-Caltech). The nonlinear response of fasteners within the model was captured using individual connector-level hysteretic spring elements. Results in this paper demonstrates a robust comparison between experimental and numerically predicted global hysteretic response. Further assessment of the numerical models is undertaken by comparing the energy dissipation.

Keywords: cripple wall; sill anchorage; nonlinear modeling

1. Introduction

Over the past four decades, moderate to large seismic events in California have caused severe damage to many single-family wood light-frame dwellings. Notable earthquakes include the 1989 Loma Prieta earthquake (M_W 6.9), the 1994 Northridge earthquake (M_W 6.8), and the 2014 South Napa earthquake (M_W 6.0). In the former two seismic events, there was significant damage to infrastructure with estimated costs in the tens of billions of dollars excluding business interruption and other indirect loses. The later, the 2014 South Napa earthquake, had a significantly smaller price tag associated with damage (between $500 million and $1 billion) as this was a more moderate seismic event and its epicenter being in further proximity from a major municipal area. Although most damage was non-structural, a considerable number of single-family wood-frame houses, predominately built pre-1950, suffered significant structural damage [1]. Due to the consistent risk posed by seismic events in California and the vulnerability of older, pre-1970, housing stock, there has been a push to retrofit susceptible wood-frame dwellings. In addition, extensive work has been done to quantify the threat of earthquakes to these dwelling as well as the added benefit provided by retrofitting homes.

At the end of 2020, a project aimed at quantifying the performance of existing and retrofitting dwellings was completed. The project, *Quantifying the Performance of Retrofit of Cripple Walls and Sill Anchorage in Single-Family Wood-Frame Buildings* was led by the Pacific Earthquake Engineering Research Center (PEER) and funded by the California Earthquake Authority (CEA). The multi-phase project was aimed towards quantifying damages to parts of wood-frame dwellings that are the root cause of most major structural damage to dwellings, namely cripple walls and sill anchorage. Field observations from past earthquakes have shown that inadequate lateral bracing of cripple walls and insufficient sill anchorage lead to moderate to major structural damage in single-family wood-frame dwellings. Cripple walls are short wood stud walls that enclose the crawl space under the first floor of a dwelling. The cripple walls combined with the sill anchorage are responsible for transferring the gravity load of the dwelling as well as the lateral loads induced by earthquakes to the foundation. It is commonly seen in pre-1970 homes that cripple walls are not structurally robust enough to meet these demands. This is primarily due to poorly constructed lateral load transfer detailing within the cripple walls. In modern construction, cripple walls are sheathed with wood structural panels underlying the finish material, but in older dwellings, the use of wood structural panels was not required by building codes. In fact, many pre-1970s cripple walls are braced with their exterior and/or interior finish materials, such as stucco or horizontal wood siding boards, giving them little resistance to transfer the vertical and lateral forces from the floor(s) above to the foundation. In the event of an earthquake, failure of either the cripple wall or a weak link along the floor to foundation load path can cause the dwelling to collapse, as shown in Fig. 1.

Fig. 1 – Collapsed horizontal siding cripple wall following 1994 Northridge earthquake [2]

To address the known issues with this vintage construction, in 2013, the Federal Emergency Management Agency (FEMA), the Applied Technology Council (ATC), and the CEA jointly funded a projected to develop a prestandard retrofit design for single-family wood light-frame dwellings. Their findings were published in 2018 as *FEMA P-1100: Vulnerability-Based Seismic Assessment and Retrofit of One- and Two-Family Dwellings* [3]. This prescriptive retrofit design consists of adding wood structural panels to cripple walls, anchor bolts to sill plate–foundation interfaces, and shear clips to cripple wall–first floor interfaces to properly brace cripple walls and transfer lateral loads to the foundation. Despite the availability of this prestandard, an experimental basis for the new detailing prescriptions was limited.

To this end, as part of the PEER-CEA project, an experimental program was executed at the University of California, Berkeley (UCB) and the University of California, San Diego (UCSD). UCSD's program consisted of testing only cripple-wall assemblies denoted as the *small-component testing program* [4], [5], [6], [7], and UCB's program consisted of testing cripple wall and first floor assemblies, denoted as the *large-component testing program* [8]. At UCSD, a total of 28 cripple wall-only assembly tests were performed. These tests focused on assessing the performance in variations of cripple wall finish materials, retrofit condition, cripple wall height, boundary conditions, anchorage condition, and vertical loading. Within this suite of information obtained, it was found that horizontal wood siding over framing was by far the weakest finish material tested. There were two unretrofitted specimens tested with horizontal siding exterior finishes differing in their height; for the 2-ft-tall specimen, the lateral strength was close to 30% of the lateral strength of the next weakest 2-ft-tall specimen, and for the 6-ft-tall specimen, the lateral strength was around 15% of the strength of the next weakest 6-ft-tall specimen [5]. It should be noted that specimens tested consisted of only framing members and wood siding boards fastened to them which is not always the case in many dwellings consisting of horizontal wood siding finishes. Many of these cripple walls also are constructed with diagonal straps, let-in bracing, and variations in nail spacing which affect the seismic performance of them.

Given the prevalence and vulnerability of cripple walls, as well as the available retrofit prescriptions and the availability of the aforementioned PEER-CEA experimental dataset, herein the focus is on developing a robust numerical model for future response prediction. Specifically, the focus of this paper is to 1) provide a best-practice strategy to calibrate connection models for fasteners and contact elements and 2) validate experimental results obtained from the PEER-CEA project of both existing and retrofit cripple walls with horizontal siding finishes using nonlinear numerical models. The program used to create the models is a MATLAB toolbox called *MCASHEW2* which is a two-dimensional modeling program developed to model wood shearwalls [9]. Numerical models are built to match the same detailing as the physical specimens tested. For validation of the experimental results with numerical models, attention is given to the lateral strength, drift capacity, stiffness, and hysteretic energy dissipation.

2. MCASHEW2

MCASHEW2 is a two-dimensional, lumped-parameter modeling program. It is a subsequent development of the computer program called *Cyclic Analysis of Shear Walls (CASHEW)* which was developed during the Consortium of Universities for Research in Earthquake Engineering (CUREE)-Caltech Woodframe Research Project for analyzing the response of light-frame wood shearwalls under reversed cyclic loading [10]. Within typical light-frame wood shearwalls, there are dowel-type connectors (e.g. nails), framing members, and sheathing panels, which make up the entire assembly, as shown in an image of a typical shearwall (Fig. 2). All contact and fastener connections are modeled with SDOF springs. The load-slip characteristics of these connections are assigned within the program. The program allows for boundary conditions as well as gravity loading to be assigned. The modeled response is captured through the iterative displacement-control method developed by McGuire et al. [11] for solving nonlinear equations of equilibrium for cyclic loading.

The loading protocol imposed on the models directly matched the loading protocol implemented during testing of the cripple walls. However, in the numerical analysis, to assure robust convergence, the displacement increment used was 0.005 inches for 2-ft-tall cripple walls and 0.015 inches for 6-ft-tall cripple walls. The load step was assigned to be sufficiently small enough to capture the nonlinearity of the first displacement cycles. The maximum load ratio was set at 500 which is a recommended value for the displacement-controlled convergence method used. In order to complete the *i*th displacement step, a residual tolerance of 1e-05 inches was used. Due to the highly nonlinear response of shearwalls, a maximum number of iterations is prescribed if the residual tolerance of the equilibrium equation is not satisfied. The maximum iterations was set as 15. Both this and the residual tolerance were assigned to be in accordance with recommended values given for successful shearwalls modeled with *MCASHEW2*.

3. Connection Hysteretic Spring Models

Within *MCAHSEW2*, eight types of elastic and hysteretic spring models are available to assign properties to the frame-to-frame, panel-to-frame, and panel-to-panel connectors. For each connection, there are two orthogonal

translational springs and one rotational spring to assign parameters to. Of the eight types of spring models available, five were used to build the numerical models. They are as follows: 1) linear elastic spring, 2) bilinear elastic spring, 3) Modified Stewart (MSTEW) hysteretic spring, 4) hold-down (HD1) spring, and 5) Nail-withdrawal (HD2) spring. The linear and bilinear spring models are elastic, while all other spring models are nonlinear inelastic. The linear spring model is a one parameter model assigned an initial stiffness.

3.1 Modified Stewart (MSTEW) Hysteretic Spring Model

The modified Stewart was developed during the CUREE program by Folz and Filiatrault [10]. It is based on the hysteretic model developed by Stewart [12] which was developed to capture the seismic response of shearwalls. The usefulness of this model for woodframe and earthquake research has been demonstrated by multiple projects, which have used the MSTEW model to accurately depict the nonlinear response of both sheathing nails and shearwalls. The model is able to capture strength hardening, strength deterioration, stiffness degradation, and pinching, which are all phenomenon exhibited by wood fastener connections. There are ten parameters required to define the model, as shown in Fig. 2.

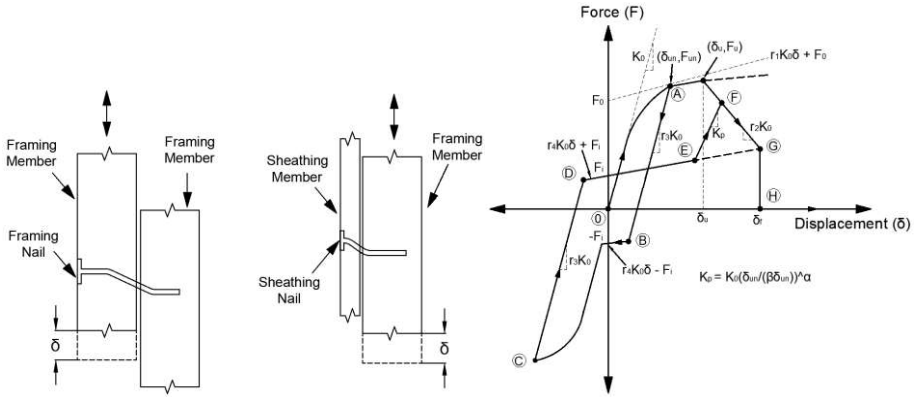


Fig. 2 – (a) schematic of force-deformation induced upon member-member shearing; and (b) Modified Stewart hysteretic model used to capture connection cyclic response [12]

4. Spring Model Parameter Determination

There are four MSTEW hysteretic spring models used for the horizontal siding models: 1) siding-to-framing perpendicular loading, 2) siding-to-framing parallel loading, 3) sheathing-to-framing perpendicular loading, and 4) sheathing-to-framing parallel loading. For the existing specimens, there are no sheathing-to-framing fasteners used due to there being no plywood attached to the interior of the cripple wall. The MSTEW parameters used for the siding-to-framing fastener connections were derived from the available database of MSTEW fastener connections. Although there is a significant body of knowledge on sheathing-to-framing fastener connection models, the data is limited in its application for use in creating horizontal siding finished cripple wall numerical models. Comparisons between results of the fastener testing programs provided a basis for assigning the MSTEW parameters used for the shear connections between sheathing/siding and framing. The parameters for the fasteners in this study were derived from the tests that most closely represented the fastener connection configurations that the cripple walls were constructed and tested with. The MSTEW parameters are influenced by the specific gravity of the sheathing and framing materials as well as the length and diameter of the fastener used. Since there is no database on the exact configurations used in this experimental program, estimations for the parameters assignments took into account the trends in MSTEW parameter variability from the available databases, specifically sheathing thickness and nail diameter.

In the instance of the horizontal siding cripple walls, fastener-connection databases from Coyne and Christovasilis et al. were used [13], [14]. Both of the test programs had the same testing configuration and loading protocol, thus allowing for comparable data between the two programs. For the horizontal siding to framing

connections, the thickness of the redwood siding was 3/4-in. and fastened with 8d common nails into 2x4 Douglas Fir framing. Trends between sheathing thickness/nail diameter and their effects on the resistance force parameter of backbone (F_0), the pinching residual resistance force (F_i), the initial stiffness (K_0), and the drift corresponding to the maximum restoring force (δ_u) were modeled. These were the parameters chosen to be analyzed due to their effects on the yield displacement, yield strength, ultimate displacement, ultimate strength, residual displacement, and residual strength of the fastener hysteresis. Using a linear regression to fit values of F_0, F_i, K_0, and δ_u based on the sheathing thickness and then the nail diameter produced values for these parameters, which would be expected for the fastener connections used in the cripple walls. The parameters from the linear regression were chosen based on the model with the highest r-squared value. This would ensure the most confidence in assigning the MSTEW parameter. The models with the highest r-squared value varied for each parameter in question. In some cases, the model was based on the sheathing thickness, and in other cases it was based on the nail diameter. There was also variation based on whether the loading implemented on the fasteners was parallel to grain or perpendicular to grain. An r-squared value closer to 1.0 demonstrates that there is a strong correlation between the MSTEW parameter in question and either the sheathing thickness or nail diameter which gives confidence in modifying the MSTEW parameter based on the linear regression. It was chosen to not modify the parameters r_1, r_2, r_3, r_4, α, and β with this methodology. For these parameters, there was little variation in their values throughout both of the testing programs. Due to the small amount of variation, an average was taken of these parameters for all of the tests performed in the two testing programs.

To account for the differences in strength of the connections with regard to wood type and specific gravity of the wood, the National Design Specification (NDS) for Wood Construction yield equations were used [15]. These adjustments to the strength of the fasteners were based on Section 12.3 – Reference Lateral Design Values. The yield limit equations prescribed in this section are based on the yield mode of the fastener connection, type of fastener, and type of lumber used for the framing and sheathing members. The parameters used to modify the contribution of the lumber materials when finding the reference lateral design values, denoted as Z, are the specific gravity of the lumber, thickness of sheathing material, nail length, and nail diameter. The first step is to determine what the Z-value is for the configuration tested in each fastener connection experimental program. Next, the Z-value is determined for the configuration tested in the experimental program at UCSD. The ratio of these two Z values is used to scale the resistance force parameter of backbone (F_0) MSTEW parameter. This parameter, along with the initial stiffness (K_0), the ratio of stiffness parameter of the ascending backbone to K_0 (r_1), and the drift corresponding to the maximum restoring force (δ_u) dictate the yield strength and ultimate strength of the fastener connection. It was chosen to only modify F_0 and preserve the rest of the remaining parameters in line with data from the fastener connection tests. This would increase the yield strength and ultimate strength of the fastener connection without modifying the yield displacement and ultimate displacement. The initial stiffness and drift corresponding to the maximum restoring force would be affected by the specific gravity of the lumbers used, but this methodology for adjusting the parameters was done to modify the strength of the fasteners, so only F_0 was changed by the ratio of Z-values. The modifications for the initial stiffness and drift corresponding to maximum restoring force were made from the linear regression method previously described. The values obtained for these parameters were compared with the available data from fastener testing programs which had similar specific gravities of the framing and sheathing members. They were reasonable enough that further changes based the Z-value was deemed unnecessary. Table 1 shows the assigned MSTEW parameters for both parallel and perpendicular-to-grain fasteners used for the horizontal siding finished cripple wall models.

Table 1 – MSTEW parameters used for siding-to-framing fasteners

MSTEW Parameters	K_0 (kip/in.)	r_1	r_2	r_3	r_4	F_0 (lbf)	F_i (lbf)	δ_u (in.)	α	β
Parallel-to-grain	10.647	0.051	-0.047	1.1	0.015	240.1	33.84	0.3600	0.72	1.06
Perpendicular-to-grain	7.534	0.049	-0.049	1.4	0.015	216.3	28.17	0.3894	0.72	1.06

For the retrofitted cripple walls, three 15/32-in plywood panels were attached to the interior of the framing with 8d common nails at 3-in. on center spacing around the edges and 12-in. on center spacing through the field. These fasteners were modeled with MSTEW hysteretic spring models. The parameters for the MSTEW models were chosen using data from Christovasilis et al. [14] on 8d common nails fastening 7/16-in. OSB to Hem Fir framing. This data was chosen because it most closely aligned with the tested configuration as well as having the least amount of variance in the data set. Modifications to the strength were done based on the comparison of the Z-values of the tested set up from Christovasilis et al. versus the configuration tested with the retrofitted cripple walls finished with horizontal siding below. Table 2 shows the assigned MSTEW parameters for both parallel and perpendicular-to-grain fasteners used for the plywood-to-framing connections in the retrofitted cripple wall models.

Table 2 – MSTEW parameters used for plywood-to-framing fasteners

MSTEW Parameters	K_0 (kip/in.)	r_1	r_2	r_3	r_4	F_0 (lbf)	F_i (lbf)	δ_u (in.)	α	β
Parallel-to-grain	8.4113	0.025	-0.27	1.028	0.005	242.4	35	0.50	0.6	1.06
Perpendicular-to-grain	6.0638	0.026	-0.028	1.021	0.01	238.1	34.2	0.559	0.55	1.06

4.1 Panel-to-panel Contact Elements

Since the siding boards are shiplapped, panel-to-panel contact elements need to be assigned to model the frictional forces that develop between the overlaps of the siding boards. An MSTEW hysteretic spring model was used to capture the frictional forces developed between the boards. This model was chosen because it allowed for the frictional forces to decrease as the displacement amplitudes of each cycle increased. When the imposed displacement increases, the nails fastening the siding to the framing would begin to pull out. When the nails pull out, there is less normal force imposed by the nailed connection to the siding boards which would, in turn, causes a reduction in the frictional force between the boards. It was decided to use a constant magnitude of frictional force until the nails reached yield displacement. The displacement at maximum restoring force parameter (δ_u) was taken as the average of the parallel and perpendicular-to-grain fastener yield displacement. In order to determine the magnitude of frictional force developed between the siding boards, an analysis was done based on the Coulomb friction model. It was assumed that the coefficient of friction between the siding boards was 0.2 which is in line with the kinetic coefficient of friction based on work done by Deta et al. [16]. The principal of energy conservation was used to find the frictional force developed. The energy of the nail upon release of the nail gun was set equal to the energy of the nail fully penetrating the siding board and the work done by the nail penetrating siding boards. It was determined that the frictional force developed between boards at each stud connection was 0.04 kips. The MSTEW model for the siding board contacts was created so that the friction is instantly developed between the siding boards which meant that the initial stiffness parameter was set sufficiently large to enforce this. The other parameters were assigned so that the friction stays constant until the nails yield and then begins to decrease as the displacement amplitude between boards increases. The MSTEW parameters for the panel-to-panel contact elements are shown in Table 3.

Table 3: MSTEW parameters used for panel-to-panel contact elements

MSTEW Parameters	K_0 (kip/in.)	r_1	r_2	r_3	r_4	F_0 (lbf)	F_i (lbf)	δ_u (in.)	α	β
Panel-to-panel contact	50	0.0001	-0.002	0.2	0.0002	40	1	0.24	1	1

5. Model Description

In total, four cripple walls were tested with horizontal siding exterior finishes [5]. These cripple walls were 2-ft-tall and 6-ft-tall, with one each emulating existing and retrofitted conditions. All cripple walls were 12 ft– 4½ in. length and constructed with the same framing details beside the length of the studs, which increased from 19½ in. to 67½ in. from the 2-ft-tall specimens to the 6-ft-tall specimens. All framing members were construction grade Douglas Fir. The horizontal siding boards were shiplapped, 1x6 nominal (3/4 in. x 5½ in.), construction grade redwood. 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. The horizontal siding was fastened with 2–8d nails per stud. Only the outermost end stud was nailed to the ends. Nails were spaced 3 in. apart on siding boards, centered about the middle of the board.

The cripple walls were retrofitted in accordance with the FEMA P-1100 prescriptive retrofit guidelines [3]. Both the 2-ft-tall and 6-ft-tall specimens were fully sheathed along the interior with 15/32-in. thick Grade 32/16 plywood and was placed in three 4-ft sections. 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. The plywood panels terminated at the top of the middle top plate. To accommodate the retrofit, additional 4x4 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 4x4 studs were toe-nailed in with 2–8d common nails top and bottom. The addition of studs and blocking plates were used to allow the plywood panels to be nailed to the cripple wall. The blocking was fastened to the sill plate between studs with 6–10d nails staggered per block. For the additional anchorage required by FEMA P-1100, there were small variations in the details of the retrofit design. The 2-ft-tall specimen added two additional ½-in. anchor bolts to create an anchor bolt spacing of 32-in. on center. For the 6-ft-tall specimen, four additional anchor bolts were added. The layout for the anchor bolts was the same as with the 2-ft-tall specimen with two additional anchor bolts positioned 12 in. interior to the outermost anchor bolts. In addition to the added anchor bolts, hold-downs were used for the tie-downs at both ends. The models created to replicate the same exact layout as the tested cripple walls to allow for the models to be able to validate the experimental results. The gravity load imposed on the models was 450 plf. All models were fully constrained at both ends of the sill plate. Fig. 3 shows the existing, 2-ft-tall specimen and a retrofitted, 2-ft-tall specimen.

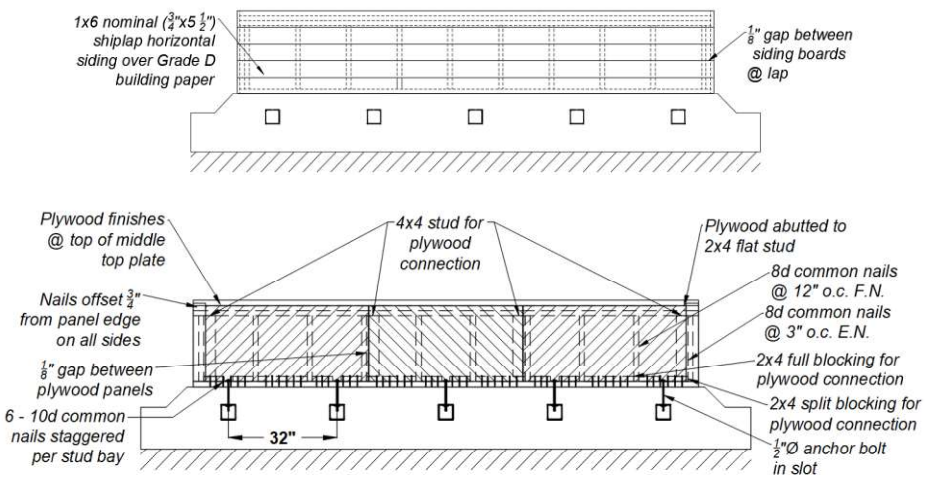


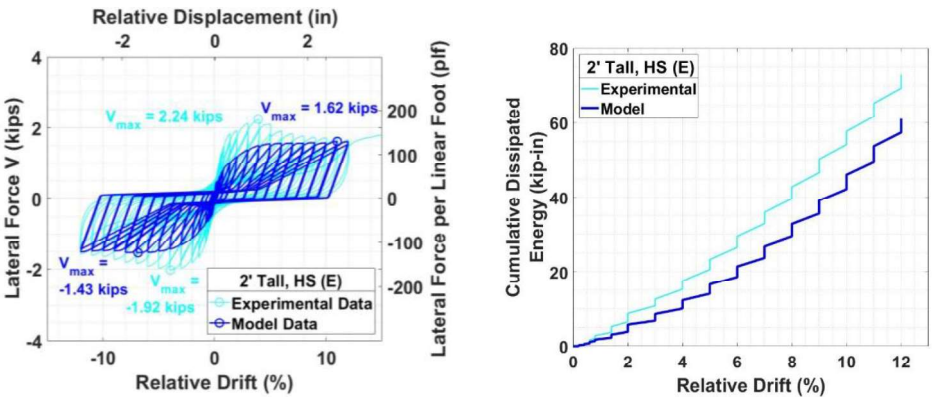
Fig. 3 – Elevation views of 2-ft-tall cripple walls with horizontal siding; **existing** condition (top) and **retrofitted** condition (bottom) [5]

6. Numerical Model Results and Discussion

The following section presents the results of the numerical models previously described with attention towards the hysteretic response and the hysteretic energy dissipation. In addition, the lateral strength, stiffness, and drift capacity for the numerical models and the tested cripple walls are compared. As previously stated, each numerical

model was built to match the construction details of the physical cripple walls. The exception to this is that the models did not have corners that wrapped around as *MCASHEW2* is a two-dimensional modeling program. The flat stud at the corner used to fastener the short sections of siding to the wrap around was also not included in the model because it is outside of the program's capabilities. Lastly, trim boards that are commonly used to cover the corners joints were not included in the model. While each of these details contribute to the overall strength, the contribution is small enough to develop models without them. Discrepancies between the numerical model and physical test due to the lack of these details in the model construction are discussed.

In Fig. 4(a), the hysteresis of the 2-ft-tall, existing horizontal siding numerical model is overlaid on the response results from the tested cripple wall (Specimen A-7). In addition, Fig. 4(b) shows the cumulative energy dissipation for both the test and the model. From the overlay of the hysteresis, it can be seen that the model underpredicts the peak lateral strength by 28% in the push direction and 26% in the pull direction. However, by the end of the 12% drift cycles, the lateral strength of the model and the test are nearly identical. The reduction of strength in the model compared with the test could be attributed to multiple reasons. The primary reason that the peak strength is smaller is due to the lack of built-up corners constructed in the model. When the cripple wall was tested, the lowest siding board and the trim board on the corner would make contact with the top of the foundation. Throughout the test, the corner trim pieces at both ends developed cracks propagating from the top of the board to the bottom due to the internal stresses developed from the bearing on the foundation. In addition, cracks developed in some of the corner siding boards. This bearing likely caused a significant increase in strength that would not be captured in the model. As the displacement amplitudes increases, the strength of the model and the tested cripple wall eventually converge as the elements bearing at the corner had either cracked or their fasteners yielded. In Fig. 4(b), it can be seen that the numerical model dissipated more energy than the tested cripple wall. The two hysteresis are comparable up to 0.6% relative drift, and then they begin to diverge from one another. As a note, relative drift refers to the drift of the cripple wall only, excluding displacement between the sill plate and foundation. By 12% relative drift, the model had dissipated 20% more energy than the experiment with a cumulative energy dissipation of 73 kip-in compared to 61 kip-in. This was the only model had more hysteretic energy dissipated than the experiment. Typically, the MSTEW hysteretic spring model tends to overestimate the energy dissipation after reaching peak strength due to the strength deterioration and stiffness degradation being more significant at these displacement amplitudes.

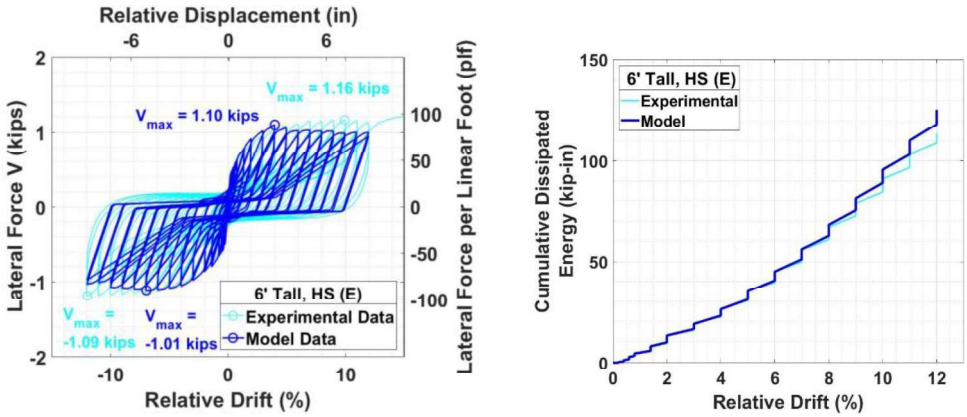


(a) (b)

Fig. 4 – 2-ft-tall, **existing** horizontal siding finished cripple wall; (a) hysteretic response; and (b) cumulative energy dissipation

The next comparison between the numerical model and the physical test is for the 6-ft-tall, existing cripple wall finished with horizontal siding (Specimen A-13). Fig. 5(a) show the overlay of the hysteretic response of the experimental data and the model data. There is a much more similar response between the two compared with the 2-ft-tall, existing cripple wall. The peak lateral strength and corresponding relative drift amplitude of the model is 1.10 kips at 4% relative drift in the push loading direction and -1.01 kips at 7% relative drift in the pull loading direction. For the tested cripple wall, these values are 1.16 kips at 10% relative drift and -1.09 kips at 12% relative drift. Therefore, in both directions of loading, the difference in lateral strength between the model and the experiment are less than 8%. At the corners of the tested cripple wall, there was significantly less bearing forces

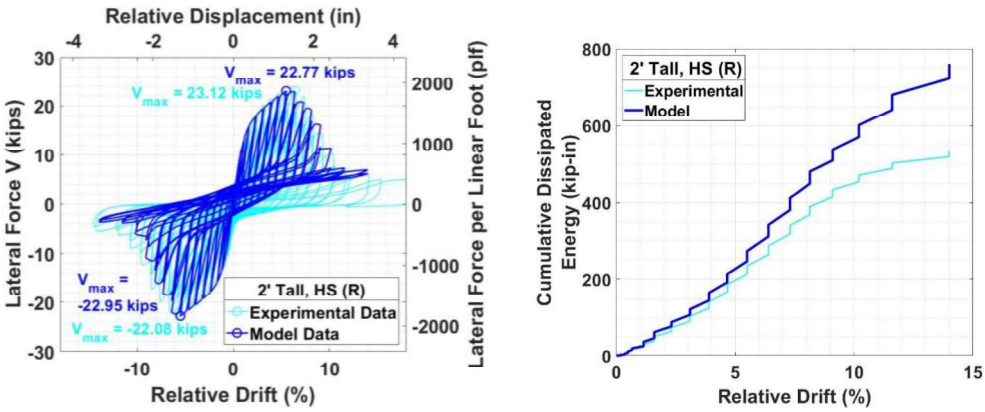
developed between the bottom siding boards/corner trim boards and the foundation due to the corner trim boards and bottom siding boards being further distanced from the foundation. The numerical model slightly underpredicted the strength of the tested specimen due to the model not having the corner components incorporated in the model, therefore, it did not capture this bearing. In Fig. 5(b), an overlay of the cumulative hysteretic energy dissipation is given for the model and the experiment. The amount of energy dissipated between the two is nearly identical up to 7% relative drift. At this point, the model overpredicts the energy dissipation for the same reasons as the previous model. By the end of the loading protocol, the model had dissipated around 11% more energy than the tested cripple wall, 126 kip-in versus 113 kip-in. Overall, the model provides a robust comparison with the hysteretic response of the experiment.



(a) (b)

Fig. 5 – 6-ft-tall, **existing** horizontal siding finished cripple wall; (a) hysteretic response; and (b) cumulative energy dissipation

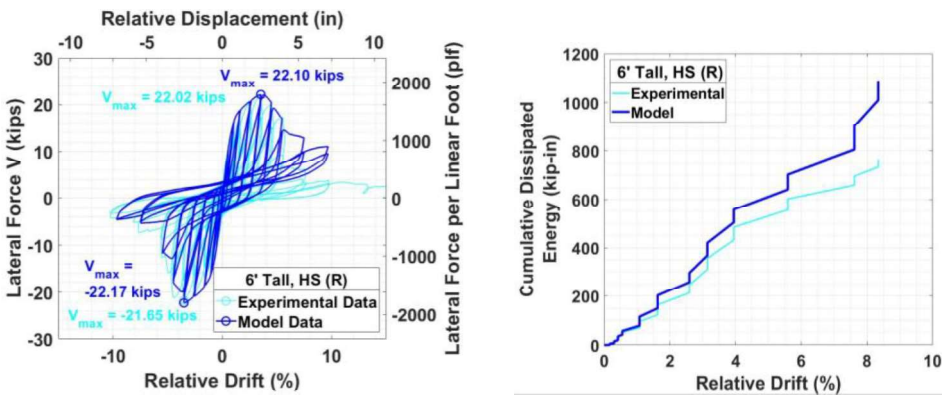
Once the numerical models for the existing specimens had been built, they were modified to add the FEMA P-1100 retrofit. Fig. 6(a) shows an overlay of the hysteretic response of the model and the tested specimen for the 2-ft-tall, retrofitted cripple wall finished with horizontal siding. The tested cripple wall, Specimen A-8, had a peak lateral strength of 23.12 kips at 6.4% relative drift in the push loading direction and 22.95 kips at 5.5% relative drift in the pull loading direction. For the model, these values are 22.77 kips at 5.5% relative drift in the push loading direction and 22.95 kips at 5.5% relative drift in the pull loading direction. Both the peak lateral strengths and their associated drifts were similar between the model and the experiment. The model overpredicted peak lateral strength by 2% in the push direction and underpredicted strength by 4% in the pull direction. While the relative drifts at peak loading did not occur at the same drift amplitude for the push loading direction, the model results and test results were within 2% of strength for both the 5.5% and 6.4% relative drift cycles. From the start of loading protocol till lateral strength is achieved, the model captures the response of the test within 4% at each displacement cycle. Because of this, the secant stiffness associated with drift at 80% of the lateral strength is within 2% for the model and the experiment. Once strength is achieved, the model and the experiment experience significant strength degradation at each subsequent drift amplitude. The response of the model does not capture the test data as well post-peak strength, but it is comparable and shows that the loss of strength is due to the failure of fasteners attaching the plywood to the framing. Once peak strength occurs, the model hysteresis tends to pinch much more than experiment hysteresis. Because of this, the shape of both hysteresis do not align as well as they do for pre-peak strength. Still, the model does a good job of capturing the post-peak response in terms of expected lateral strength. In Fig. 6(b), a comparison of the cumulative hysteretic energy dissipation is overlaid for the numerical model and the tested specimen. The model slightly overestimates the energy dissipation up till peak lateral strength is achieved, and then it begins to diverge more significantly. Since there is an increased amount of MSTEW hysteretic springs used in the retrofit model versus the existing model, it would be expected that the typical overestimation of energy dissipation that occurs at larger displacement amplitudes would be more pronounced than with the existing model. At peak lateral strength, the hysteretic energy dissipation of the model is around 15% more than that of the experiment, but by the end of the test, the difference in energy dissipation is close to 40%.



(a) (b)

Fig. 6 – 2-ft-tall, **retrofitted** horizontal siding finished cripple wall: (a) hysteretic response; and (b) cumulative energy dissipation

The last model built for horizontal siding finished cripple wall was the 6-ft-tall, retrofitted cripple wall (Specimen A-14). In Fig. 7(a), an overlay of the numerical and the experimental hysteresis is provided. The peak lateral strength and associated relative drift for the model and the experiment are nearly identical in the push loading direction and close to the same for the pull loading direction. The peak strength in the pull loading direction is 22.10 kips for the model and 22.02 kips for the experiment with both occurring at 3.7% relative drift. In the opposite loading direction, the model slightly overpredicts the strength with a lateral strength of 22.17 kips versus 21.65 kips. This accounts for a 2% difference in strength. Both the model and the test reached strength at 3.7% relative drift in the pull loading direction. From the start of the loading protocol until around 1% relative drift, the model and the tests are in alignment, then the model overpredicts the strength up until peak strength. Because of this, the secant stiffness associate with the relative drift at 80% peak lateral strength is 5% larger for the model than the test. After strength is achieved, the strength deterioration between the model and the test are similar for the following two displacement cycles. There are abrupt drops in strength for the test in the last two displacement cycles. The model does well to capture this in the pull loading direction but does not do well in the push loading direction. The strength of the model and test are nearly identical for the last displacement cycle in pull loading while the strength of the model is around 2.5 times larger for the last displacement cycle in push loading. In Fig. 7(b), the cumulative hysteretic energy dissipation of the model data is overlaid with that of the experimental data. Similarly to the 2-ft-tall, retrofitted cripple wall data, the amount of hysteretic energy dissipated is comparable up to peak strength, and then the model begins to significantly overestimate the energy dissipation. At peak strength, the model had dissipated 17% more energy than the experiment. By the end of the loading protocol, the model had dissipated 42% more energy than the tested cripple wall.



(a) (b)

Fig. 7 – 6-ft-tall, retrofitted horizontal siding finished cripple wall; (a) hysteretic response; and (b) cumulative energy dissipation

7. Conclusions

The purpose of this study was to conduct numerical simulations of wood finished cripple walls, in an effort to define best practices for building connection-level models of horizontal siding finished cripple walls. Horizontal siding is used in older homes often as the sole means of providing lateral shear resistance as it is attached to vertical framing members. As a continuous horizontal sheathing member, it gains strength due to its attachment to the vertical framing members via the fasteners (nails) as well as via contact between the siding boards. In this work, the siding-to-framing fasteners were modeled with MSTEW hysteretic springs, which had properties that originated from an MSTEW model calibrated using a prior fastener connection testing program. The parameters of these models were modified to account for the type of fastener, types of lumber, and thickness of siding used in the testing program. Contact elements were implemented to capture the interface resistance that develops between siding boards. Fastener model parameters were chosen to replicate the friction generated between siding boards as the cripple wall displaces. The following observations regarding optimal selection criteria of the MSTEW model can be made:

- Using available databases for fastener testing provides a good basis for selecting MSTEW parameters due to the large number of connection tests performed with similar fastener type, lumber type, and lumber thickness.
- Modifications to the stiffness, strength, and displacement parameters can be made using linear regressions of available data to account for differences in fastener diameter, sheathing thickness, and specific gravity of the lumber used.
- Reference lateral design Z-values from AWC NDS are effective in modifying strength parameters to account for the types of lumber used.

The following conclusions can be made about the response of the numerical models compared with the results of the tested cripple walls:

- The hysteretic response of the numerical models generally provides a robust comparison with the measured hysteretic response of the test specimens.
- The largest discrepancy between a model and an experiment was for the 2-ft-tall, existing cripple wall. This discrepancy can be partially attributed to the increased strength of the test specimen due to bearing of corner elements on the foundation, a feature not explicitly accounted for in the numerical model.
- For the existing cripple walls, the model and experiment attained similar lateral strengths by the end of the loading protocol.
- For the retrofitted cripple walls, the model and experiment are in good agreement until the final few displacement cycles. This can be largely explained by the convergence issues realized in the numerical model, which likely artificially stiffens the model response predictions. Overall, the cumulative hysteretic energy dissipation is similar between the model and the experiment up until large displacements (>5% drift), where the model typically dissipates more energy due to the MSTEW model using static parameters which underestimate strength deterioration and stiffness degradation at large displacement amplitudes.

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