EXPERIMENTAL TESTING AND SEISMIC PERFORMANCE ASSESSMENT OF WOOD-FRAME SHEAR WALLS WITH COMMON CONSTRUCTION DEFECTS

FINAL PROJECT REPORT

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for

Federal Emergency Management Agency (FEMA)

CTP Program

FEMA

Region X

January 13, 2025

Executive Summary

The seismic performance of wood stud-wall building components in low-rise residential construction have been widely tested and modeled, and design limits are established and codified in the various building codes. However, the performance of improperly designed or poorly constructed houses has not been thoroughly studied. This study conducted both laboratory testing of code-compliant and non-compliant wall components and assessed seismic risk using computational models and nonlinear dynamic analysis. The study was motivated by the lack of building code enforcement within portions of the Municipality of Anchorage, Alaska coupled with the corresponding variation of damage that occurred in the 2018, a Mw 7.1 earthquake. In areas where codes are not enforced through plan review, inspection, and permitting, damage tended to be higher following shaking. There were no functioning ground motion sensors in the area, so intensity of shaking can only be estimated through surveys and on-the-ground reporting.

This report includes details of laboratory testing to evaluate the hysteretic capacity of 14 configurations of wood frame shear walls typical of construction in Alaska and seismic performance assessment conducted using nonlinear modeling and analysis software. In total, we conducted 37 wall tests, with variation in fastener type and spacing and panel type and configuration. Multiple tests of each variation were conducted to assess variability within each configuration. Testing illustrated the reduction of strength and stiffness associated with non-code-compliant shear walls, when compared to those built to code standards. We measured the deformation capacity of each wall configuration, and found it to correlate strongly with the peak strength of each wall for a particular type of fastener. For more brittle fasteners (e.g., staples), overall shear wall behavior was more brittle.

We used aggregated test data to calibrate nonlinear models of three archetype houses and six wall configurations representative of residential construction in Alaska. These models were subjected to Incremental Dynamic Analysis (IDA), in which ground motion records are applied to an undamaged building at increasing intensities until a collapse threshold is reached. The houses are one- and two-story structures designed to specified wall designs, with capacities based on design strengths of code-compliant walls. One house features a large number of windows, leading to tight fastener spacing, while others have less windows, leading to less highly loaded walls. In total, we ran 24 IDAs to determine the median collapse capacity of each configuration. We fit cumulative distribution functions to the range of intensities that cause collapse to evaluate the probability of collapse at a range of ground motion intensities.

The results indicate that structures built to or above design standards generally meet ASCE 7-22 target reliability standards, with probabilities of collapse at MCER level ground motions below 10%. Collapse risk was most strongly correlated with the strength of wall panels, and fastener spacing has the largest impact on strength. Houses with half the edge nails (spaced at double design spacing) are 3 to 5 times more likely to collapse during an MCER level event. Halving the number of nails again (12 inches on center for a design spacing of 3 inches on center) results an another three-fold increase in the probability of

collapse. T1-11 fastened improperly leads to a similar increase in collapse probability (3 to 5 times) when compared to correctly fastened walls due to the significant loss in wall strength. Wall configurations that provided more ductility (e.g., those with nails) had more gradual increases in collapse risk with increasing spectral accelerations than those with more brittle failure modes (e.g., those with staples).

These findings are consistent with the anecdotal evidence from post-earthquake inspection, particularly in Eagle River, AK. Houses with sparsely spaced nails, or single-row of nails along a T1-11 seam presented more displacement-based damage, such as drywall cracking and opening of exterior panel seams.

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1 Introduction

Modern wood frame construction typically performs well during earthquakes (FEMA P-2139-2 2020), but the behavior or older or non-engineered buildings is more variable, and has not been widely studied. More than half of economic losses sustained during the 1994 Northridge earthquake were attributed to damage and collapse of residential wood frame construction (Kircher et al. 1997). In the 1995 Great Hanshin (Kobe) earthquake, more than 95% of fatalities were a result of housing collapse or fire after the earthquake, with rates of damage higher in older houses, larger multi-family structures, and terraced housing (Hirayama 2000). In the United States, most residential structures are wood-frame and were built prior to the onset of modern seismic codes (Kirkham et al. 2014), with 75% of structures constructed prior to 1990, and half constructed before 1974. These structures may be vulnerable to earthquakes. In some regions of the country, including Alaska, building codes are variably enforced, with sometimes only a land use permit required for new construction. These structures may be susceptible to collapse in large earthquakes; Alaska has some of the highest seismic hazard in the country (Petersen et al. 2024).

On November 30th, 2018, a Mw 7.1 earthquake shook Southcentral Alaska, affecting the most populous region in the state and inflicting significant damage to transportation infrastructure and commercial and residential structures (Hassan et al. 2022; Cabas 2021). Ground acceleration data recorded from instrumented buildings indicate the shaking was 40 - 120% of Design Earthquake intensities for Anchorage, and far from shaking intensities of a risk-targeted maximum considered earthquake (MCE$_R$) event. Notably, inspection reports indicated that the damage to wood-frame buildings seems to have been more severe and more widespread in areas within the Municipality of Anchorage (MoA) where building codes are not actively enforced, particularly Eagle River. The subset of the MoA where building codes are enforced through permitting, plan review, and inspection is called the Building Safety Service Area (BSSA), and is often referred to as the Anchorage Bowl. Fig. 1 illustrates the boundary of the BSSA and the number of residences that requested assistance in the aftermath of the 2018 earthquake, illustrating the significant discrepancy in damage in and outside of the BSSA.

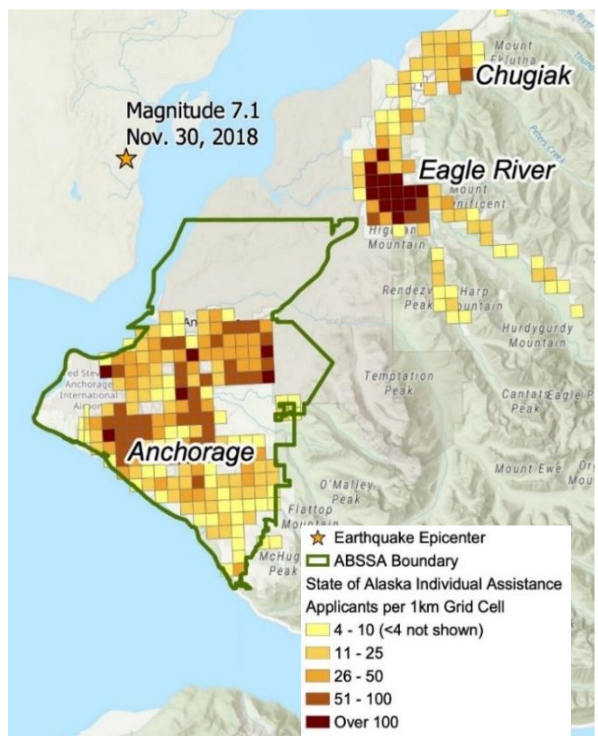


Fig. 1. Boundary of the BSSA and requests for repair assistance following the November 2018 earthquake (Askov et al. 2019).

1.1 Earthquake Intensity

The measured shaking intensity is illustrated in Fig. 2, quantified on the in terms of the spectral accelerations at a period of 0.2 seconds, which coincides with the short natural response periods of low-rise residential structures. Across the region, there is wide variety in the recorded values, with most stations recording peak values between 0.5 and 0.9 g, but some stations measuring short-period spectral accelerations values upwards of 1g. The variations of the short-period measured responses can be seen in Fig. 3. Ground motion data and spectra values are from the Center for Engineering Strong Motion Data (CESMD) database, with spectra values shown for a 5% damped system. There were two stations with short period spectral accelerations above 3 g, and these were not included in the calculation of the standard deviation, as it is suspected that there were measurement errors. There was no damage indicative of such high accelerations. Despite the relatively large magnitude (Mw 7.1) of the earthquake, the recorded and estimated accelerations are generally at or below design level.. This is primarily a result of the hypocentral depth—the fault ruptured at a depth greater than 30 miles (50 km)—which increased the distance between the epicenter and more populated urban areas (West et al. 2020).

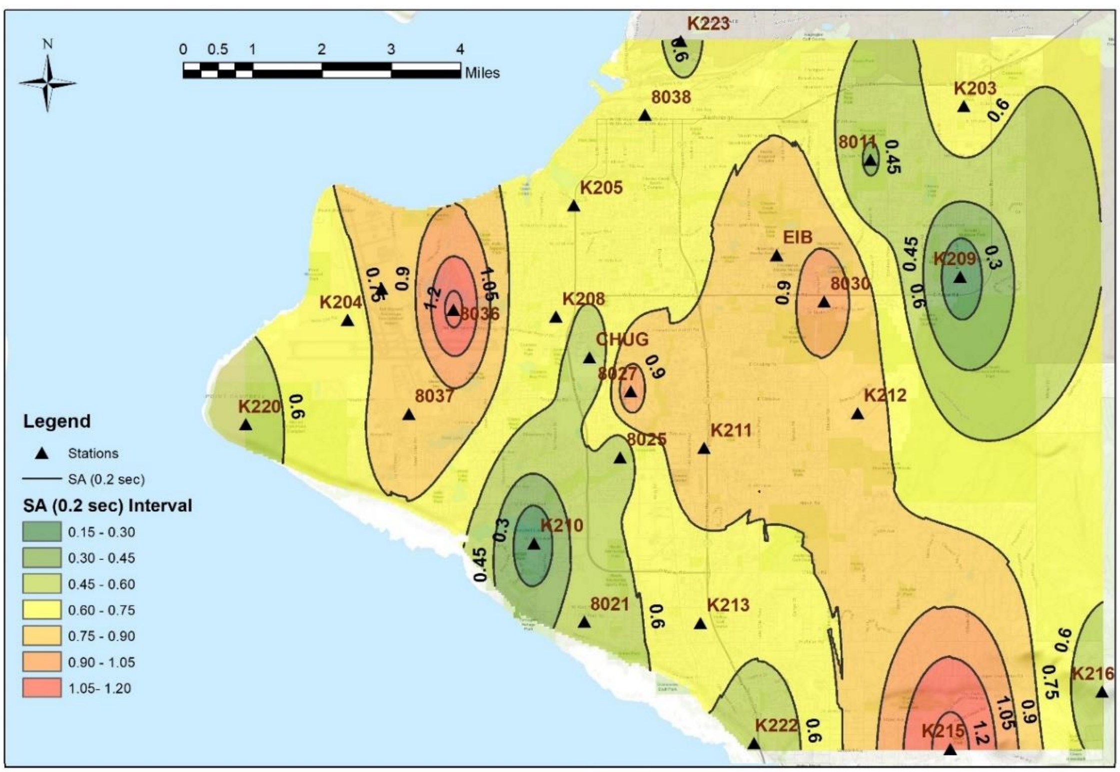


Fig. 2. Spatial variation of spectral responses at T= 0.2 sec (Dutta et al. 2019).

Despite comparable estimated shaking within the Anchorage Bowl and Eagle River, post-earthquake inspection and requests for assistance reveal a discrepancy in the extent of damage in these two urban areas. A lack of functional instrumentation outside of the BSSA means the exact intensity of shaking is unknown in Eagle River. Results from the United

States Geological Survey (USGS) "Did You Feel It?" surveys indicate shaking in Eagle River was comparable to the intensity of shaking within the Anchorage Bowl, where instrumentation validates survey responses (USGS 2018). This indicates that the discrepancy in the extent of damage may be related to the quality of construction or other structural factors. Structural response in the Anchorage area is also highly susceptible to site amplification effects (Thornley et al. 2022) due to the subsurface and topographical conditions.

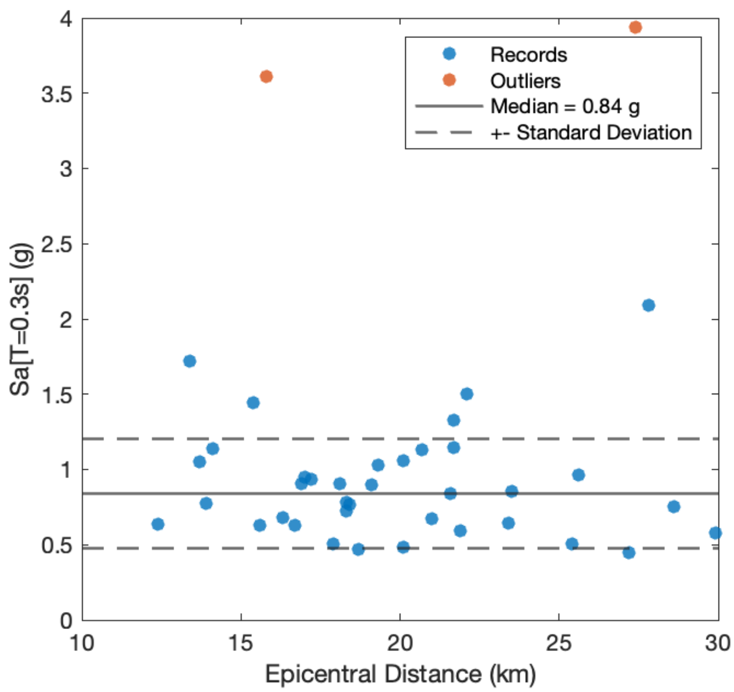


Fig. 3. Variation of ground motion record spectral accelerations (T = 0.3 sec) across the Anchorage Bowl.

1.2 Anchorage Building Stock

The population of Anchorage has grown significantly in the last century, with the largest increase occurring when the Trans-Alaska Pipeline was constructed and began producing oil between the mid-1960s and mid-1980s. This led to a temporary exponential increase in housing construction, and subsequent drop when the boom ended. Fig. 4 shows the number of housing units constructed since 1930, separated by population area within the MoA. Overlaid on this figure is cumulative distribution of construction year of housing in the MoA, indicating that approximately 70% of houses were built prior to the widespread enforcement of modern building codes in the early 1990s.

1.3 Damage to Wood-Frame Buildings

Damage to buildings within the BSSA during the 2018 earthquake was largely tied to ground failure and subsequent foundation settling (Jibson et al. 2020; West et al. 2020; Cabas et al.

2021). Poor performance of structural systems was observed in both commercial and residential structures built during the oil boom (1975 – 1985), as well as houses outside of the BSSA (Hassan et al. 2022). Damage to residential structures included slope failure and liquefaction, ground failure from poorly stabilized fill, collapse of chimneys and fireplaces, movement of houses off foundations, large cracks in walls, large residual drifts of building components, and collapse of entire structures (Hassan et al. 2022; Maison et al. 2021).

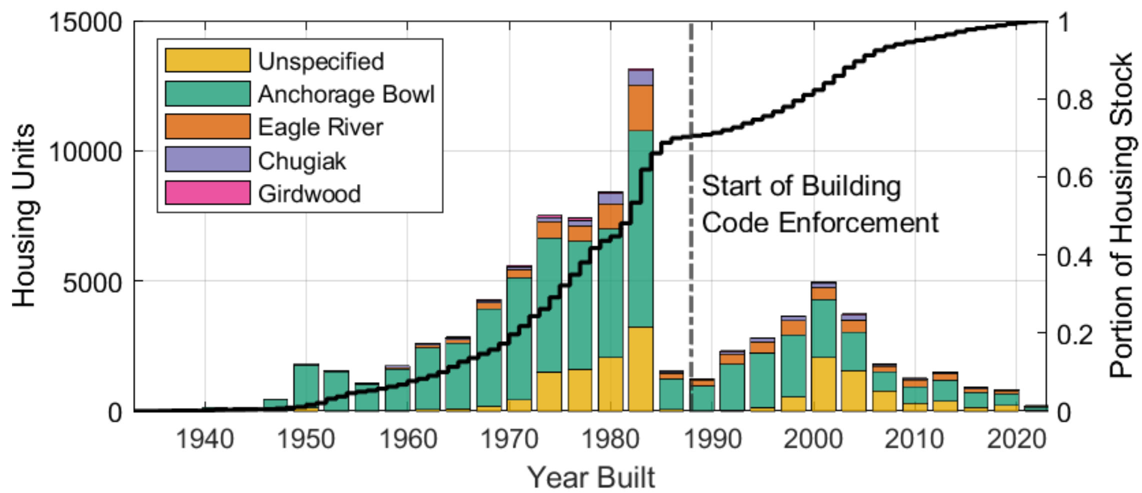


Fig. 4. Housing unit construction by year in regions of the MoA.

1.4 Research Methodology

The goal of this project was to quantify the effects of common structural deficiencies in seismic lateral systems in residential structures constructed in geographic areas without building code enforcement, determining the seismic risk posed by each deficiency, and then identifying steps to mitigating the risks.

Achieving this goal required three phases. The first phase required identifying what deficiencies exist and which are most common. Fortunately, the 2018 earthquake prompted a large number of post-earthquake building inspections throughout Anchorage and Eagle River, providing a source of knowledge about what defects exist, and how common they are. Clearly, this method is subjective, as it relies on the memory and records of the inspectors. And no information was collected about any structures whose owners do not want engineers or inspectors looking at damage in their homes. Many of the engineers that we interviewed mentioned that some owners they encountered did not want them on their property. This could be because they fear expensive repairs, insurance rate increases, or do not trust the government. However, the interview method is the best tool available without investing far more financial resources to conduct a more complete survey. During the immediate aftermath of the earthquake, a large number of reports were collected and cataloged by the Municipality of Anchorage about post-earthquake inspections. Unfortunately, this data was lost during a data transfer with insufficient backups (Wayne Bolen, personal communication, May 18, 2022). The second phase required determining the dynamic structural behavior of component-level elements (shear walls) both with and without deficiencies. Given the complexity of wood panel shear walls, which involve a variety of panel-types and thicknesses; fastener types, sizes, and spacings; hold-down types; and frame parameters, these data could not be obtained using models. Experimental tests were necessary. To obtain the required behaviors, thirty-seven walls were constructed and tested to failure.

The third phase utilizes numerical computer models to predict the behavior of entire three-dimensional structures using the dynamic behavior of the components tested in phase 2. Comparing this behavior to established seismic limits allows us to ascertain the probabilistic risk of collapse.

With all three phases completed, the seismic risk of hypothetical residential structures with and without each deficiency can be predicted and compared. This comparison allowed us to identify which deficiencies pose the greatest risk, and could then enable us to suggest for policy-makers the most cost-effective strategies for mitigating the risks.

1.5 Interviews

Prior to testing, we collected damage and inspection reports generated in the aftermath of the earthquake and conducted interviews with those who performed post-earthquake inspections or repairs. From these interviews, we identified common deficiencies in wood-frame homes built without code enforcement. The most common deficiencies include; lack of nailing in the mudsill, large nail spacing, small nail diameter, single row of nails at lapped edge of siding panels, missing or unattached hold-downs, and undersized staples.

2 Experimental Testing of Wood Shear Walls

Based on the findings of the interviews, the deficiencies reported were grouped into primary and secondary parameters. Primary parameters were those that were mentioned in multiple interviews and/or the interviewees indicated that these deficiencies were relatively common. Secondary parameters were those that seemed less common, but could potentially have dramatic effects on the structure's behavior.

Primary parameters were:

- Nail spacing
- Nail size (box vs common)
- Performance of nails vs staples
- Staple size
- Single-nailed seams in T1-11

Secondary Parameters

- No nails in mudsills
- No hold-downs

Given the difficulty in finding similar test results in literature, and in an attempt to reduce the number of specimens, a code-compliant baseline wall was chosen, and then configurations with deficiencies were selected to compare to the baseline configuration. The baseline configuration was an 8 ft x 8 ft wall constructed with two 7/16" thick 4 ft x 8 ft OSB panels oriented vertically and 8d common nails (0.131" Dia. spaced at 6" o.c. around panel edges with the same nails spaced at 12" o.c. in the field. Further details about the wall are provided below.

Testing protocol conforms with procedures detailed in ASTM E2126-11: Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings (ASTM 2011). The test is intended to capture the degradation behavior caused by cyclic loads typically induced by ground motions. Testing was conducted at the University of Alaska Anchorage's ConocoPhillips Structures Testing Lab (STL).

2.1 Test Frame and Data Acquisition

The test set-up and typical wall configurations are shown in Fig. 5. Reactions are resisted by a steel moment frame, which is rigidly affixed to a 3-foot thick concrete strong floor. Walls were pre-constructed horizontally outside the test space. The 2x6 bottom plate was pre-drilled to accommodate the location of anchor bolts protruding up from the test frame. Hold-downs were also pre-installed on the studs in line with anchor bolt locations. After the wall was constructed, the steel transfer beam was attached, which included lifting points. The wall was then lifted and set down vertically in place using an overhead crane. Once in place, the washers and nuts were installed on anchor rods and hold-downs and torqued to approximately 50 lb-ft. The bracing rollers were moved into place so that they were touching the steel beam, and the actuator was bolted to the steel beam.

Load was applied horizontally at the top of the panel using a MTS244.41 hydraulic actuator outfitted with a 110 kip load cell (MTS661.23E-01) load cell. A W10x15 steel beam transfers forces from the actuator to the wall assembly through a pinned connection, which allows rotation in the in-plane direction. Bracing rollers attached to the moment frame restricted out-of-plane movement of the beam and top of wall. The beam was affixed to the 2x6 top plate of the wall specimen with a minimum of twenty-two ½-in lag screws. At the base of the wall, 5/8-inch diameter anchor bolts connect the base plate to a steel HSS 6x6x3/8 section. The bolts were rigidly connected to the steel tube, simulating anchor rods embedded in concrete. The HSS section is bolted to the strong floor with 1-1/2 inch diameter anchor rods. Displacements were recorded using string potentiometers (pots), two 12-inch pots (Measurement Specialties SM2-12) were aligned vertically along the edge of the wall at the base of the frame to record the uplift of the chords, and one 24-inch pot (Measurement Specialties SM2-24) was aligned horizontally at the top of the frame and attached to the steel beam and an exterior wall. During the displacement-controlled test, forces were recorded by a 110-kip load cell (MTS661.23E-01) attached in-line with the actuator. The data were collected by a computer-controlled data-acquisition system and recorded at 20 Hz.

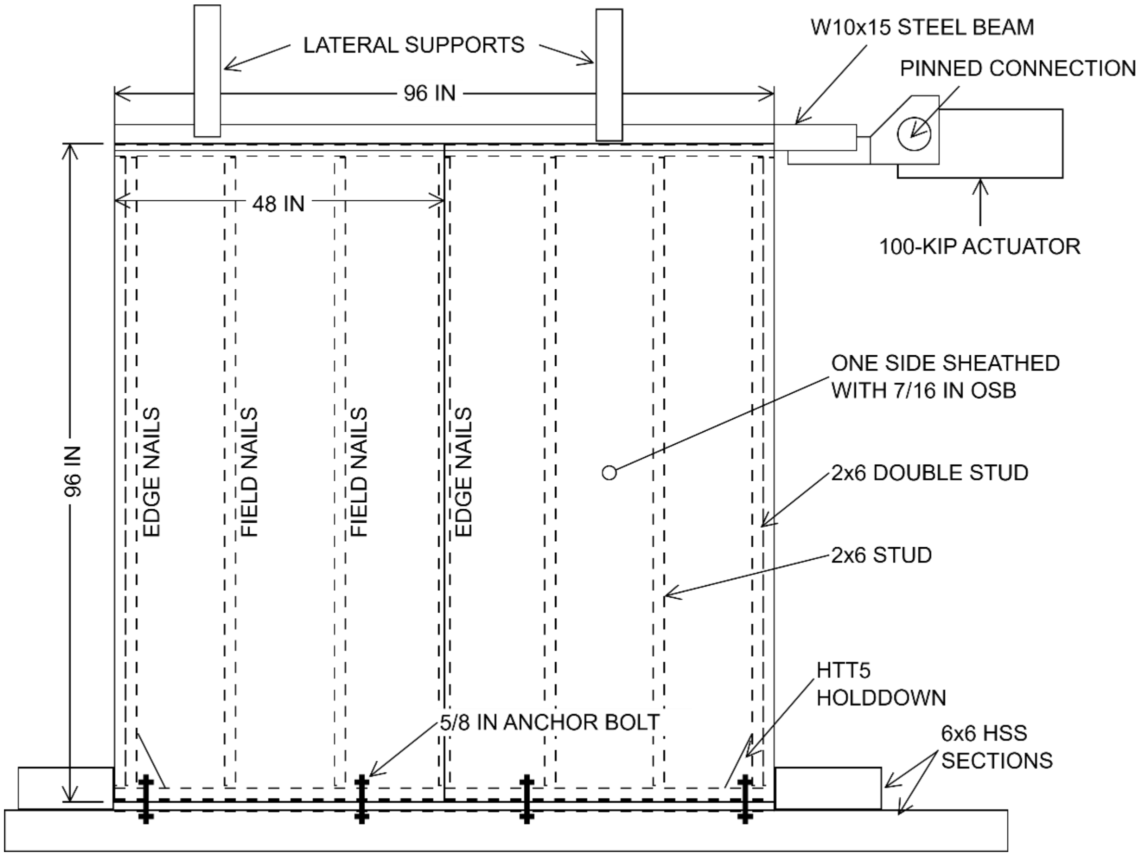


Fig. 5. Testing set up and typical wall configuration.

2.2 Testing Protocol

The CUREE protocol was developed to simulate cyclic demands induced by ground shaking (ASTM 2011). The displacement-based protocol is illustrated in Fig. 6, and involves a series of increasing displacements, with smaller, intermediate cycles between each "primary" cycle. The amplitude of displacement in this protocol is relative to a 100% drift, which was selected as 4.0 inches for this study.

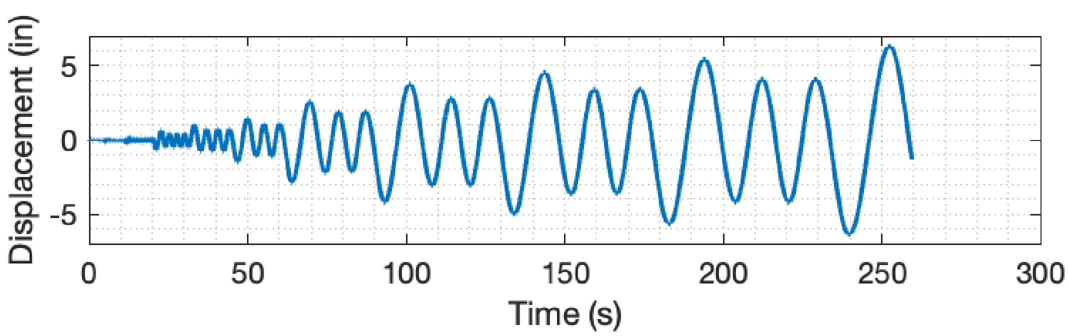


Fig. 6. CUREE protocol with a 100% displacement of 4 in.

The Test standard (ASTM 2011) allows for speed control in terms of frequency (in Hz) or a linear speed of the actuator. We used the latter in this study, choosing a speed of 1.50 inches/second. This resulted in a cycle period of 0.80 seconds during initial cycles, and up to 20 seconds at final cycles. For most tests, we applied displacements up to +/-6.40 inches (160% of the baseline drift), despite a loss of virtually all lateral resistance. Some tests created a damaged wall that posed a risk to personnel safety or equipment, and were stopped before the full displacement protocol could run.

2.3 Wall Configurations

The standard wall configuration is illustrated in Fig. 7a. Each test wall is 8 ft x 8 ft, with 2x6 studs spaced nominally at 16 in. on center. Bottom and top plates are attached to the studs with 3 end-nailed 16d common nails (3-1/2 x 0.162-in). Studs are either Douglas Fir or Hemlock Fir. Chords are double 2x6 studs connected together with two 16d common nails every 8 in. Sheathing is 7/16-in. thick Oriented Strand Board (OSB) or 7/16-in. thick T1-11 siding, which has properties similar to plywood. Siding fasteners are differentiated between edge and field nail spacing, where edge nailing refers to the outer perimeter of each siding panel, and field nailing refers to stud lines within the panel's area, as shown in Fig. 5. Fig. 7a. shows one wall assembly in the test frame.

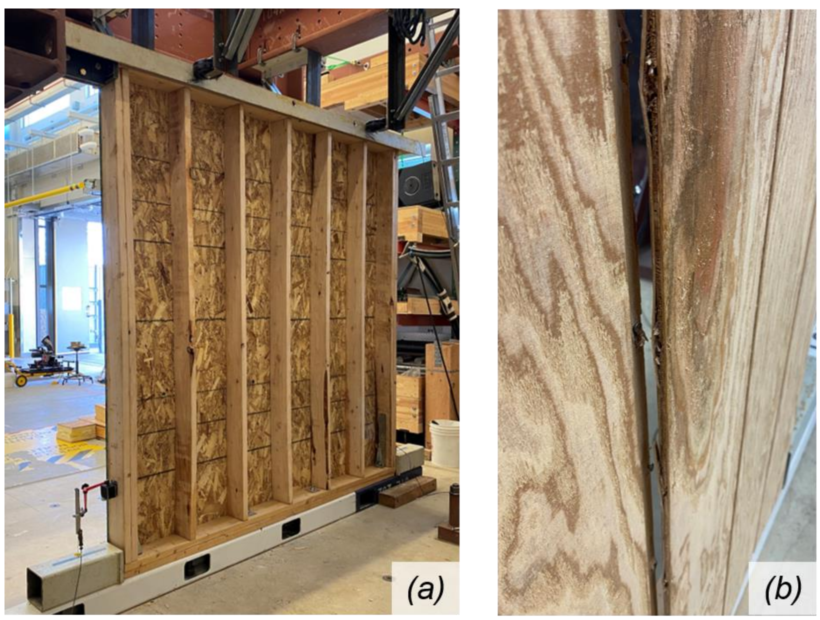


Fig. 7. (a) test wall with OSB sheathing (b) Post-test T1-11 seam with single row of nails.

All walls were attached to the rigid base with 5/8-inch anchor bolts centered on the 2x6 bottom plates and 3-inch square steel plate washers spaced at approximately 30 in. on center. At each end of the wall assembly an HTT5 hold-down was used to connect the chord members to ¾-inch anchor bolts. The HTT5 are manufactured by Simpson Strong Tie and widely used and available in Alaska. Primary walls configurations included in this study can be found in Table 1. Additional details about each specimen can be found in Appendix A.

Inspectors noted that T1-11 siding was frequently improperly attached along the seams between two panels. Instead of a row of nails at the edge of each panel into the stud as specified by the IRC and IBC, a single row of nails is often placed in the small area where the panels overlap. We replicated this deficiency in two locations in the 8 ft x 8 ft wall assembly by modifying the panels into a (2 ft)-(4 ft)-(2 ft) configuration, which allowed the center panel to have this condition along both vertical edges. These tests are described as "double row" or "single row" to indicate the placement of nails at the seams. Fig. 7b shows the seam between T1-11 panels with a single row of nails at the end of the cyclic loading protocol. Panel thickness at the seam is half the full thickness to accommodate a lap joint. This thinner section is more susceptible to failure under lateral loads.

Table 1. Wall tests conducted in this study

		Sheathing	Fastener	Edge Spacing	Count	Notes
		OSB	0.092" nails	6" oc	1	Finish Nails
		OSB	0.131" nails	6" oc	1	No holddown
Box Nails	*	OSB	0.113" nails	3" oc	1	
Box Nails	*	OSB	0.113" nails	6" oc	3	
Box Nails		OSB	0.113" nails	12" oc	1	
Common Nails	*	OSB	0.131" nails	3" oc	4	
Common Nails	*	OSB	0.131" nails	6" oc	5	
Common Nails		OSB	0.131" nails	12" oc	3	
Common Nails		OSB	0.131" nails	6" oc	2	No nails in mudsill
Staples	*	OSB	14 gauge	4" oc	3	
Staples	*	OSB	16 gauge	3" oc	3	
Staples		OSB	18 gauge	3" oc	1	
T1-11	*	T1-11	0.131" nails	6" oc	1	Double row of nails
T1-11		T1-11	0.131" nails	6" oc	3	Single row of nails

*Configurations that meet 2021 IRC prescriptive requirements.

In addition to the configurations listed in Table 1, we tested walls that included no hold-down at corners of wall assembly, 0.092-in diameter finish nails, and 18-gauge staples. These wall configurations performed poorly and were not repeated. Interviews indicated that finish nails and 18-gauge staples are less common than other deficiencies within the MOA.

2.4 Performance Quantification

For each test, we quantified strength, stiffness, and deformation capacity of the wall following the procedure outlined in ASTM E2126 (ASTM 2011). A backbone curve was fit to each recorded hysteretic response, and the above parameters are derived. The backbone consists of points from the hysteretic curve where the highest force was recorded in each primary cycle. Fig. 8 illustrates the hysteretic response and backbone curve for a wall with 0.131-in. diameter nails spaced at 6 inches on center. This is the baseline wall configuration, which is code-compliant per prescriptive requirements defined in the IRC.

Stiffness is computed as the force/displacement prior to system yielding. The wall assembly is considered to have reached its yield limit state when the slope of the backbone curve decreases by 5% or more. Strength is the peak load resisted by the wall specimen during cyclic testing. Ductility is the ratio of displacement between the ultimate displacement, the displacement at which the specimen last resists a load equal to 80% of the peak load, and the yield displacement. These definitions are illustrated in Fig. 8.

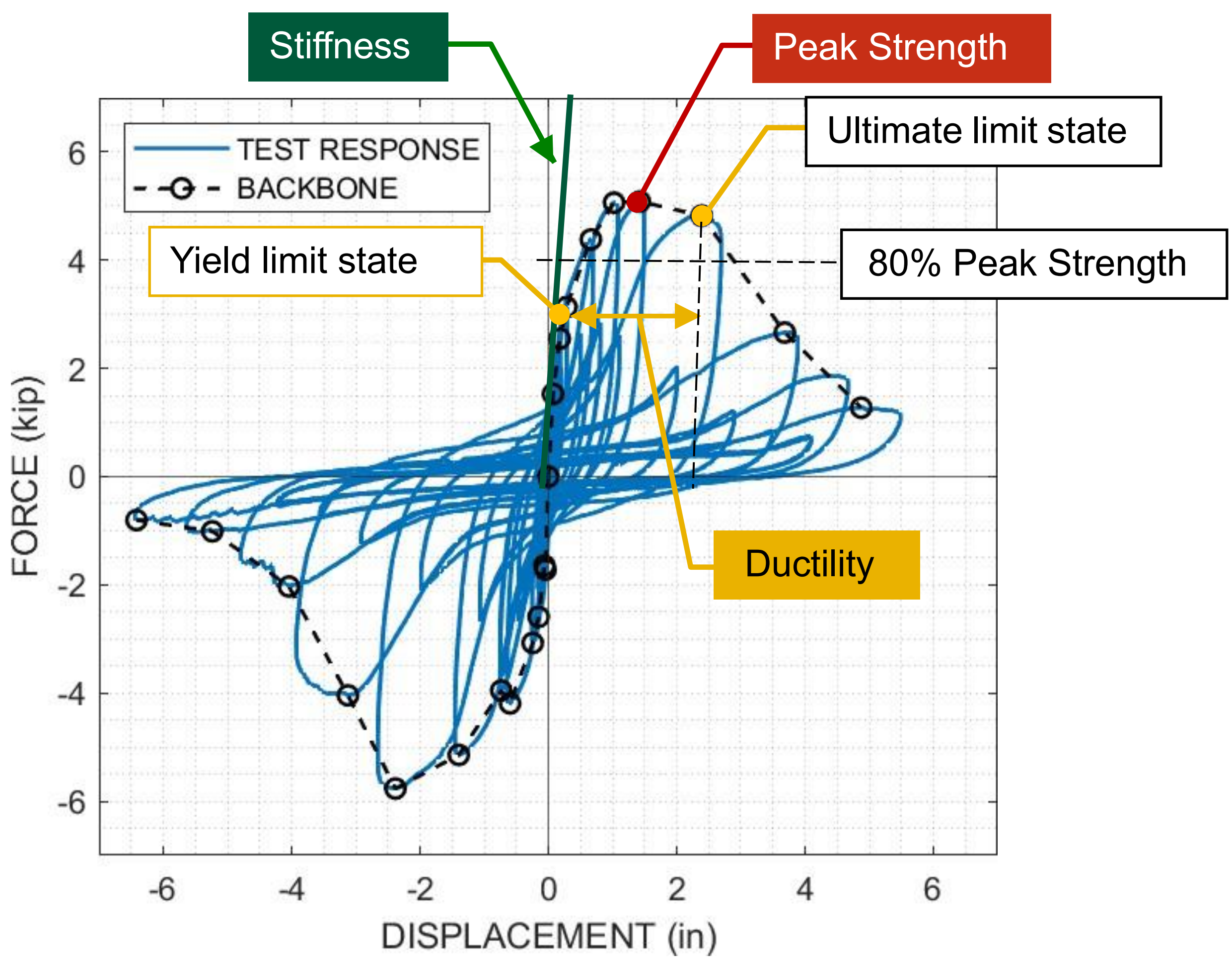


Fig. 8. Hysteretic response of wood-shear wall with 0.131-in. diameter nails with 6-in. edge, 12-in. field spacing.

3 Laboratory Testing Results

A summary of the test performance quantifications from the test results is shown in Table 2. For each test configuration listed in this table, we conducted at least three repeats. Strength, stiffness, and ductility are average values. The failure mechanism and location of failure in wall assemblies varied among tests, dependent on fastener and sheathing type, and the resulting distribution of forces. Fig. 9 illustrates three common mechanisms. For tests with OSB sheathing, crushing of the sheathing surrounding the fastener, or fastener pull through were common. For tests with T1-11, whose behavior resembles that of plywood, nails typically backed out of the studs. Smaller 16-gauge staples typically ruptured at the sheathing-stud interface, but larger 14-gauge staples tended to stay intact while the OSB surrounding the staples failed.

In many cases, failure occurred at the top or bottom plate of the wall assembly. In this failure mode, the nails at the top or bottom edge of the panel backed out or ruptured and then the end nails connecting the top plate to the studs withdrew under the increasing cyclic load. This failure also sometimes occurred at the bottom edge of the wall assembly.

Table 2. Primary wall assemblies tested in this study

Fastener Type	Fastener Spacing (edge/field)	Sheathing	Strength (kip)	Stiffness (kip/in)	Ductility
2-1/2x0.131 inch nails	3/12*	7/16 inch OSB	9.0	13.4	4.8
	6/12*+		5.4	15.0	11
	12/12		3.5	13.5	14.1
	6/12 no mudsill		4.7	7.5	3.3
	6/12 double row	T1-11	4.9	10.6	7.8
	6/12 single row		3.6	5.1	8.8
2-3/8x0.113 inch nails	6/12	7/16 inch OSB	5.4	18.3	10.9
14-ga staples	4/8		7.1	9.6	3.2
16-ga staples	3/6*		7.9	23.4	6.2

*Configurations that meet 2021 IRC prescriptive requirements.
+ Baseline configuration

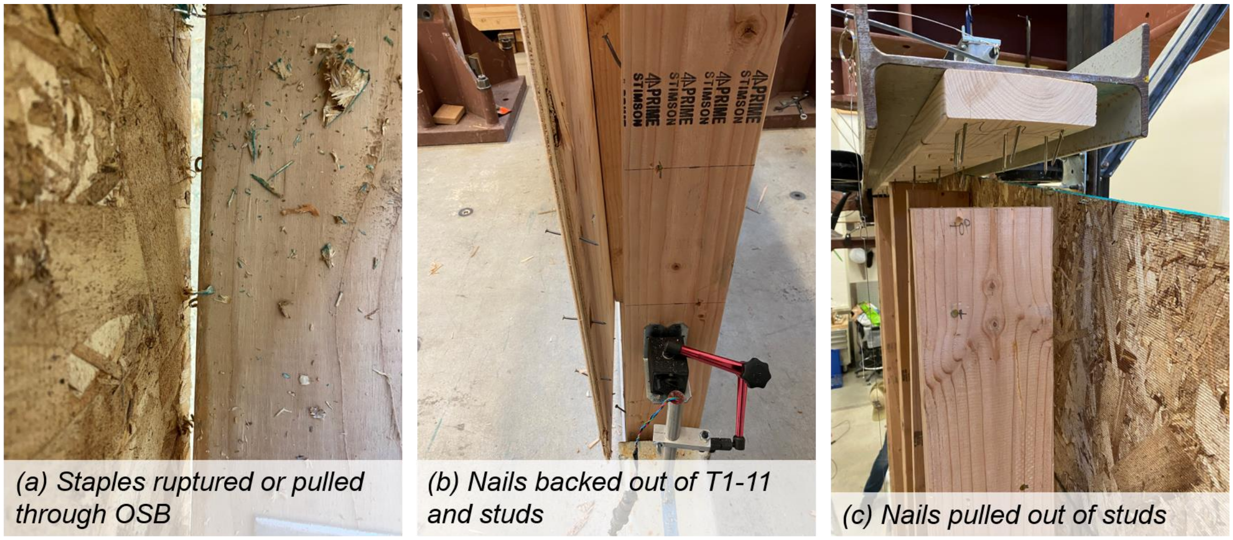

(a) Staples ruptured or pulled through OSB
(b) Nails backed out of T1-11 and studs
(c) Nails pulled out of studs

Fig. 9. Failure mechanisms in wood-frame wall assemblies.

3.1 T1-11 Seam Nailing

When a single row of nails is placed at the edge of a T1-11 panel, the panel and nails separate at low levels of displacement and force, which leads to rupture of the narrow edge of the panel. **Fig. 10** shows the difference in strength provided by an additional row of nails. The double row of nails is roughly equal to the strength of the baseline wall (4.9 kips, or 91%), the single row of nails is held only 3.6 kips, or 66% of the baseline. While the wall assembly with a single row of nails is more ductile, it is likely to lead to more damage at lower shaking intensities than houses with nails correctly placed. It is recommended to mitigate this condition by adding a second row of nails. Since T1-11 is often used as both sheathing and siding, it is typically exposed and not difficult to inspect and retrofit.

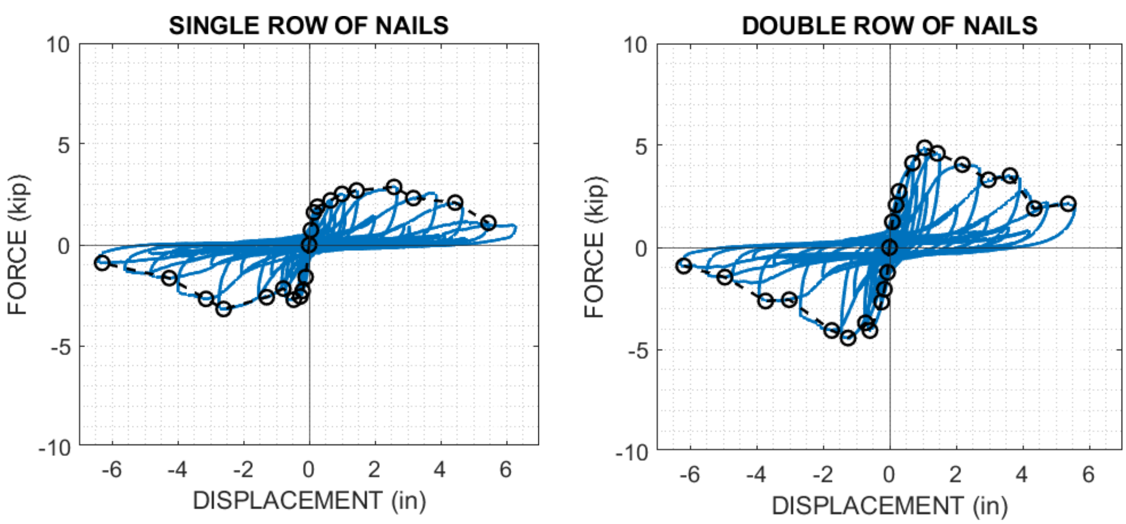


Fig. 10. Wall assemblies with single and double rows of nails at edge of T1-11 panels.

3.2 Staples

In the current edition of the International Residential Code (IRC) (ICC 2021b) 16-ga staples are permitted, and prescribed to be spaced at 3-in. edge, 6-in. field (see Table R602.3(2)). In prior iterations of the IRC, 14-ga staples were permitted, with 4-in. edge, 8-in. field spacing specified. We tested wall assemblies with these combinations and the results are shown in Fig. 11.

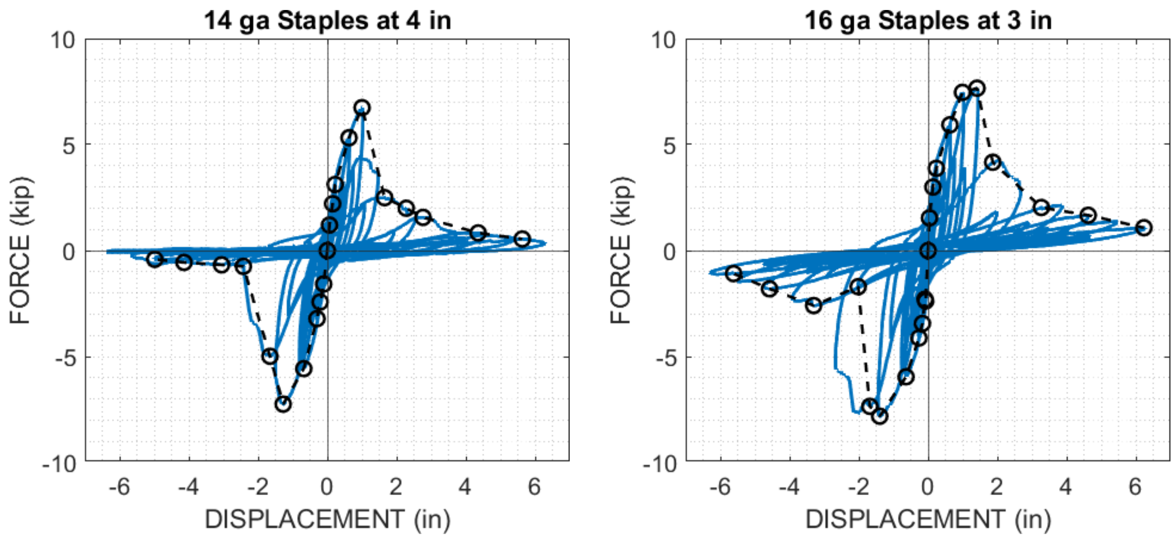


Fig. 11. Hysteretic response of walls staples. with (a) 14-ga staples spaced at 4 inches on center and (b) 16-ga staples spaced at 3 inches on center.

Both configurations are relatively strong (146% and 131% of the baseline configuration, respectively) and experience brittle failure. The ductility of the walls with staples were only 56% and 29% of the ductility of the baseline configuration.

3.3 Nail Spacing

Nail spacing, and the resulting quantity of driven nails resisting loads, had the largest effect on wall strength and ductility. Fig. 12 shows the hysteretic response of walls with 2 ½ x 0.131-in nails with 3, 6 and 12-in. edge spacing. The walls have large variation in peak strengths, exhibiting a nearly linear relationship between nail spacing and strength. As spacing increases, ductility also increases (again almost linearly), while stiffness remains relatively constant (Table 2). The 3-inch spacing had 166% of the strength of the baseline configuration (6-inch spacing), while the 12-inch spacing had only 64% of the strength of the baseline configuration. Meanwhile, the 3-inch spacing had only 44% of the ductility of the baseline wall, while the 12-inch spacing had 128% of the baseline ductility.

Of note, the 2021 SPDWS (ANSI/AWC 2021) indicates that the 3-inch spacing has a design strength that is 187% of strength of these walls with 6-inch spacing, which is more than the 166% revealed in the testing.

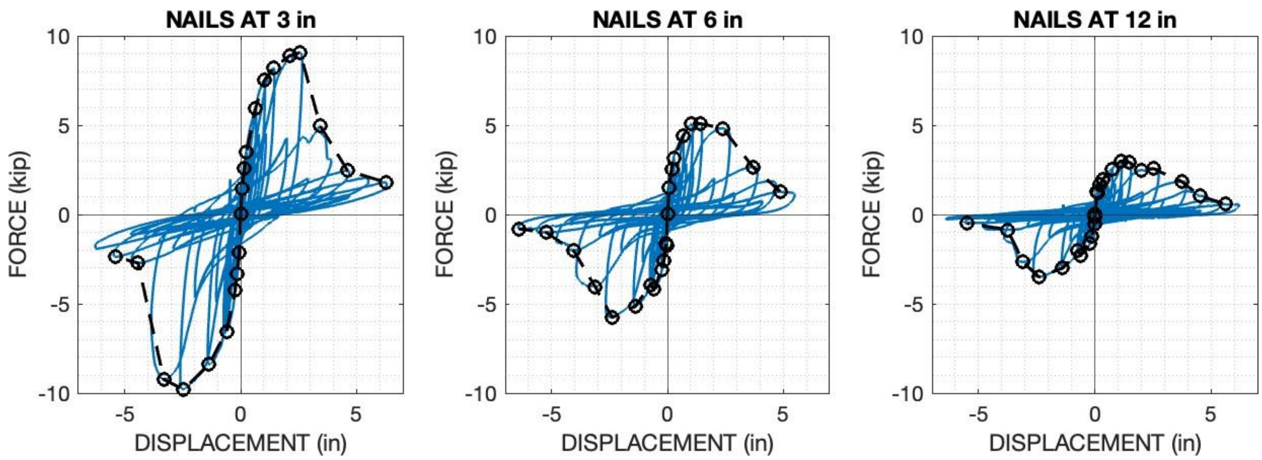


Fig. 12. Hysteretic response of walls with varied nail spacing.

3.4 Nail Size

Nail size, in particular the difference between 8d common nails (0.131 inches diameter) and 8d box nails (0.113 inches diameter) had very little effect on the overall performance of the walls. As shown in Table 2, the strength, stiffness, and ductility are almost identical for the two sizes of nails at the 6/12 spacing.

3.5 Testing Summary

The observations above comparing the various parameters to the baseline configuration allowed a more narrow selection of parameters in the subsequent modeling effort. Since the nail size had little effect, only common nails were used in the modeling portion of the project. Similarly, since the 14 ga staples at 4 inches o.c. and the 16 ga staples at 3 inches o.c. had similar performance, only the 16 gage staples were modeled. Since nail spacing had a large effect, all three spacings were maintained. Six configurations were chosen to include in the computational modeling:

- 8d common nails at 3 inches on center
- 8d common nails at 6 inches on center
- 8d common nails at 12 inches on center
- 16 ga staples at 3 inches on center
- Single row of nails at 6 inches o.c. in T1-11
- Double row of nails at 6 inches o.c. in T1-11

The tests with finish nails and lacking hold-downs resulted in extremely weak and brittle walls and were not selected to be modeled.

A summary of the results of the strength and ductility of the primary wall configurations is shown in Fig. 13. Strengths and ductility are normalized to the baseline configuration.

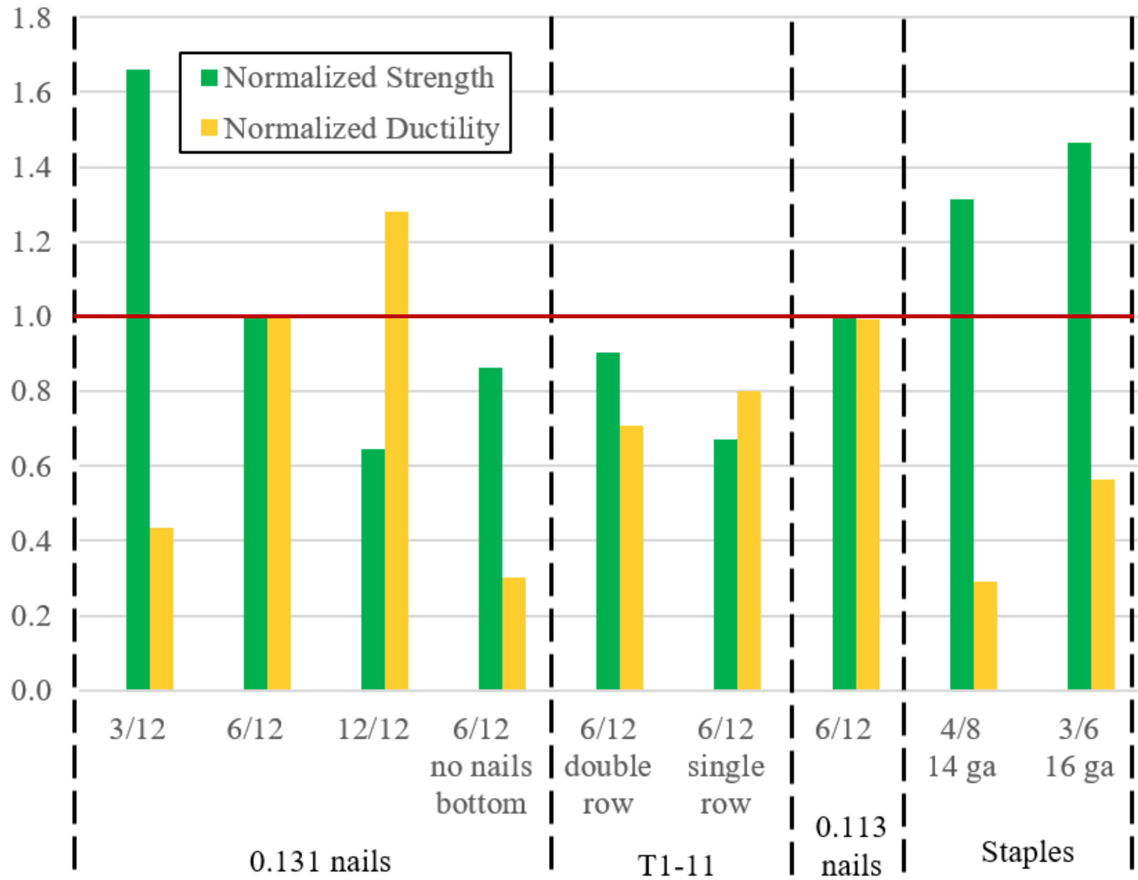


Fig. 13. Hysteretic response of walls with varied nail spacing.

4 Computational Seismic Performance Assessment

To quantify the impact of these construction variations on seismic performance, we developed three-dimensional, nonlinear simulation models of archetype houses and ran a series of ground motions. We first identified typical house archetypes for the region, and designed each archetype to meet a particular design standard. Lateral structural components (walls) of archetype houses are calibrated using test data, and are then included in the full building model. Each ground motion record is applied at an increasing intensity until a collapse threshold is reached; the intensities that cause collapse for each record are aggregated to find a median collapse capacity for each building.

4.1 Archetype Houses and Design

Residential construction in Southcentral Alaska is represented by three archetype single-family houses in this study. Each archetype, listed in Table 3, is designed to meet code requirements with a certain shear wall configuration and analyzed for all shear wall configurations considered in this assessment. Houses are all 38 ft in the E-W direction, and 26 ft in the N-S direction and are shown in Fig. 14. Each house was designed using the 2021 SPDWS

(ANSI/AWC 2021), which is referenced by the 2024 International Building Code (IBC) (ICC 2021a). Using a typical plan layout, masses were calculated and then utilized in the Equivalent Lateral Force method from ASCE/SEI 7-22 (2022) to determine shear wall forces. These were compared with calculated wind loads to ensure that the seismic forces governed. The walls were then designed using the "Perforated Wall Method" from SPDWS with enough wall openings so that the calculated forces were just below the capacity of the target shear wall design (typically governed by nail spacing). As such, each house is close to its structural limit according to the IBC and NDS design methodology. Details of each house design can be found in Appendix E.

The perforated wall strengths were then compared to the strength of the analogous group of unperforated 8 ft. wall segments, to be used in the computational models. For example, the perforated wall in SF2 provides a strength of 8.8 kips per side along E-W walls. This is just slightly more than the required wall force of 8.7 kips. The wall strength was compared to the strength of four independent 8-foot wall segments, equal to 11.6 kips, resulting in a strength ratio of 0.76. This ratio must be applied to the strength of the walls in the computation model so that they perform analogous to the perforated wall. A similar procedure was followed using calculated deflections to determine the Stiffness Ratio. Applying the calculated wall force of 8.7 kips to the four 8-ft modeled walls resulted in a force of 271 plf, which causes a deflection of 0.266 inches. However, the same 8.7 kip load applied to the much stiffer 38 foot long perforated wall causes a deflection of 0.135 inches, which results in a stiffness ratio of 1.98.

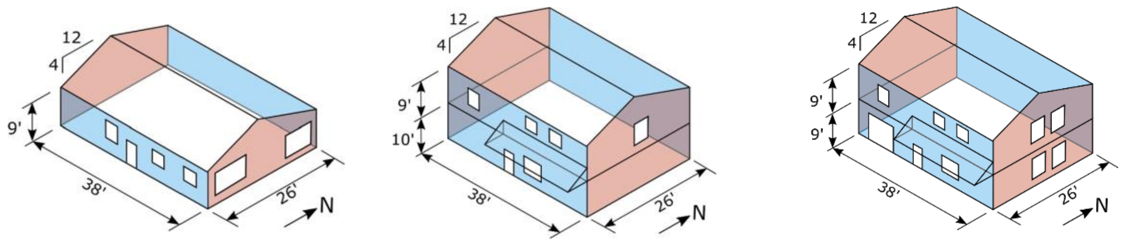
Thus, in the case of the EW walls in SF2, the perforated design walls are weaker and stiffer than the four 8-foot walls in the model, and the modeled walls were modified accordingly. The Strength and Stiffness Ratios for all of the archetype houses are shown in Table 3.

4.2 Computational Models

Using open-source earthquake engineering simulation software, *OpenSees* (McKenna et al. 2010), we developed nonlinear models representative of each archetype house and shear wall configuration. These three-dimensional models, illustrated in Fig. 15 consider only the structural system, neglecting the effects of wall finishes and nonstructural components, as these have a small impact on the seismic behavior ANSI/AWC (2021) and are consistent across house archetypes.

Table 3. Design of archetype houses.

			Modeling E-W Walls			Modeling N-S Walls		
ID	Stories	Nail Spacing	No. 8x8 Walls	Strength Ratio	Stiffness Ratio	No. 8x8 Walls	Strength Ratio	Stiffness Ratio
SF1	1	6" o.c.	1	1.5	1.59	1	1.5	1.59
SF2	2	6" o.c.	4	0.76	1.98	3	1.1	1.23
SF3	2	3" o.c.	3	0.55	1.19	2	0.93	1.4



(a) SF1 (b) SF2 (c) SF3

Fig. 14. Dimensions and Layout of Archetype houses

The model consists of a frame of elasticBeamColumn elements, with diagonal struts in locations where shear walls are present. These struts capture nonlinear behavior, and are calibrated using the Pinching4 material, which captures changes in stiffness and degradation. Masses are applied throughout the model based on anticipated dead loads, 25% of live loads and 20% of snow loads (ASCE/SEI 7-22 2022). The floor system and roof are represented with a rigidDiaphragm, which constrains specified nodes to translate and rotate as a rigid plane. Rayleigh damping with a critical damping ratio of 5% is applied at the first and third modes (Jayamon et al. 2018). The first mode period of each building, determined through eigenvalue analysis, is 0.2 s.

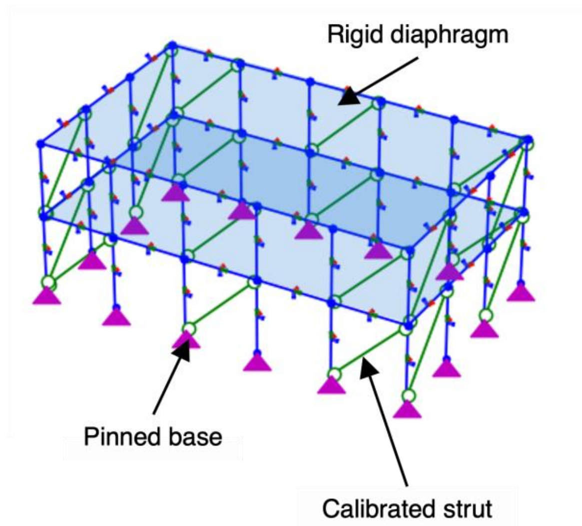
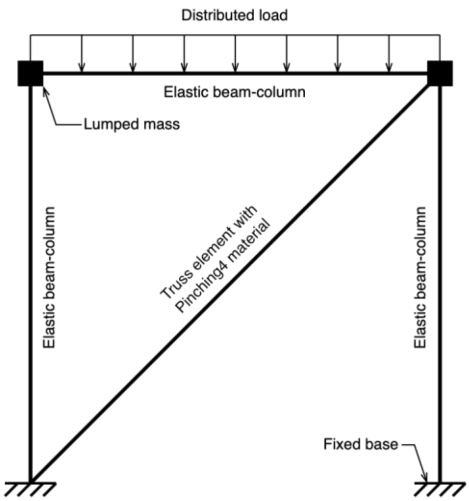


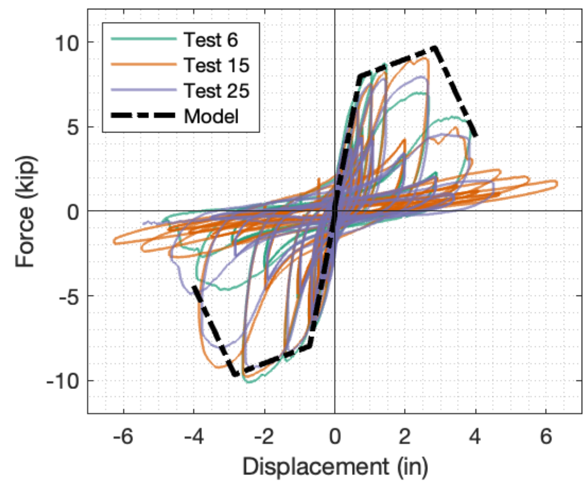
Fig. 15. 3D model schematic for one house archetype

For each shear wall configuration, we calibrated a standalone 8 ft x 8 ft wall with elasticBeamColumns and a diagonal strut, as shown in Fig. 16a based on test data. For each configuration, we had between 3 and 5 load-displacement curves (hystereses) from which to calibrate strength, stiffness, deformation capacity, and cyclic degradation. Positive and negative branches of backbone curves are averaged (ASTM 2011) for each test and wall configuration. Fig. 16a shows the *OpenSees* model used to calibrate an 8 ft x 8 ft wall section to test data. First, beam and column components were calibrated to test data for a wall with no sheathing. Next, a truss element—carrying only axial load—was calibrated such that the panel section matched the behavior of the wall assembly under cyclic load. The test backbones and calibrated backbone curve for one configuration are shown Fig. 16b.

4.3 Nonlinear Dynamic Analysis

For each archetype house and wall configuration, we determined the collapse capacity using Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell 2002), where ground motion records are scaled to increasing intensity until a drift threshold is reached. Intensity is based on the spectral acceleration of the ground motion at the period of the structure, $Sa(T = 0.2s)$, and scaled in increments of 0.2 g. We used 22 ground motion record pairs included in FEMA P-695 (2009) for far-field locations. These ground motion records include variation in frequency content and duration, have been widely used in seismic safety analysis, and are appropriate for Southcentral Alaska's seismic hazard (Petersen et al. 2024; Mueller et al. 2015). Ground motions are scaled based on the geo-mean of the spectra pair and applied to the model in both orthogonal directions.

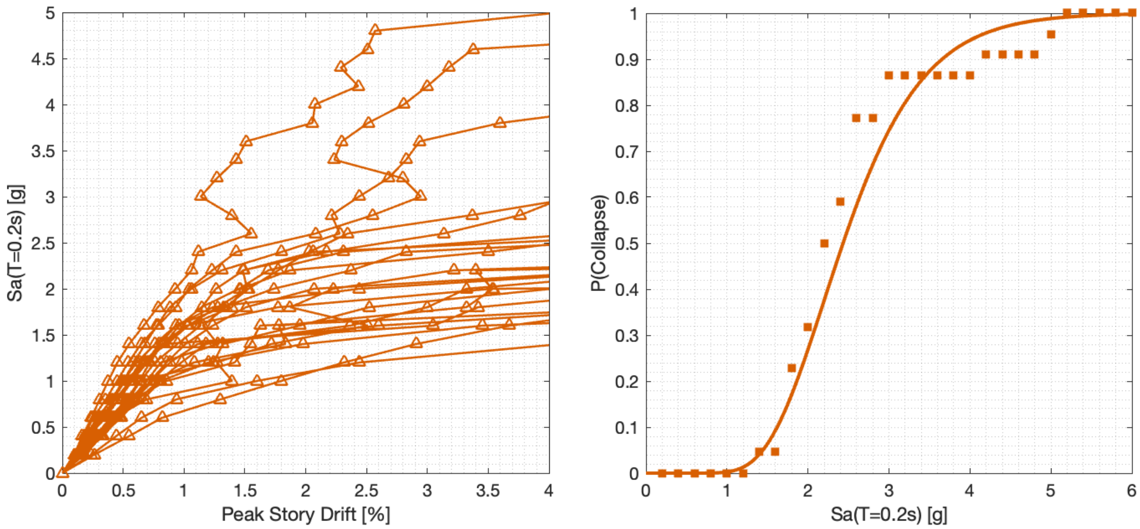




(a) *OpenSees* model of calibration wall. **(b)** Test data and calibrated backbone curve.

Fig. 16. Calibration of *OpenSees* models.

Story drifts are computed during each step of the analysis from node displacements. If drifts exceed 4% in either direction, the structure is considered collapsed. This value is chosen based on the drifts at which wall components lost most of their strength resistance, and at which the IDA curves are virtually horizontal, and is in line with similar assessment of wood frame structures, *e.g.,* Goda and Yoshikawa (2013), Christovasilis et al. (2009). Fig. 17a shows the IDA response for the two-story house designed for 8d nails spaced at 3 inches, with walls correctly detailed. Each line represents one of 22 ground motion record pairs; each marker represents a single analysis at a particular intensity. From these data, we can determine the probability that a ground motion of a given intensity will lead to collapse. Fig. 17b illustrates the fractional probability of collapse at each intensity increment and the associated collapse fragility curve. This curve represents the lognormal cumulative distribution for the collapse intensities.



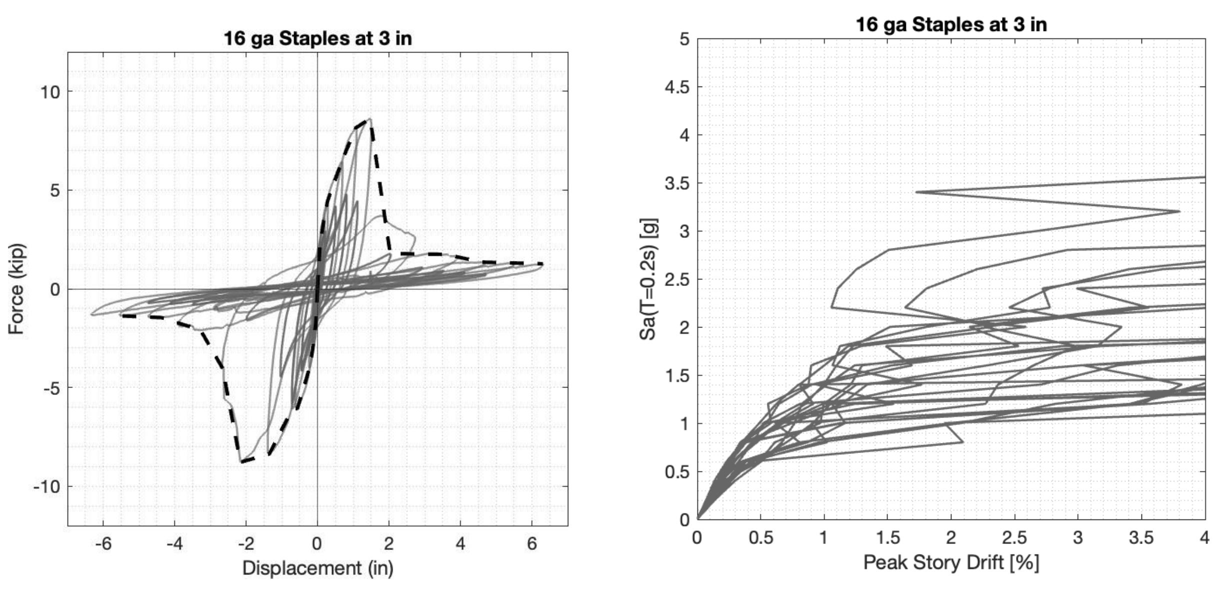
(a) IDA results for two-story house built with 3 inch nail spacing.

(b) Fractional probability and collapse fragility curve for building shown.

Fig. 17. Development of collapse fragility curves

5 Seismic Performance Assessment Results

The following figures illustrate the relationship between cyclic test behavior and IDA results for two wall configurations in SF2. Peak strength during cyclic testing is a strong predictor of the collapse capacity of the corresponding model during IDA.

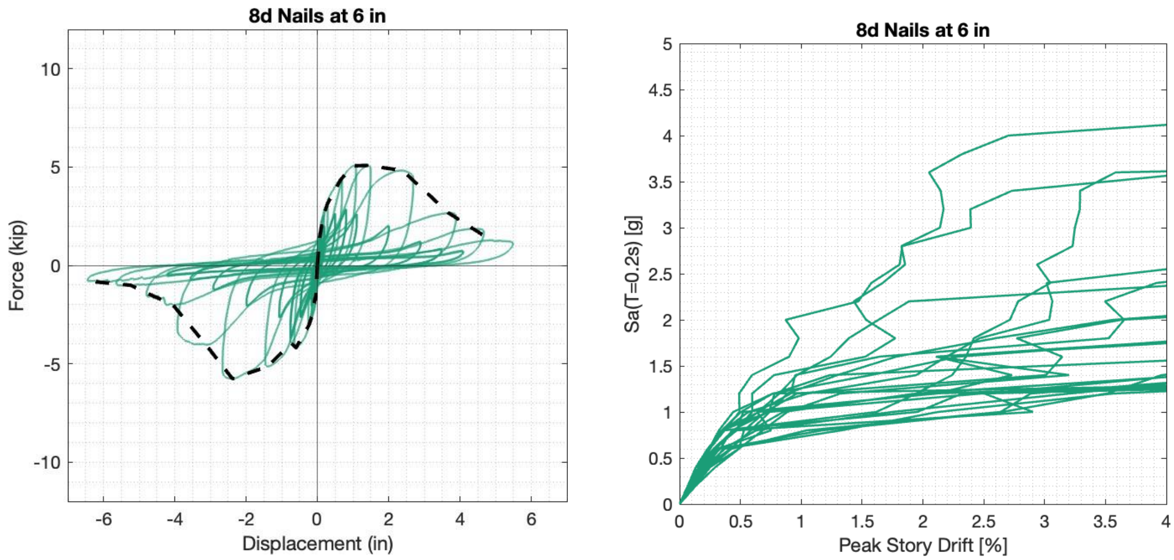


(a) Cyclic force-displacement response

(b) IDA results

Fig. 18. walls with OSB fastened with 16-gauge staples spaced at 3 inches.

We subjected each of the 18 nonlinear models to IDA using the 22 ground motion record pairs detailed in FEMA P-695 (2009). From these analyses, we compute the median collapse capacity based on the ground motion intensities at which the model recorded drifts in excess of the collapse threshold. These collapse capacities are listed in Table 4.



(a) Cyclic force-displacement response (b) IDA results

Fig. 19. Walls with OSB fastened with 8d nails spaced at 6 inches (SF3).

Table 4. Median collapse capacity for each configuration at Design Earthquake and intensities

		Med. Collapse Capacity Sa (T=0.2s) [g]		
Panel	Fastener	SF1	SF2	SF3
OSB	8d Nails at 3 in	3.4	3.1	2.4*
OSB	8d Nails at 6 in	2.7*	2.4*	1.9
OSB	8d Nails at 12 in	2.2	1.6	1.3
OSB	16 ga. Staples at 3 in	2.6	2.5	1.9
T1-11	Double row of nails at 6 in	2.7	2.3	1.8
T1-11	Single row of nails at 6 in	2.2	1.6	1.3

*Configuration used for house design

The collapse fragility curve for each archetype can be found in Fig. 20, Fig. 21, and Fig. 22. These curves illustrate the relative collapse risk for each wall configuration for a given house design. The probabilities of collapse for each configuration at DE and MCE_R intensities are listed in Table 5. Notably, only configurations which match or exceed (*i.e.,* closer nail spacing) design specifications meet or nearly meet ASCE 7-22 collapse probability target reliability at the MCE_R level, which is 10% for Risk Category II structures (ASCE/SEI 7-22 2022). The house designed for and built with 3 in nail spacing has a collapse probability of 13 at the MCE_R, which is above the target reliability level and likely a result of modeling assumptions (*e.g.,* perforated wall approximations). The accelerations from the November 2018 earthquake are shown as a grey band that represents +/-1 standard deviation of the measured short-period spectral responses in the Anchorage bowl, as shown in Fig. 3. It is notable that the spectral accelerations experienced during the November 2018 earthquake predict very low collapse probabilities (1% - 5%), even for structures with deficient construction. This is consistent with the reported results of the actual building stock during the event, in which only a couple of buildings within the Municipality of Anchorage experienced non-geotechnical-related collapse.

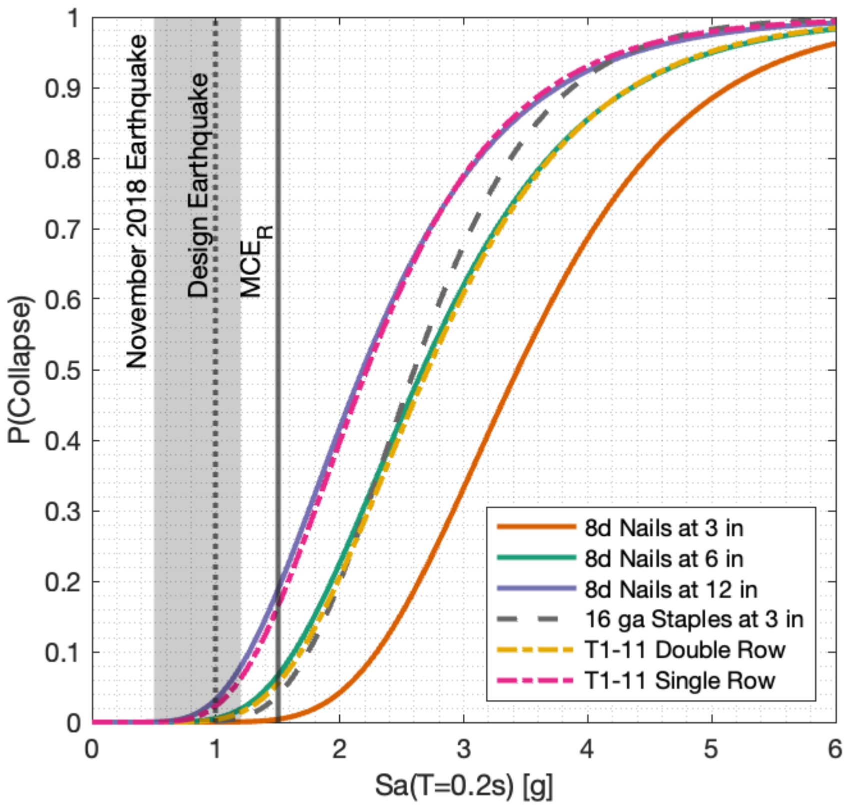


Fig. 20. Collapse fragility curves for one-story house designed for 6 in nail spacing (SF1).

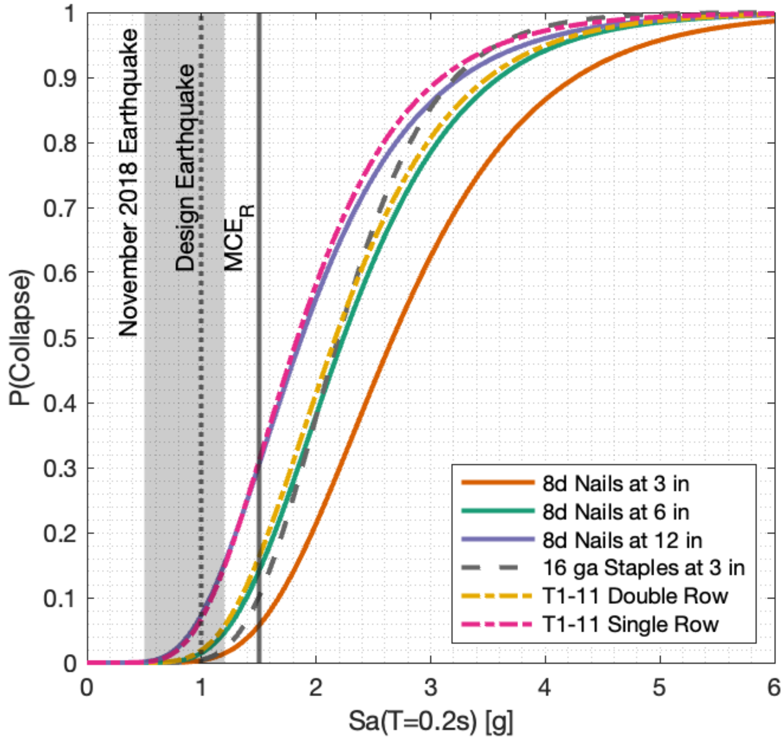


Fig. 21. Collapse fragility curves for two-story house design for 6 in nail spacing (SF2).

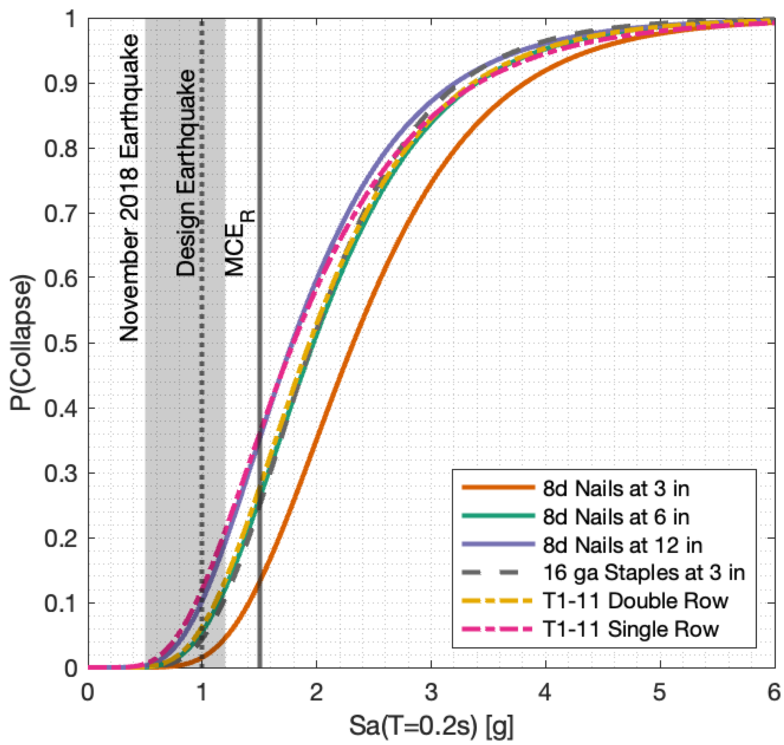


Fig. 22. Collapse fragility curves for two-story house design for 3 in nail spacing (SF3).

Table 5. Probability of collapse for each configuration at Design Earthquake and MCE_R intensities

		Probability of Collapse					
		Design			MCE$_R$		
Panel	**Fastener**	**SF1**	**SF2**	**SF3**	**SF1**	**SF2**	**SF3**
OSB	8d Nails at 3 inches o.c.	0.0*	0.0*	1.5*	0.4*	0.0*	13*
OSB	8d Nails at 6 inches o.c.	0.5*	0.5*	5.5	6.6*	8.9*	26
OSB	8d Nails at 12 inches o.c.	3.3	10	10	19	43	35
OSB	16 ga. Staples at 3 inches o.c.	0.1*	0*	4.4	3.8*	1.8*	24
T1-11	Double row of nails at 6 inches o.c.	0.3*	0.4*	6.2	5.5*	8.6*	28
T1-11	Single row of nails at 6 inches o.c.	2.3	11	12	16	45	34

*Wall configuration meets design specification for archetype

**Wall configuration exceeds design specification for archetype

6 Conclusions

This study outlines efforts to quantify the cyclic performance of wood-frame construction that is both code compliant and non-code compliant. Construction often varies from code-prescribed configurations due to the variety of different enforcement strategies of building codes (both at the design and construction stages) across the country. Anchorage, AK, and the performance of its building stock in the 2018 earthquake, offer a unique glimpse into this variation due to the hugely different levels of enforcement in two geographically adjacent areas that experienced similar shaking intensities.

We conducted cyclic laboratory testing of 37 walls, and quantified strength, stiffness, and ductility for each wall using the methods outlined in ASTM E2126 (ASTM 2011). The results indicated that fastener spacing has the most significant impact on strength (i.e., more fasteners lead to higher overall strength), and that fastener type has the largest impact on ductility. Ductility is a function of peak strength, as it is dependent on the point at which forces drop below 80%. Failure mechanisms that include crushing of sheathing and nail backout tended to be more ductile than those that experienced fastener breakage. Structures with low peak strength also tended to be more flexible and ductile.

Seismic performance analysis consisted of developing three-dimensional nonlinear models for three archetype houses: a one-story house designed for 6 in nail spacing, a two-story house designed for 6 in nail spacing, and a two-story house designed for 3 in nail spacing. For each of these archetypes, we assessed collapse fragility for the house with 6 wall con- figurations: OSB with 8d common nails spaced at 3, 6, and 12 in, 16 ga. staples spaced at 3 in, and T1-11 panels with nails spaced at 6 in with a single or double row of nails at panel edges. We ran IDA with 22 ground motion record pairs for each of these configurations and determined the collapse fragility by computing the median ground motion intensity that causes collapse, in terms of spectral acceleration at the period of the structure. We then fit a lognormal CDF to these intensities to compare the relative safety of each archetype-configuration pairing.

The results indicate that structures built to or above design standards generally meet ASCE 7-22 target reliability standards, with probabilities of collapse at MCE_R level ground motions below or around 10%. Collapse risk was most strongly correlated with strength of wall panels, and fastener spacing has the largest impact on strength. Houses with half the edge nails (spaced at double design spacing) are 2 to 5 times more likely to collapse during an MCE_R level event. T1-11 fastened improperly leads to a similar increase in collapse probability when compared to correctly fastened walls due to the significant loss in wall strength. Wall configurations that provided more ductility (*e.g.,* those with nails) had more gradual increases in collapse risk with increasing spectral acceleration than those with more brittle failure modes (*e.g.,* those with staples). For example, at higher ground motion intensities (*e.g.*, Sa = 3g), the differential in the probability of collapse between a house designed with staples instead of nails, is larger than at lower intensities (*e.g.,* Sa = 2g) This could be described as a higher level of resilience in large earthquakes.

These structures all have short fundamental periods, around 0.2 s. Based on design spectra for the building site, they see very large seismic forces. This analysis focuses on a "worst-case" seismic event.

These models indicate that some structures, such as those with large nail spacing or incorrectly fastened T1-11 panels, have high collapse risks at design and MCE_R level ground motions and non-negligible collapse risk at Design-level ground motions. Shaking during the November 2018 earthquake was generally at or below design earthquake intensity, and as expected, houses in Anchorage did not see collapse on a large scale during the event. While the moment magnitude of this earthquake is relatively high, the intensity of shaking was much lower than both of these design levels. Residents perceive that earthquake as "The Big One", but should be prepared for a much bigger earthquake in the future.

7 Recommendations for Mitigation

There are several key takeaways from this project. The first is that design codes work. All the models that were properly designed met the intended probability of collapse threshold of less than 10 percent. This a positive indication that with proper design and inspection, residential structures have a low probability of collapse during an MCE_R level event.

It should also be noted that in general, closely spaced fasteners increase strength of wood walls, which reduces the probability of collapse. Ductility is dependent on the points at which the lateral systems begin to yield and when they see a significant loss in capacity, which leads to a decrease in ductility as strength increases Both strength and ductility affect seismic capacity, but strength has larger impact. It was found that nails are more ductile than staples, though staples provide a higher initial strength. In structures with similar strengths, those that were more ductile had higher performance.

Based on these findings, we recommend the following:

- Building codes should be enforced through plan review and onsite construction inspections
- T1-11 laps should be double nailed, and inspectors should pay particular attention to this during construction.
- Additional nails should be added in shearwalls in houses suspected of having inadequate length of shear walls or a low number of fasteners.
- Houses should be inspected to ensure adequate hold-downs
- Despite being stronger but less ductile in testing, modeling indicates that staples provide the same level of collapse prevention. However, given the choice, we recommend nails.

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Appendix A: Interview Summaries

Selected Interview Summaries

Date: March 30, 2022
Attendees: Dale McCoy (LCG Lantech), Scott Hamel and Polly Murray (UAA)

- State requires IBC 2012, Municipality of Anchorage is 2018 IBC
- Land-use permits in Eagle River, including for expansion (requires details of existing structures and site)
- Conducted 100+ post-earthquake inspections, mostly houses, some light commercial
- Damage
    - Lots of separations (diaphragm to wall)
    - Chimney failures
    - Slope failure and liquefaction (especially in Anchorage bowl) - primarily fixed by jacking up foundation and injecting foam or grout
    - Ground failure, often from poorly stabilized fill
- Deficiencies
    - Missing hold downs (residential, commercial), in one case hold down wan't aligned
        - Muni doesn't require hold downs if uplift is less than 1500 lb for ground story, 1000 lb for second story
    - Lack of wall ties/out of plane bracing
    - "Lots of short cuts" (especially among developers)
    - Insufficient nailing in exterior walls
    - Connections that were not designed
    - Plywood web joists would buckle due to lack of rim joists
- Muni requires at least two inspections (foundation, framing)
- Muni has record of earthquake repairs, in safety zone
- Muni Amendments to building code are confusing and could lead to substandard nailing patterns
- Wood foundations are not recommended

Date: May 18, 2022
Attendees: Wayne Bolen, Don Hickel, Phillip Calhoun, Ross Noffsinger (MOA); Scott Hamel and Polly Murray (UAA)

- Common deficiencies:
  - Incorrect sheathing
  - Wrong nail spacing and sizes (finish/siding nails instead of framing)
  - Wrong size or spec of studs
  - T1-11 with single nails at 12”
  - Wrong materials (7/16” vs 5/8” T1-11)
  - Poor stud quality
  - No boundary nails
  - Overdriven nails
  - No nailing to mud sill
  - staples (14 ga allowable in code [IRC says 15 or 16 ga], not always followed)
- ¾ of inspected houses had nailing issues
- Inspectors are often hired by builders (and not thorough)
- Failure in wood foundations from sealing, poor construction, no sill
- Within service area ~500 homes sank, lots of fine silty soil
- No collectors or ties at roof (mostly commercial)
- Within service area, third party plan review. Or engineer can sign off on plans
- History of building
  - Pre 70s/80s – no nail guns, 16d nails for framing
  - Late 70s – many issues (siding), no quality control
- Earthquake damage
  - Nonstructural damage in commercial buildings (near airport heights)
  - Collapse of chimneys/fireplaces
- Step foundations, stepped walls not tied down
- Horizontal sheathing not blocked
- No shear transfer in roof to wall connection
- Toe-nailing limited to 50 plf
- Holddowns – not used or not enough capacity
  - Assume none in Eagle River
- Anchor bolts not within 1/2” of sheathing
- Cross-grain bending
- Cut washers (non plate) in eagle river
- Outreach
  - Contractor training – credit hours (ICC) – contact Mark Panilo
  - Inspections offered through MoA
- Suggested archetypes
  - Split level house
  - Soft stories

- Prow houses
- Stepped hillside foundations

Date: August 23, 2022
Attendees: Andre Spinelli (Spinell Homes); Scott Hamel and Polly Murray (UAA)

Observations of damage from November 2018 earthquake

- 1984 house in Eagle River detached from foundation due to lack of hold downs and anchor bolts. House had no shear panels

Observations of construction quality issues

- Loose holddown nuts
- Nails too short in hold downs (TECO nails)
- Straps around stairwell openings were cut during floor installation

Archetypes of interest

- Tall crawl spaces, houses on slopes
- Prow houses
- Split and tri-level homes

Notes on design and building procedures

- Code used to call for 14 ga. staples, now calls for 16 ga.
- Code amendments modify blocking requirements
- Special wind requirements in BSSA with Muni amendments
- Special load testing for garage floors by Muni

Outreach recommendations

- Carpenter's Union
- Mat-Su homebuilders association

Inspection and plan review process

- Most new homes are inspected
- Dave Owens – AHFC 101, 102 inspection

Date: March 30, 2022
Attendees: Dave Stierwalt (Reid Middleton), Scott Hamel and Polly Murray (UAA)

- Reid Middleton conducted 192 post-earthquake inspections, half school, half commercial, a few residential
- Damage
    - Nonstructural cracks everywhere, cracks injected with epoxy generally
    - Cracks repaired with fiber wrapping (also retrofit strategy)
    - Yielding in rod/bracing
    - Top of wall failure
    - Failure at diaphragm connection
    - Missing hold downs and sills
    - Cross-grain ledger failure
    - Cracking in drywall
- Building irregularity bumps requirements from IRC to IBC
- Residential plans, generally, do not require details of structural/lateral systems
- Hold downs generally weren't used until 5-10 years ago
- Muni may know how many new home plans called out hold downs
- AGC contractor group might be helpful
- NHBA framing guide

Date: March 28, 2022
Attendees: Joe Lawendowski (Pannone Engineering Services), Scott Hamel and Polly Murray (UAA)

- Conducted residential and commercial inspections, wood frame and CMU buildings
- Damage and deficiencies
    - Soil failure, sinking, liquefaction, and differential settlement
    - Separation at roof, along middle line
    - Liquifaction issues, sand bubbling up into basements and causing foundations to sink
    - Excavation issues, houses shifted down slopes
    - Lack of nails - use of 18 gauge staples in 78 - 83
    - T1-11 studs not doubled at overlap
    - Tall walls without blocking
    - Exterior walls built with 2x4s
    - Unreinforced masonry, sometimes built incrementally
    - Lack of hold downs and anchor bolts
    - Posts not connected to footings
    - Overdrilled fasteners
    - General lack of blocking
- Inspection observations
    - Tension cracks in soil
    - Walls out of plumb
    - Cracks in drywall
    - Racked doors
- Building observations outside BSSA
    - Too many windows
    - Inspection (ICC) required for loans
    - Plans not spec'd out
    - Odd designs with bumpouts, bi-level construction

Date: February 11, 2022
Attendees: Anita Baker, Pauletta Bourne, and Sally Cox (Division of Community and Regional Affairs, State of Alaska), Scott Hamel and Polly Murray (UAA)

- Funding sources
    - Community development block grant - HUD funded
    - CBDG mitigation program
- Damage from lack of design adn oversight
- Some dissemination of seismic hazard from Muni (soil failure only)
- Soil conditions are vulnerable to liquefaction
- Does the muni have housing stock maps? What percentage of homes have certain deficiencies?
- 2012 IBC in place during the 2018 earthquake
- USGS intends to generate new hazards maps for the area
- They plan to hire inspectors, buy out vulnerable homes
    - $35 million
    - 70% to be spent on low to middle income residents
- How to increase inspection throughout the city?
    - Train firefighters to look for nonstructural vulnerabilities, structural issues
- Muni has records of year, type of construction, and damage assessment?
- 50 homes currently need reinspection
- Homeowners not required to disclose hazards
- Discussion of condos on La Honda drive
    - 80s era, cantilever construction
    - Inadequate foundations

Appendix B: Wall Testing Details

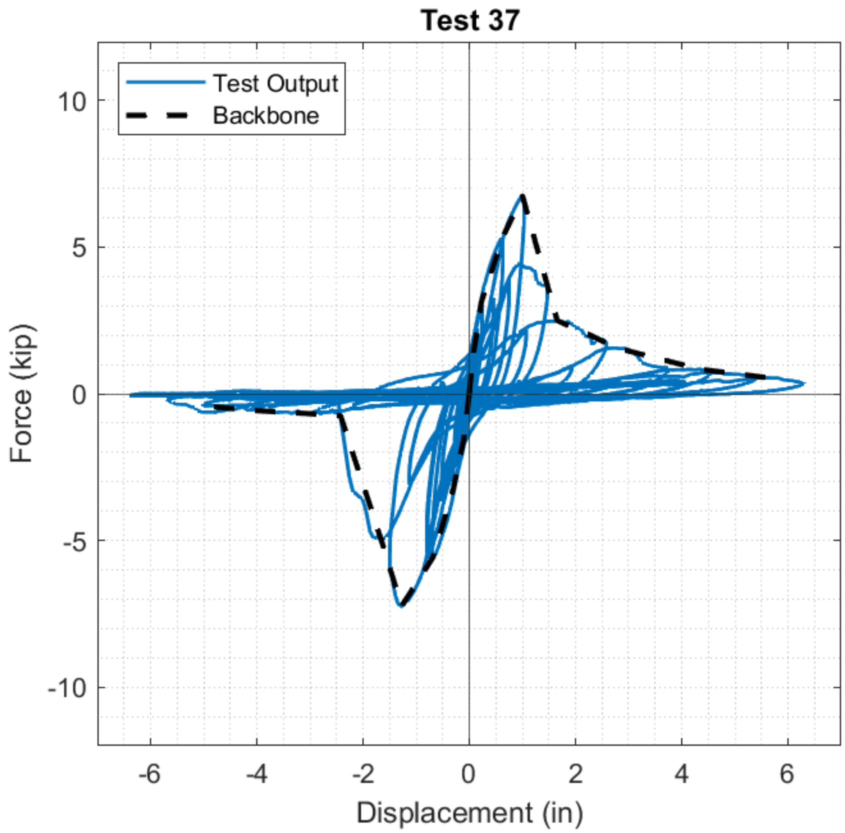
Date	Test No.	Test ID	Designation	Panel Type	Nail Type	Sheathing Nail Size	Sheathing Nail Spacing (edge)	Nail Spacing (field)	Framing Type	Stud Spacing	# of Center stud(s)	Top/bottom plate to stud fasteners
2022-05-27	1	O-113-H-3-a	O-113-H-3	7/16" OSB	Bright nails	2-3/8" x 0.113"	3"	12"	Grade 2 Hem Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-06-02	2	O-113-H-6-a	O-113-H-6	7/16" OSB	Bright nails	2-3/8" x 0.113"	6"	12"	Grade 2 Hem Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-06-10	3	O-113-H-12-a	O-113-H-1	7/16" OSB	Bright nails	2-3/8" x 0.113"	12"	12"	Grade 2 Hem Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-06-16	4	O-131-H-6-a	O-131-H-6	7/16" OSB	Bright nails	2-1/2" x 0.131"	6"	6"	Grade 2 Hem Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-06-23	5	O-16g-H-3-a	O-16g-H-3	7/16" OSB	Staples	16 ga	3"	6"	Grade 2 Hem Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-06-30	6	O-131-H-3-a	O-131-H-3	7/16" OSB	Bright nails	2-1/2" x 0.131"	3"	12"	Grade 2 Hem Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-07-12	7	O-092-H-6-a	O-092-H-6	7/16" OSB	Finish nails	2" x 0.092"	6"	12"	No. 2 Hem Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-07-12	8	O-131-H-6-r	O-131-H-6	7/16" OSB	Bright nails	2-1/2" x 0.131"	6"	12"	Grade 2 Hem Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-07-14	9	O-131-N-6-a	O-131-N-6	7/16" OSB	Bright nails	2-1/2" x 0.131"	6"	12"	Grade 2 Hem Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-07-15	10	O-131-H-12-a	O-131-H-1	7/16" OSB	Bright nails	2-1/2" x 0.131"	12"	12"	No. 2 Hem Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-07-19	11	O-18g-H-3-a	O-18g-H-3	7/16" OSB	Staples	18 ga	3"	6"		16" o.c. (nominal)	1	3-3"x0.131"
2022-07-20	12	T-131-H-1-a	T-131-H-1	T1-11	Bright nails	2-1/2" x 0.131"	6"-1 row center	12"		16" o.c. (nominal)	1	3-3"x0.131"
2022-07-20	13	T-131-H-2-a	T-131-H-2	T1-11	Bright nails	2-1/2" x 0.131"	6"-2 rows center	12"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-07-22	14	T-131-H-1-b	T-131-H-1	T1-11	Bright nails	2-1/2" x 0.131"	6"-1 row center	12"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-07-22	15	O-131-H-3-b	O-131-H-3	7/16" OSB	Bright nails	2-1/2" x 0.131"	3"	12"	Doug Fir	16" o.c. (nominal)	2	3-3"x0.131"
2022-08-08	16	O-113-H-6-b	O-113-H-6	7/16" OSB	Bright nails	2-3/8" x 0.113"	6"	12"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-08-09	17	O-113-H-6-c	O-113-H-6	7/16" OSB	Bright nails	2-3/8" x 0.113"	6"	12"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-08-10	18	T-131-H-1-c	T-131-H-1	T1-11	Bright nails	2-1/2" x 0.131"	6"-1 row center	12"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-08-11	19	O-16g-H-3-b	O-16g-H-3	7/16" OSB	Staples	16 ga	3"	6"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-08-16	20	O-16g-H-3-c	O-16g-H-3	7/16" OSB	Staples	16 ga	3"	6"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-08-18	21	O-131-H-6-b	O-131-H-6	7/16" OSB	Bright nails	2-1/2" x 0.131"	6"	12"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"

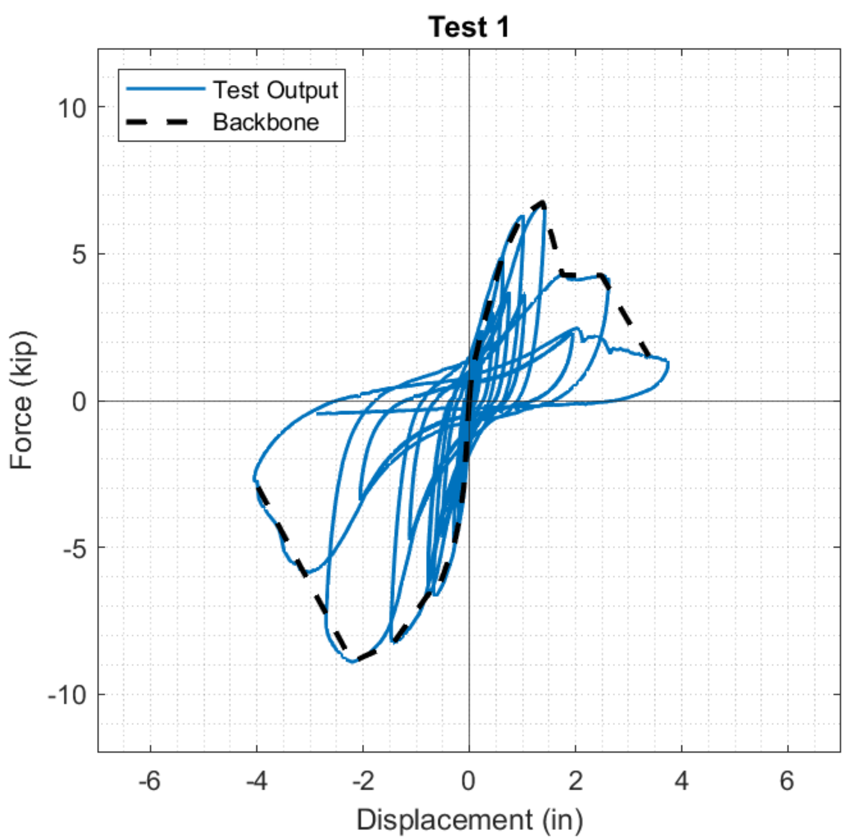
Date	Test No.	Test ID	Designation	Panel Type	Nail Type	Sheathing Nail Size	Sheathing Nail Spacing (edge)	Nail Spacing (field)	Framing Type	Stud Spacing	# of Center stud(s)	Top/bottom plate to stud fasteners
2022-08-18	22	O-131-H-6-c	O-131-H-6	7/16" OSB	Bright nails	2-1/2" x 0.131"	6"	12"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-08-23	23	O-131-H-12-b	O-131-H-1	7/16" OSB	Bright nails	2-1/2" x 0.131"	12"	12"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-09-15	24	O-131-H-3-c	O-131-H-3	7/16" OSB	Bright nails	2-1/2" x 0.131"	3	12"	Doug Fir	16" o.c. (nominal)	2	3-3"x0.131"
2022-10-25	25	O-131-H-3-d	O-131-H-3	7/16" OSB	Bright nails	2-1/2" x 0.131"	3	12"	Doug Fir	16" o.c. (nominal)	2	3-3"x0.131"
2022-11-18	26	0-0-H-0	0-0-H-0	n/a	n/a	n/a	n/a	n/a	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-11-18	27	O-131-H-12-c	O-131-H-1	7/16" OSB	Bright nails	2-1/2" x 0.131"	12"	12"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-11-23	28	O-131-B-6-a	O-131-B-6	7/16" OSB	Bright nails	2-1/2" x 0.131"	6"	12"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-12-01	29	T-131-D-1-a	T-131-D-1	T1-11	Bright nails	2-1/2" x 0.131"	6"-1 row, two seams	12"	Hem Fir No. 2	24" o.c. (nominal)	1	3-3"x0.131"
2022-12-02	30	O-131-B-6-b	O-131-B-6	7/16" OSB	Bright nails	2-1/2" x 0.131"	6"	12"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-12-02	31	O-131-H-6-d	O-131-H-6	7/16" OSB	Bright nails	2-1/2" x 0.131"	6"	12"	Doug Fir	16" o.c. (nominal)	1	3-3"x0.131"
2022-12-14	32	T-131-D-2-a	T-131-D-2	T1-11	Bright nails	2-1/2" x 0.131"	6"-2 rows, two seams	12"	Doug Fir	24" o.c. (nominal)	1	3-3"x0.131"
2023-02-21	33	T-131-D-1-b	T-131-D-1	T1-11	Bright nails	2-1/2" x 0.131"	6"-1 row, two seams	12"	Hem Fir No. 2	24" o.c. (nominal)	1	3-3"x0.131"
2023-03-06	34	T-131-D-1-c	T-131-D-1	T1-11	Bright nails	2-1/2" x 0.131"	6"-1 row, two seams	12"	Hem Fir No. 2	24" o.c. (nominal)	1	3-3"x0.131"
2023-03-21	35	O-14g-H-4-a	O-14g-H-4	7/16" OSB	Staples	14 ga	4"	8"	Doug Fir No. 2	16" o.c. (nominal)	1	3-3"x0.131"
2023-03-28	36	O-14g-H-4-b	O-14g-H-4	7/16" OSB	Staples	14 ga	4"	8"	Doug Fir No. 2	16" o.c. (nominal)	1	3-3"x0.131"
2023-03-29	37	O-14g-H-4-c	O-14g-H-4	7/16" OSB	Staples	14 ga	4"	8"	Doug Fir No. 2	16" o.c. (nominal)	1	3-3"x0.131"

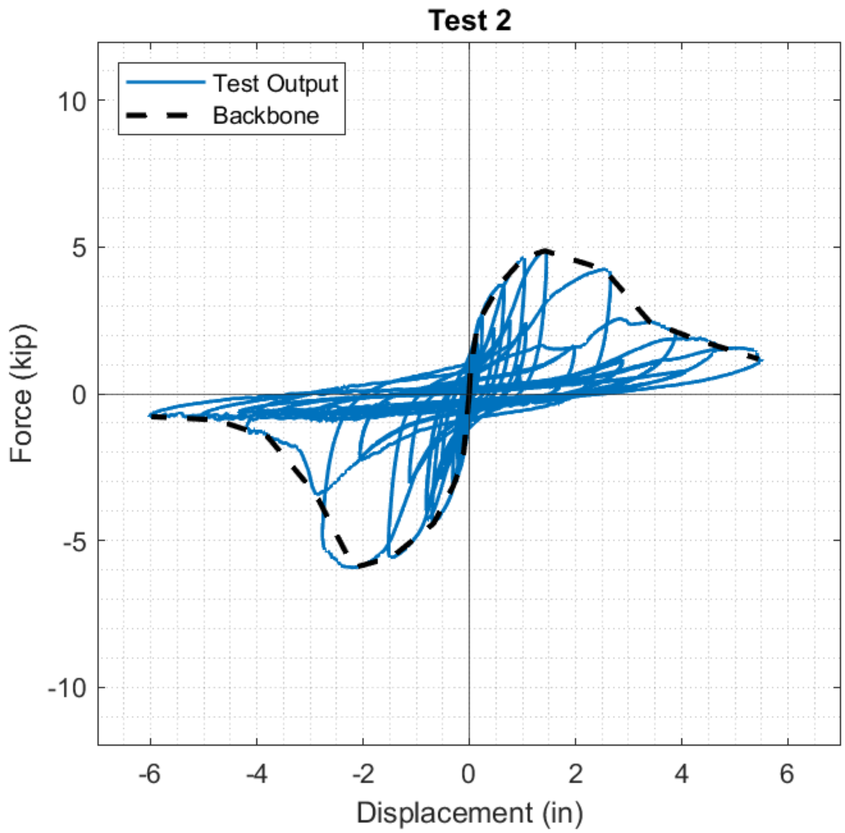
Date	Test No.	Double end stud fasteners	Hold-downs	Code-Compliant	Peak Strength (lb)	Shear Strength, vpeak (plf)	Cyclic Ductility Ratio, D	Stiffness	Build Notes	Failure Mechanism
2022-05-27	1	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	8910.4	1114	2.9	18.7	sheathing nails in south panel renailed (missed stud), close to edge of OSB panel	OSB (south panel) ripped at center stud, top plate detached from north studs
2022-06-02	2	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	5927.5	741	13.6	25.3	warping in 2x6s, used big wrench during construction (maybe prestressed studs?)	bending in nails in center stud, cracking in interior studs
2022-06-10	3	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	2893.9	362	17.9	12.0	some missed nails (renailed) in center stud	nail bending and pull out
2022-06-16	4	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	5715.1	714	14.2	16.0		nails pulled at end studs and top and bottom plates, OSB more firmly attached to interior studs
2022-06-23	5	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	7124.7	891	7.0	21.2		bowing at OSB panel at center stud
2022-06-30	6	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	10121.2	1265	7.1	20.0		
2022-07-12	7	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	2839.6	355	8.6	12.6	hand driven finish nails	small nail heads don't hold OSB panels, some nails sheared, most nails bent
2022-07-12	8	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	5434.4	679	7.1	13.2	same framing and OSB, finish nails removed and replaced with 8d nails	
2022-07-14	9	3"x0.131" nails, 2 rows 8" o.c.	none	No	2782.8	348	8.8	9.3	many missed nails along bottom plate (~12 o.c.)	nail withdrawal along bottom plate, and at top of north end studs
2022-07-15	10	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	3464.3	433	13.1	10.5	more studs warped	pull through of nails along top of panels, edge of south panel
2022-07-19	11	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	4901.8	613	9.6	14.3		staples broke and pulled out at bottom of studs
2022-07-20	12	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	5315.1	664	11.9	12.9		
2022-07-20	13	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	5718.5	715	10.2	12.9		
2022-07-22	14	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	5759.8	720	12.1	13.7		
2022-07-22	15	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	9787.7	1223	5.8	15.2		
2022-08-08	16	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	4697.2	587	9.5	15.2	top plate was very twisted	nail withdrawal on north panel at top plate (twisting)
2022-08-09	17	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	5641.0	705	9.6	14.4		nail pull through at top of OSB panels
2022-08-10	18	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	4959.0	620	12.1	12.9		
2022-08-11	19	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	8788	1099	5.5	27.3	several doubled staples	staples broke and pulled out at bottom of studs
2022-08-16	20	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	7814	977	6.0	21.5	one direction of nails missing in south double end studs	
2022-08-18	21	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	5764	721	14.0	14.4		

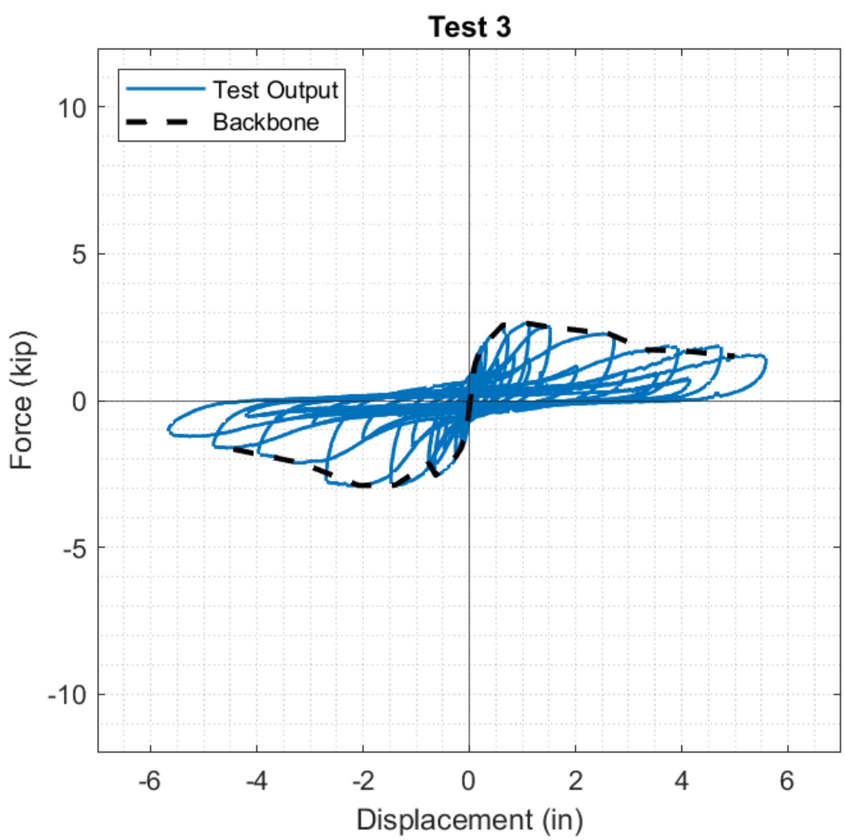
Date	Test No.	Double end stud fasteners	Hold-downs	Code-Compliant	Peak Strength (lb)	Shear Strength, vpeak (plf)	Cyclic Ductility Ratio, D	Stiffness	Build Notes	Failure Mechanism
2022-08-18	22	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	5071	634	10.8	15.9		
2022-08-23	23	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	3475.0	434	14.5	17.4		
2022-09-15	24	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	6717	840			center studs not nailed together	
2022-10-25	25	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	9245	1156	6.3	18.3		
2022-11-18	26	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	315	39	0.0	0.0		
2022-11-18	27	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	3500	438	14.7	12.7		
2022-11-23	28	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	4405	551	2.8	7.9	No Nails along sill plate	
2022-12-01	29	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	4370	546	5.2	4.5		
2022-12-02	30	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	4929	616	3.7	7.0	No Nails along sill plate	
2022-12-02	31	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	5035	629	8.9	15.8	Contractor Built	
2022-12-14	32	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	4882	610	7.8	10.6		
2023-02-21	33	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	3305	413	0.0	0.0		
2023-03-06	34	3"x0.131" nails, 2 rows 8" o.c.	HTT5	No	3181	398	21.3	10.9		
2023-03-21	35	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	6965	871	3.4	9.8		
2023-03-28	36	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	7087	886	2.1	4.6		
2023-03-29	37	3"x0.131" nails, 2 rows 8" o.c.	HTT5	Yes	7246	906	4.1	14.4		

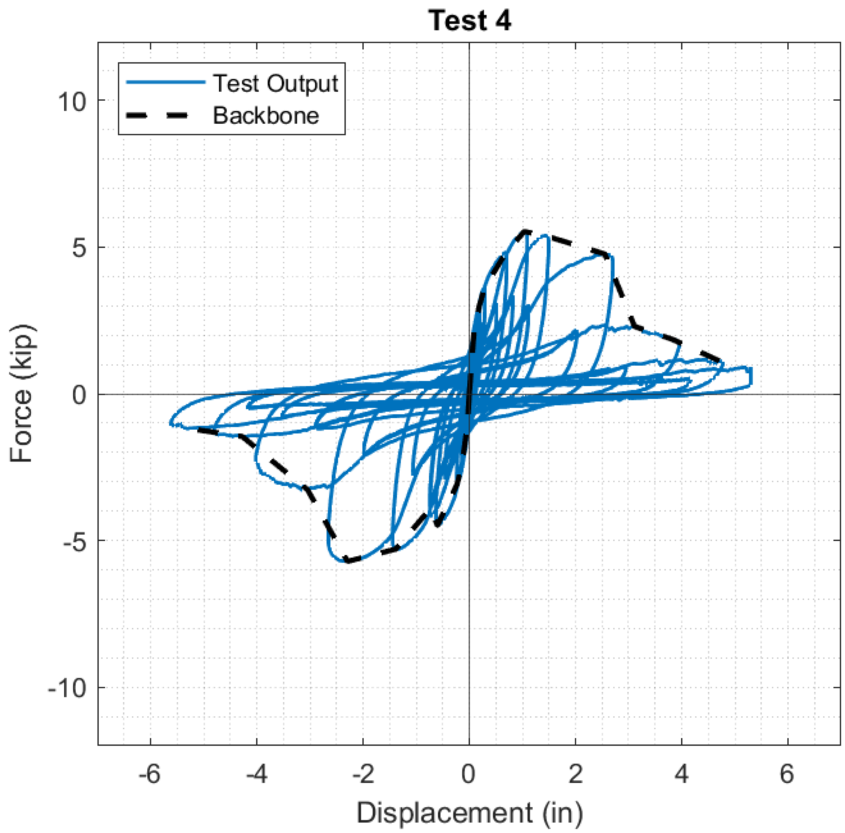
Appendix C: Plots of Test Data

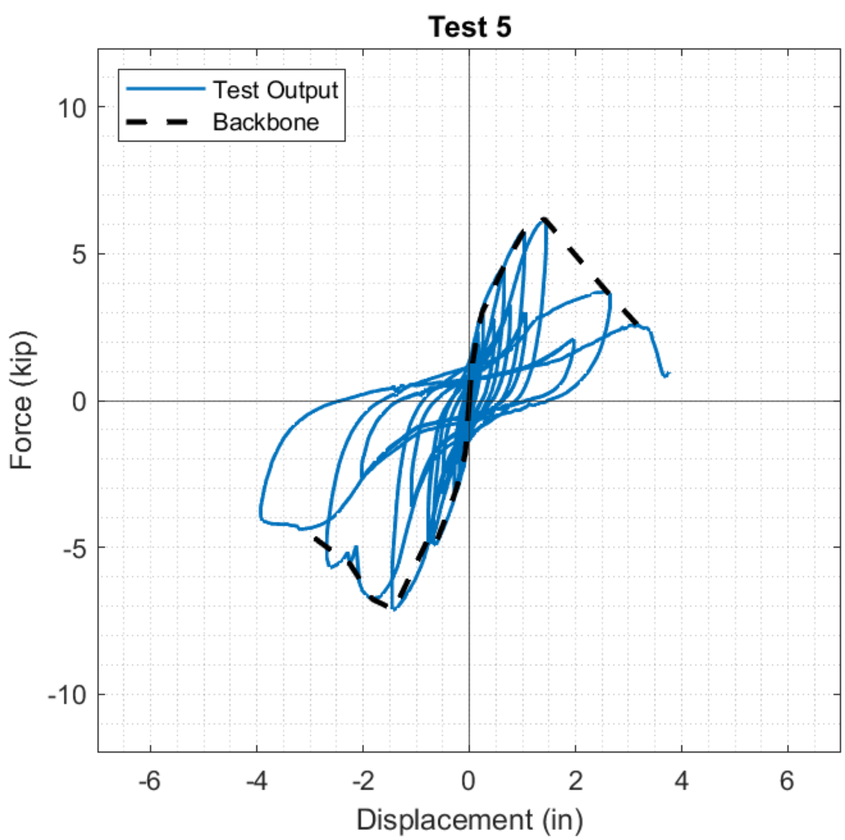

Test 37
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6


Test 1
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6

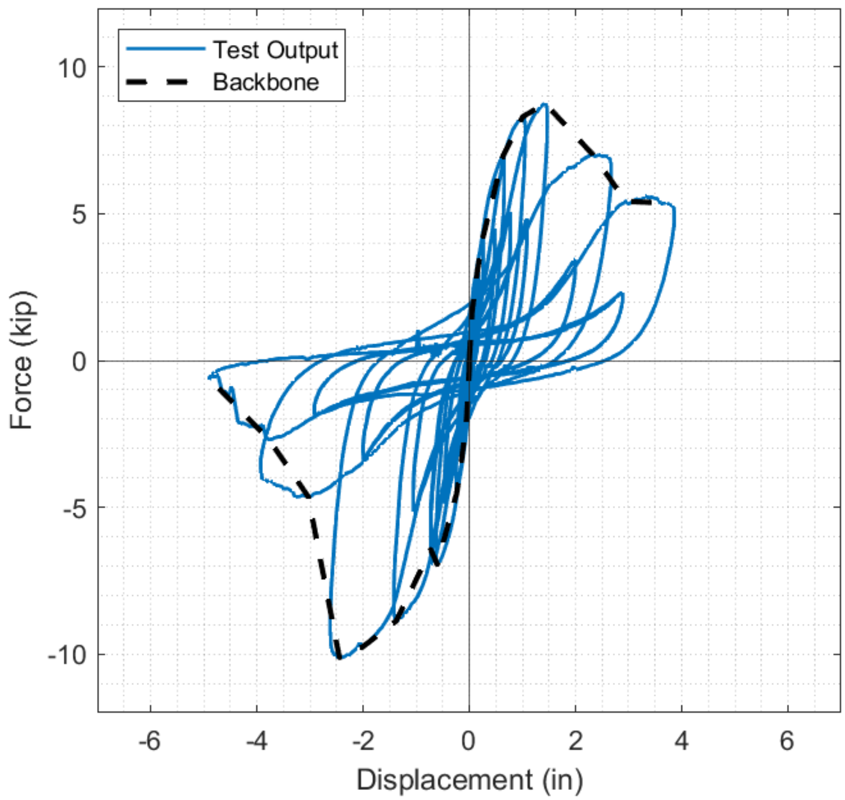

Test 2
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)


Test 3
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)


Test 4
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6

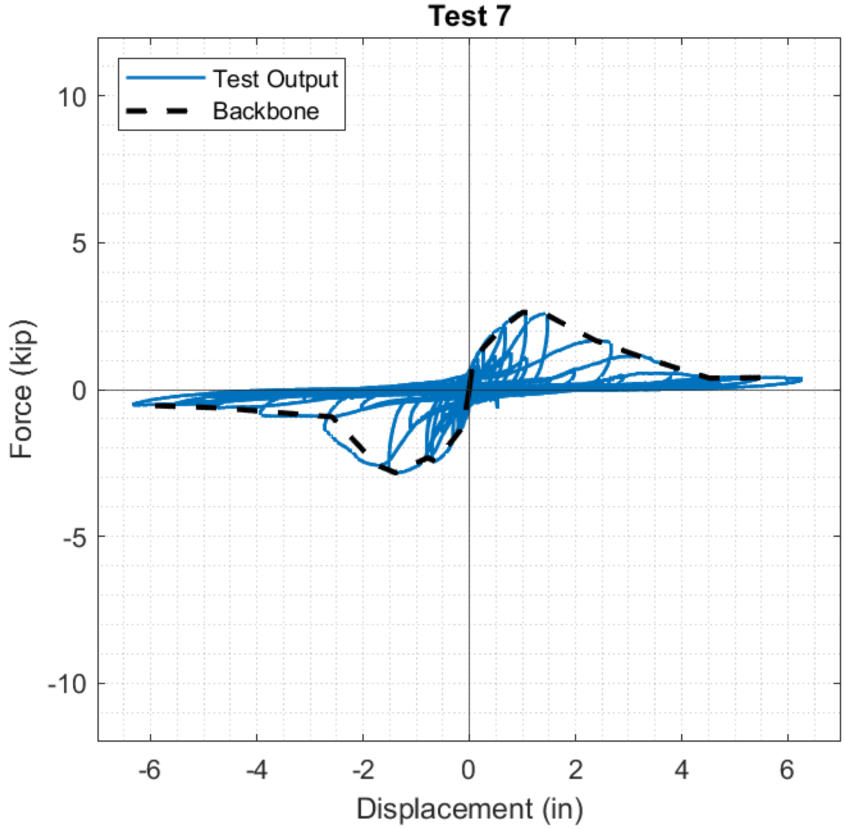

Test 5
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6

Test 6

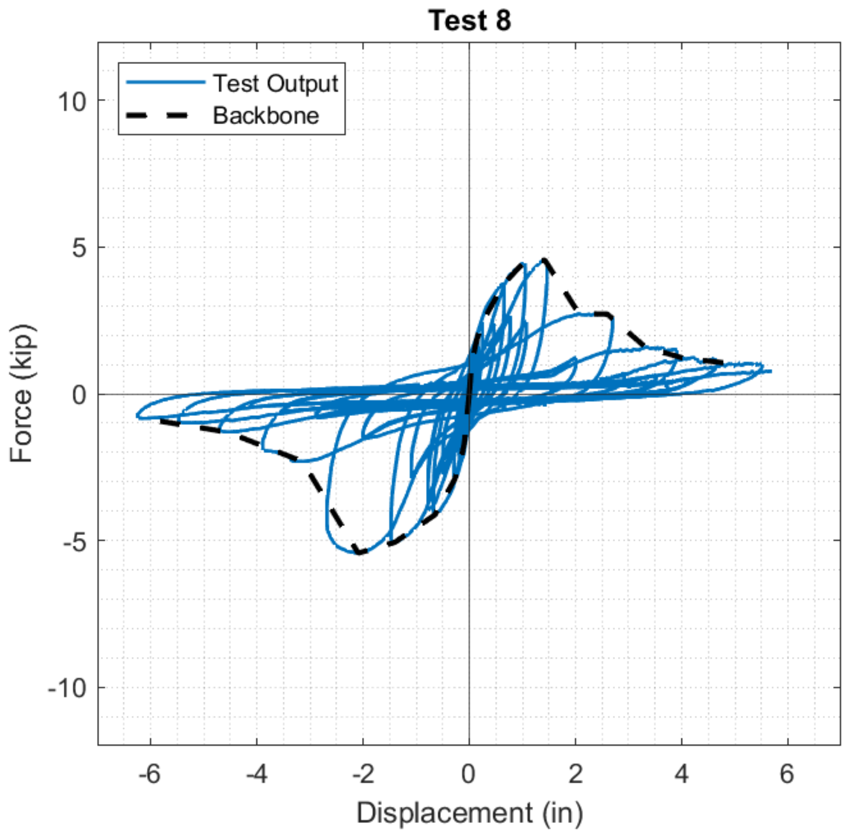


Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)

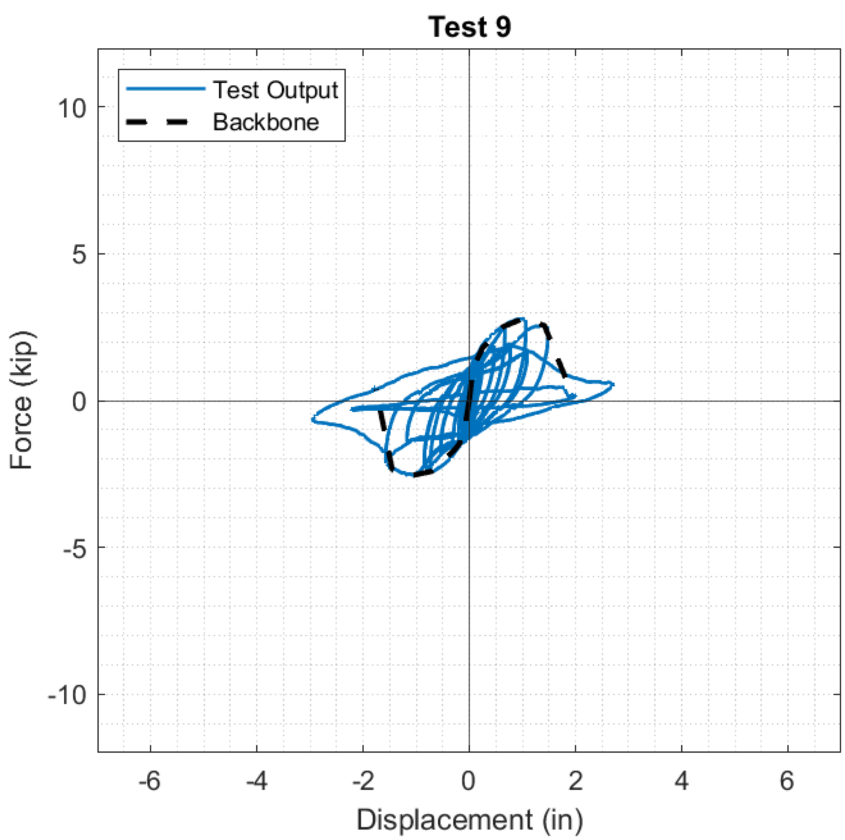
Test 7



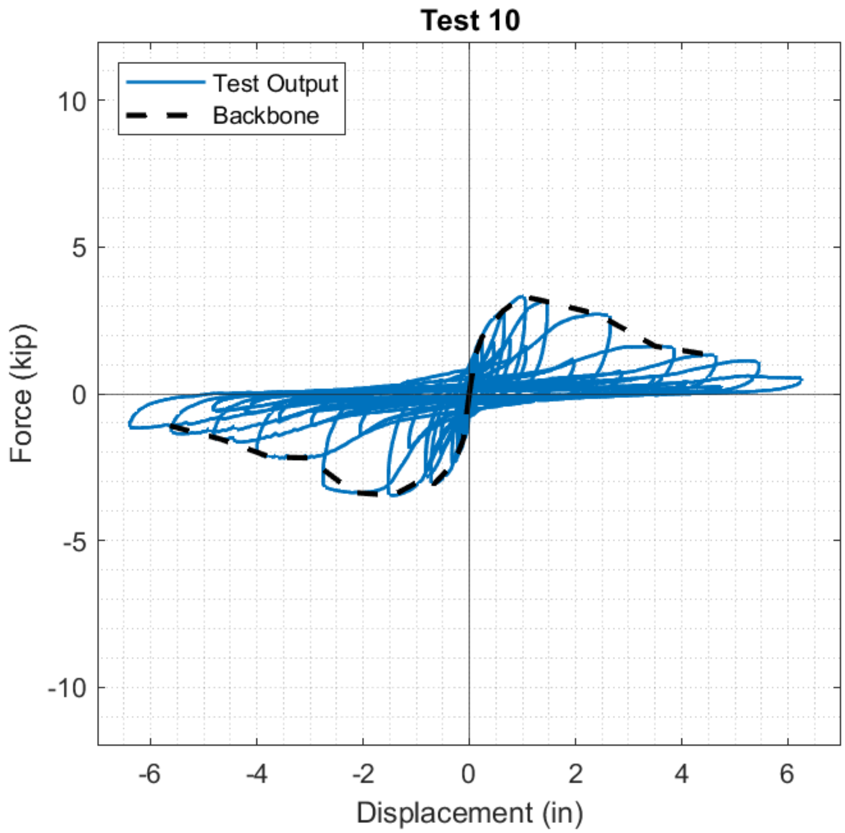
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)

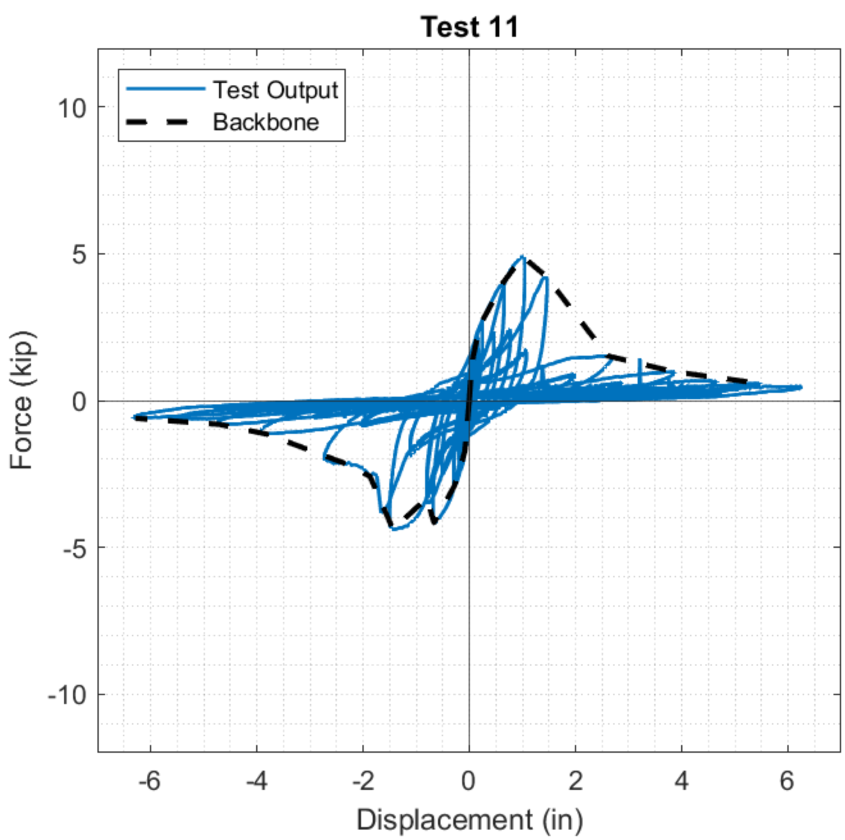


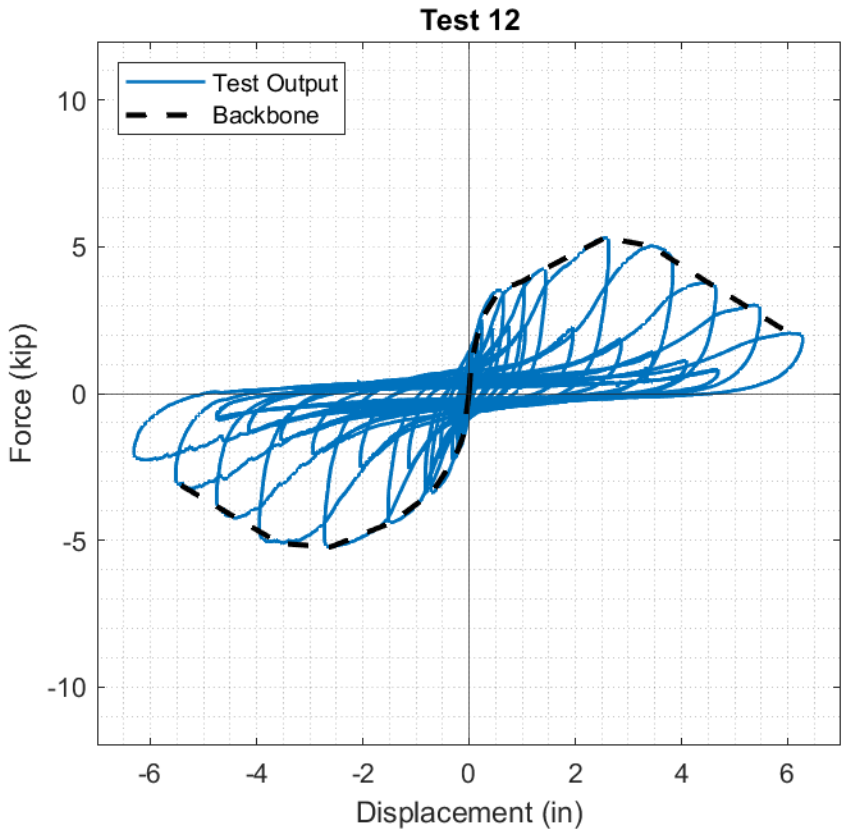
Test 8
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)

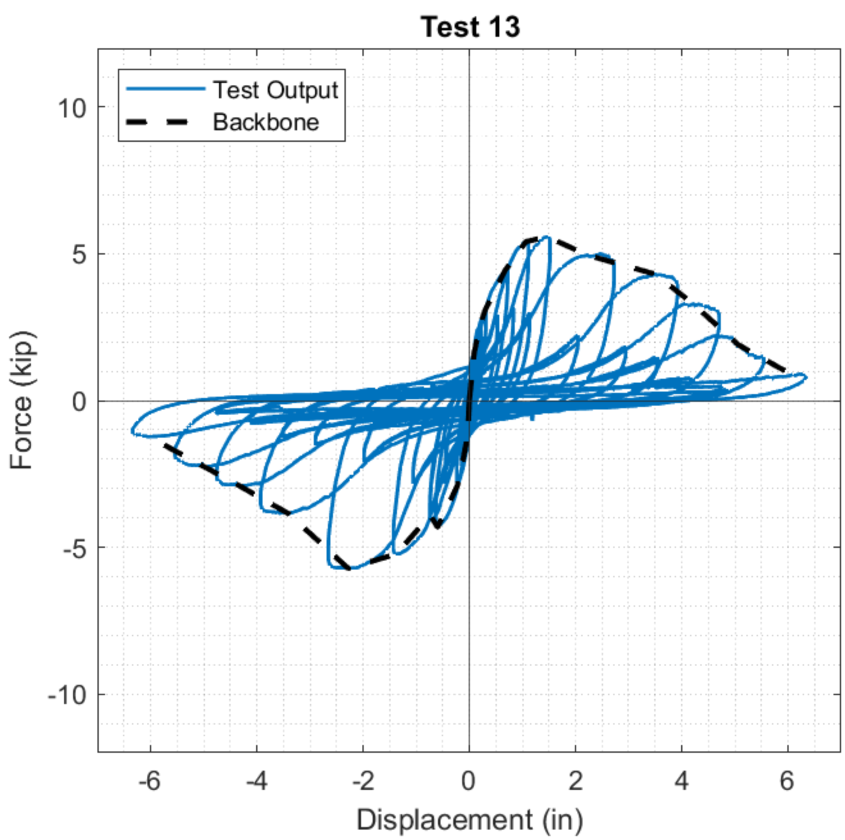


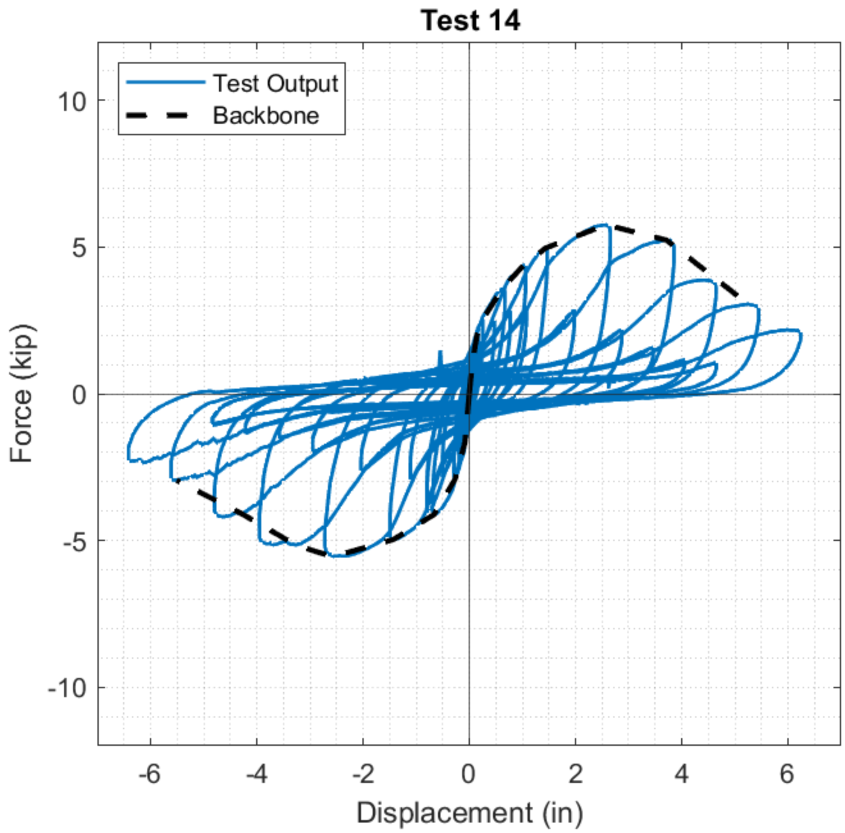
Test 9
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)

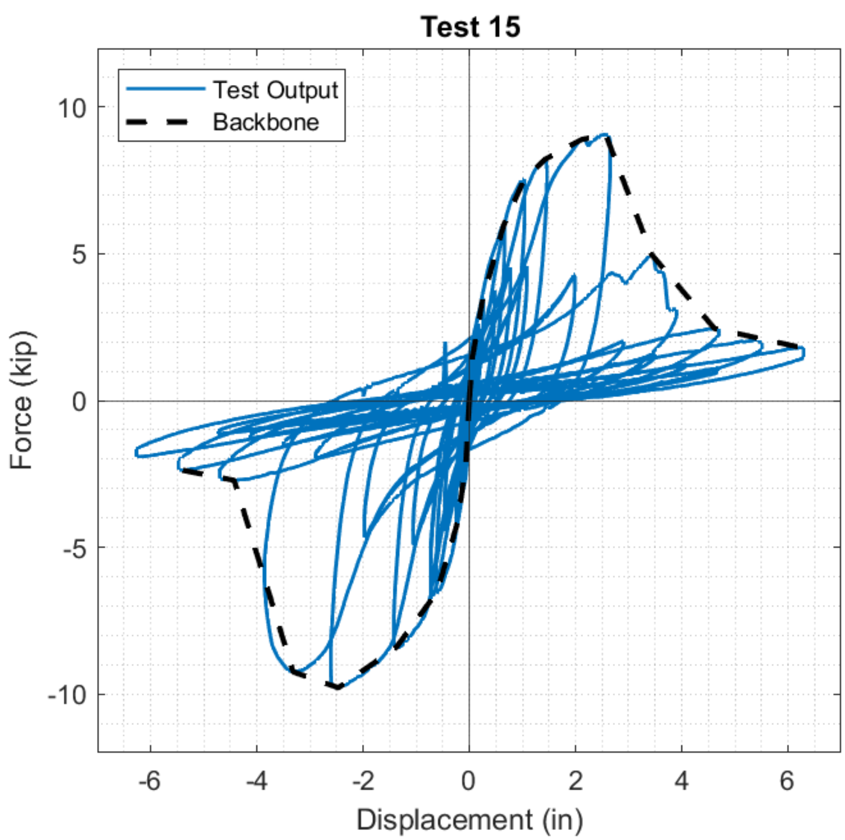

Test 10
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6

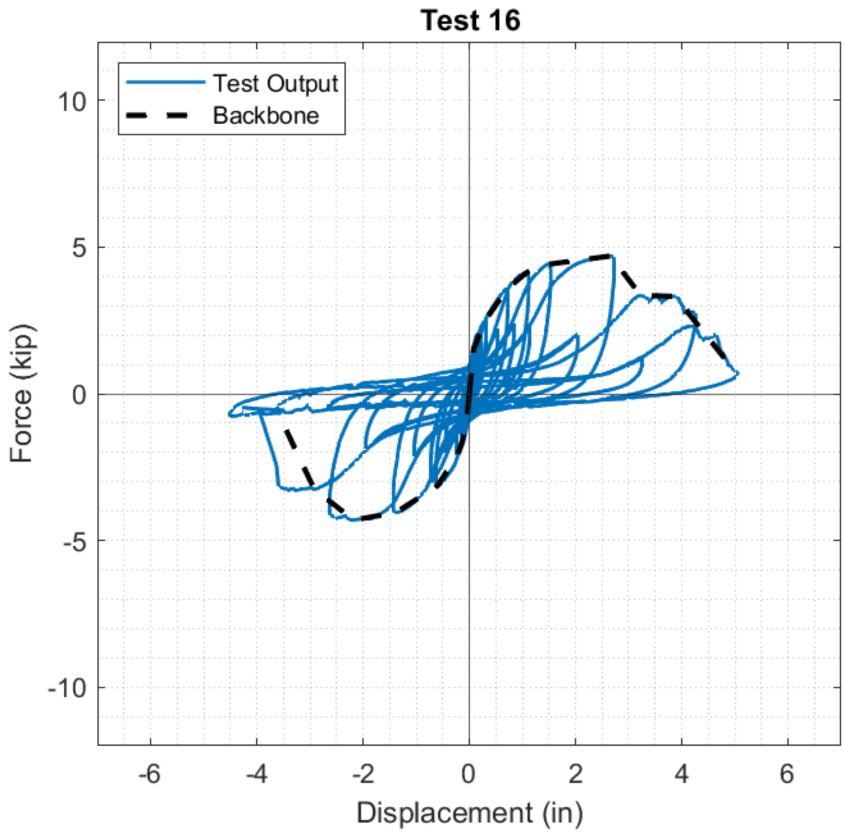

Test 11
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6

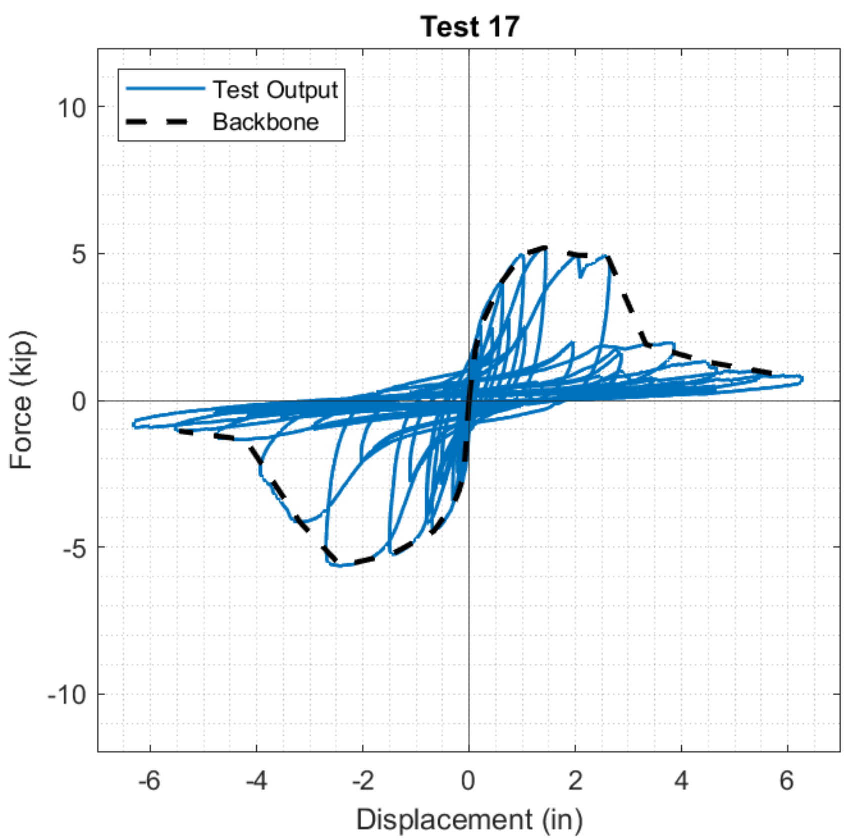

Test 12
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6


Test 13
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6

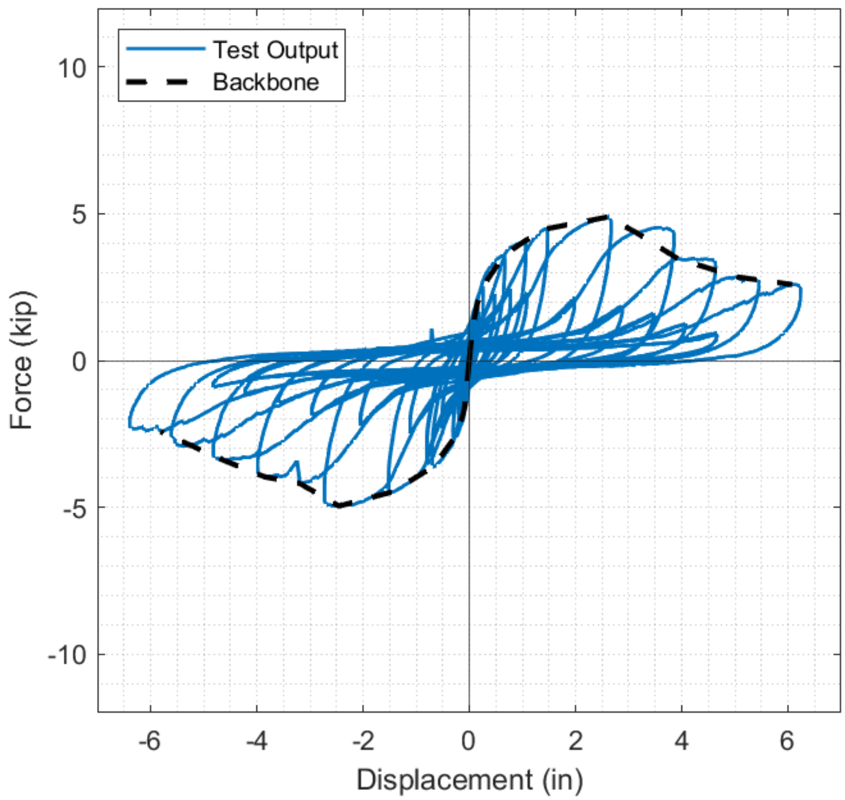

Test 14
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)


Test 15
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)

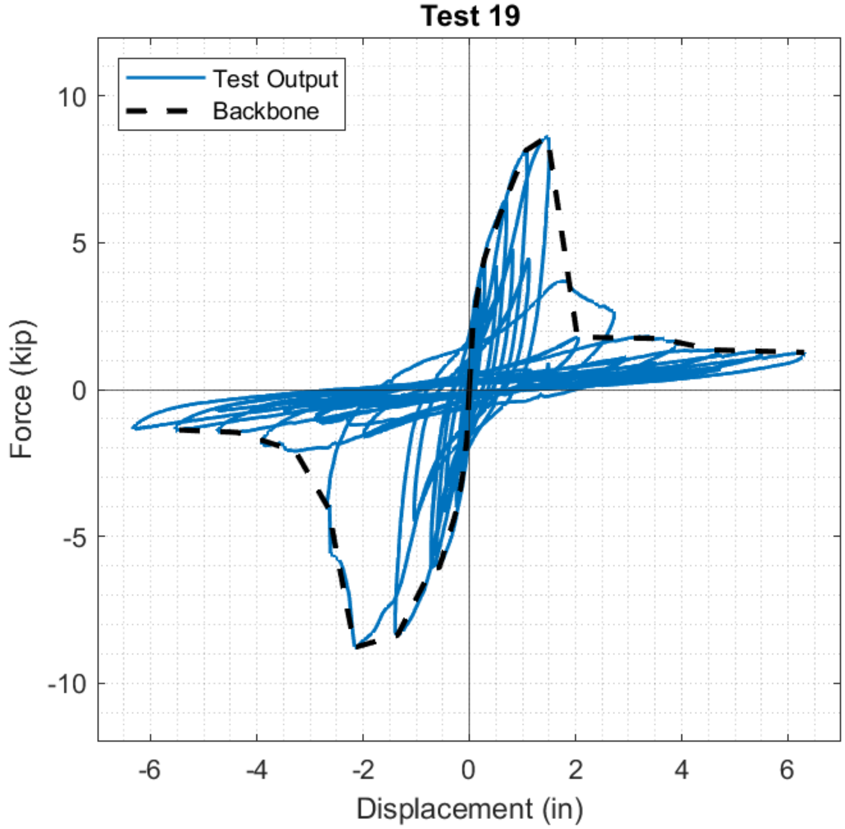

Test 16
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6

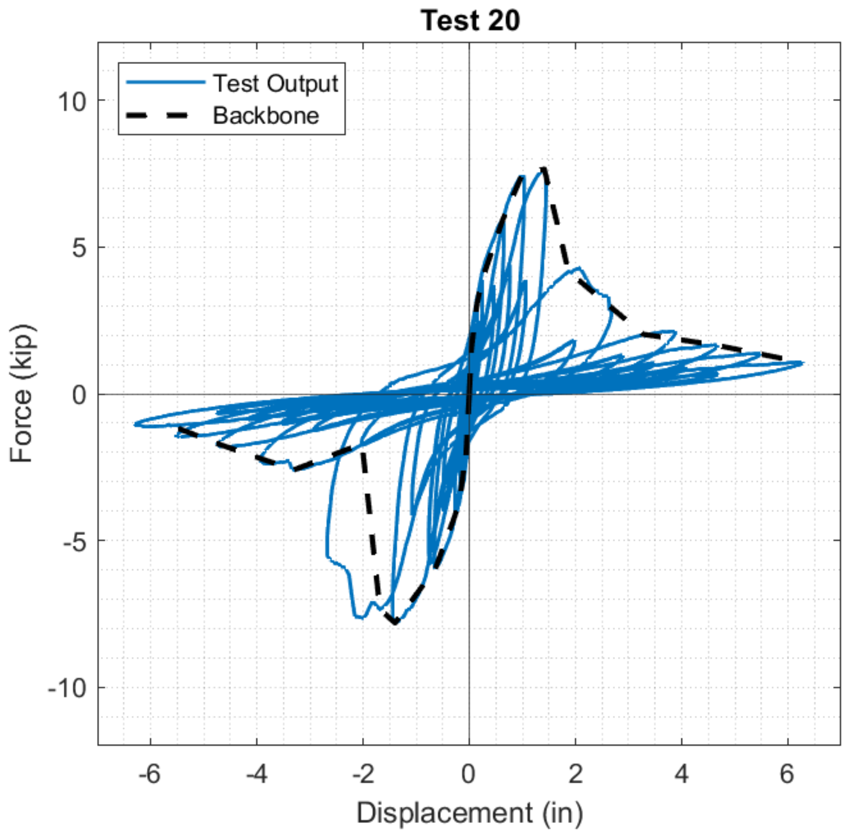

Test 17
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6

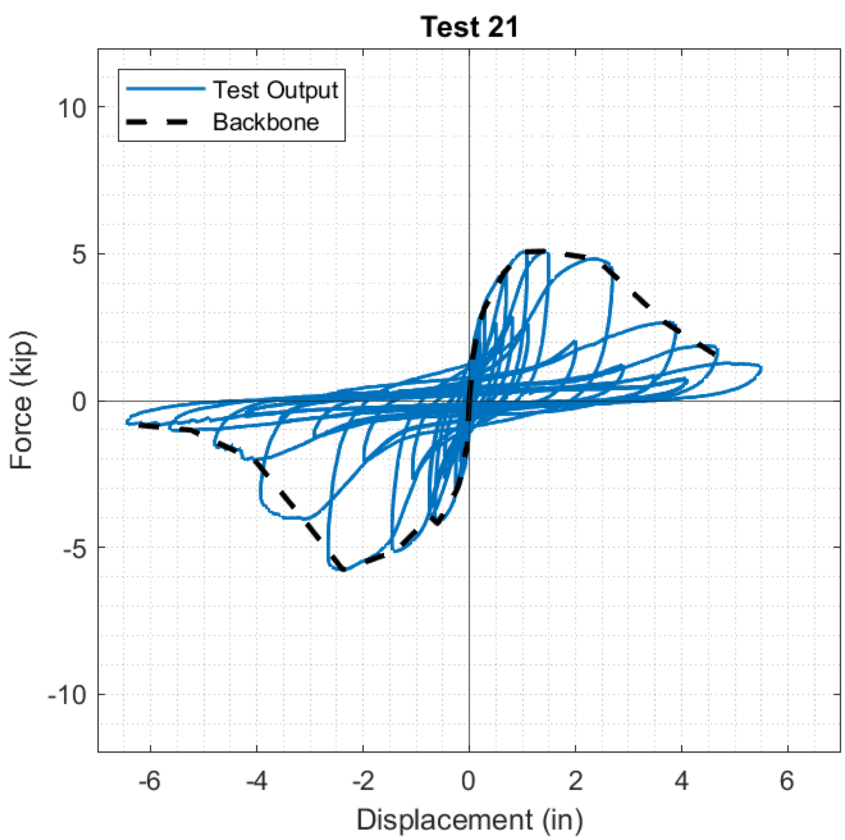
Test 18

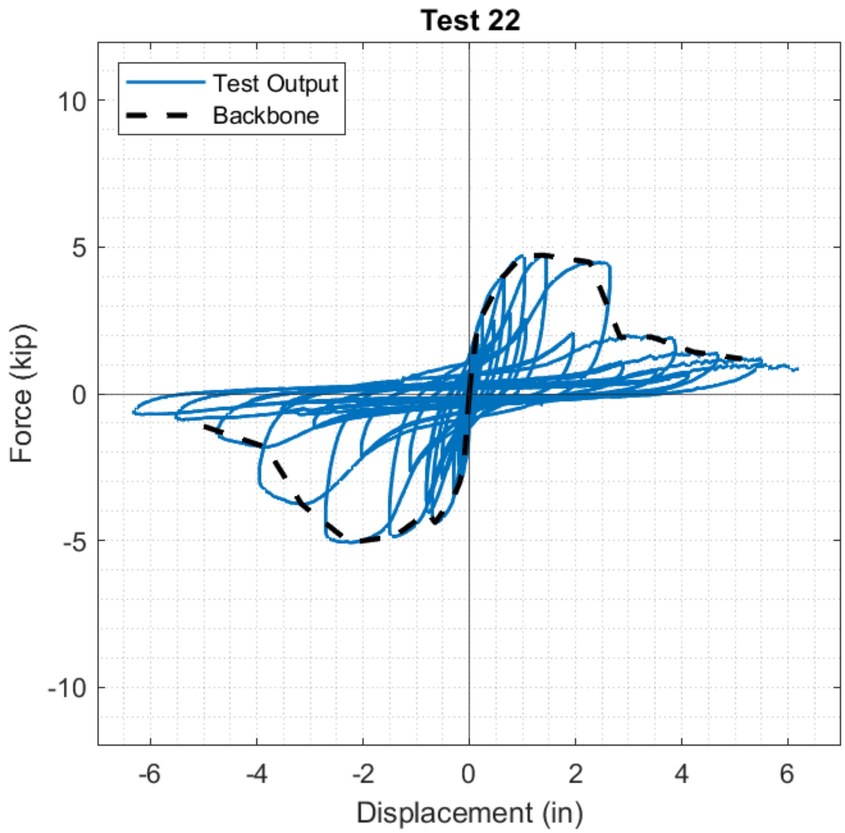


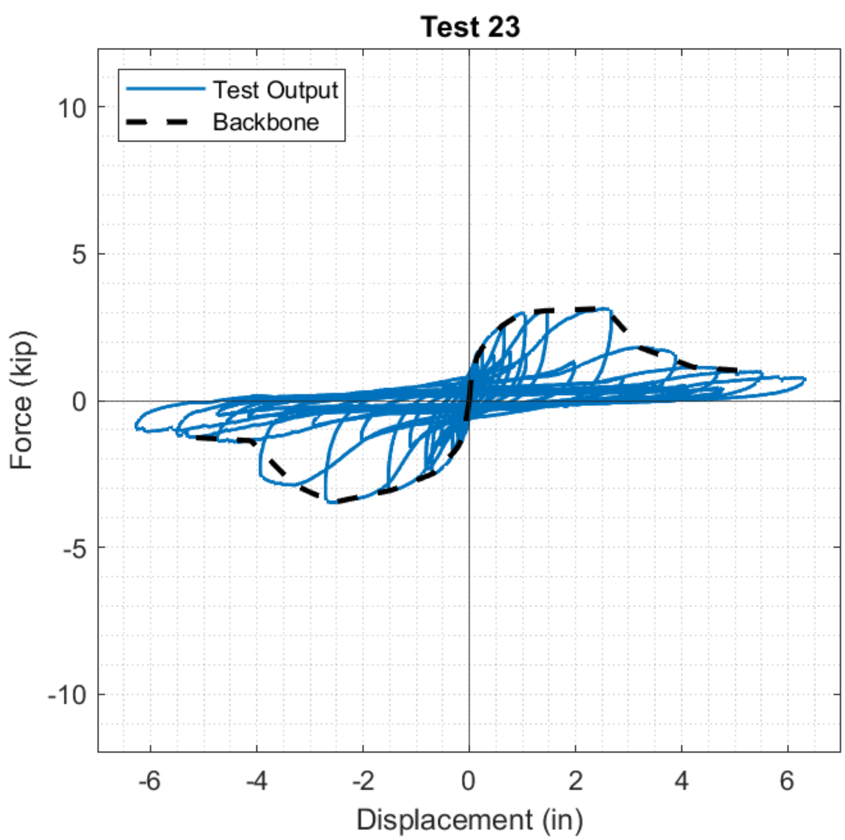
Test 19

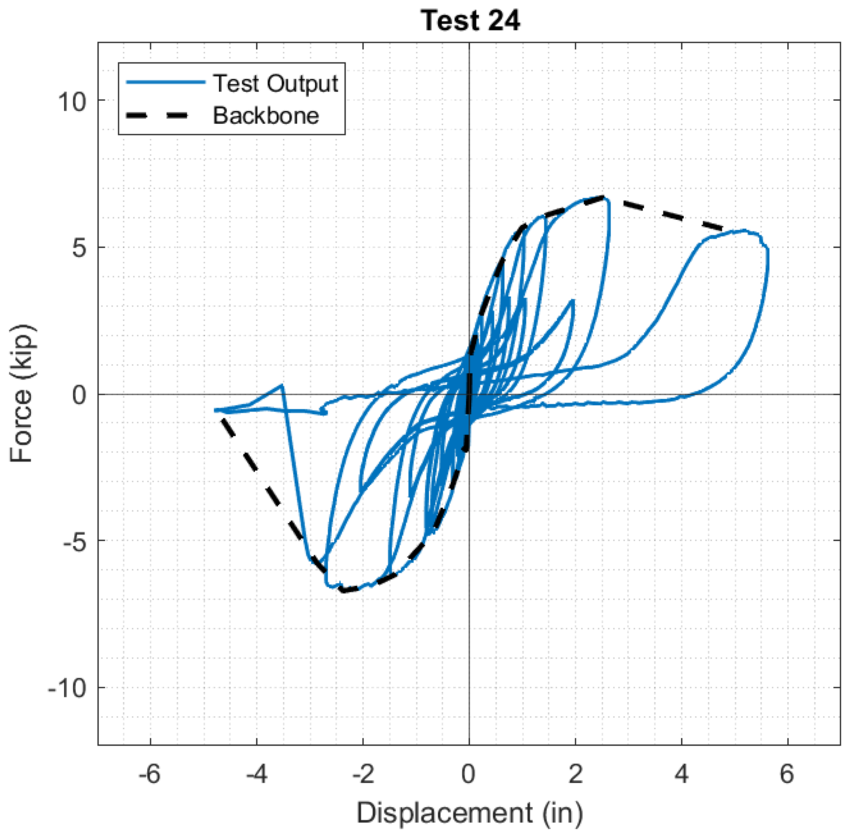


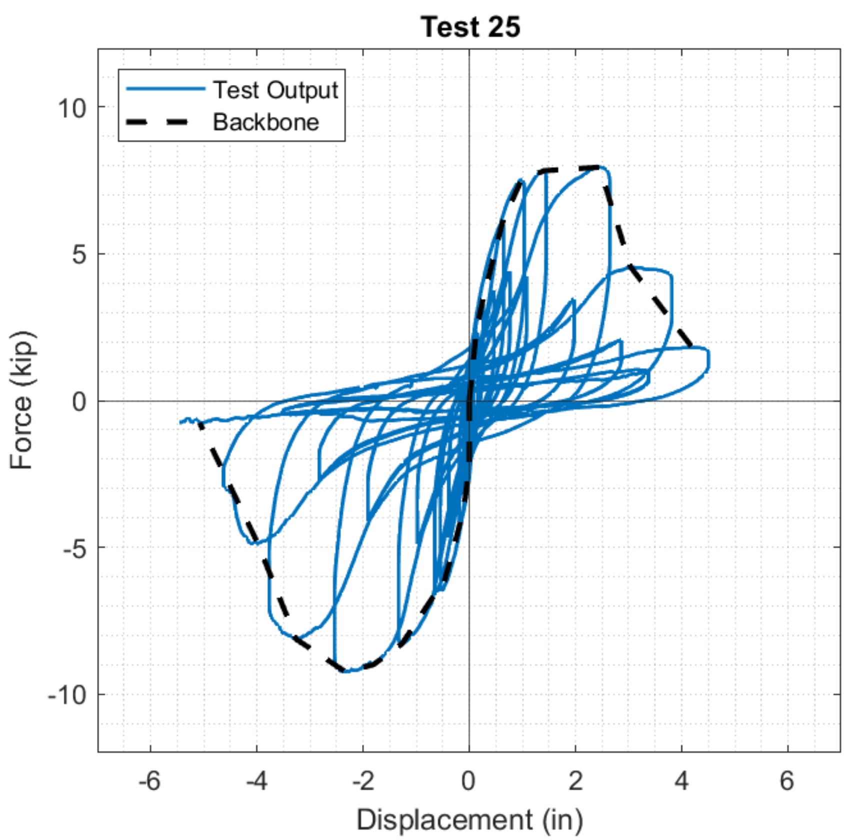

Test 20
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)

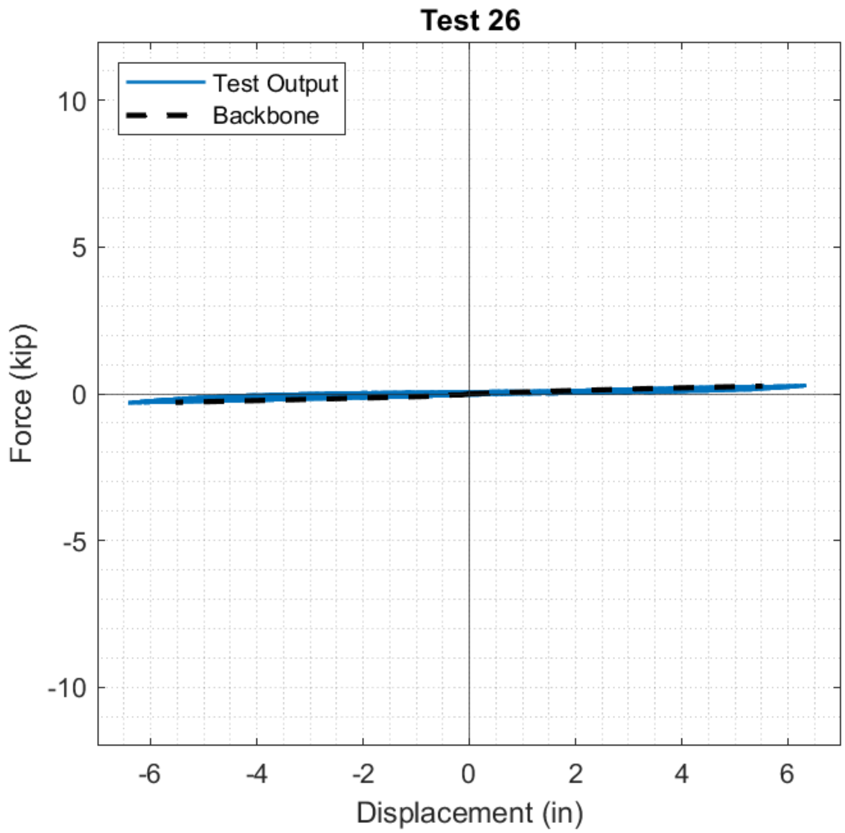

Test 21
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)


Test 22
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)

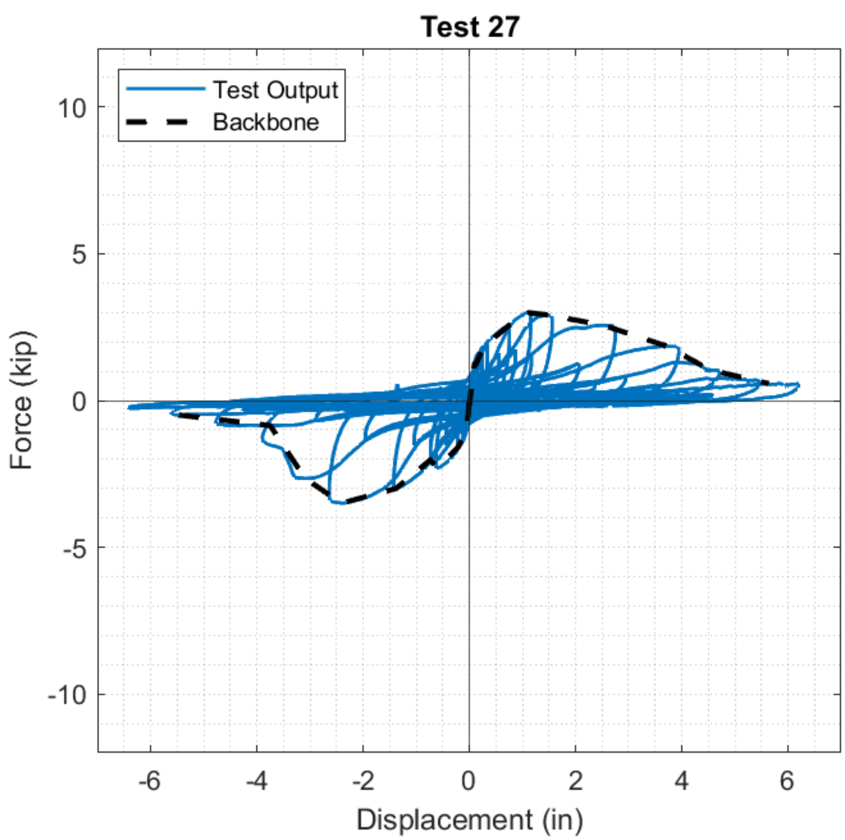

Test 23
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)


Test 24
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)

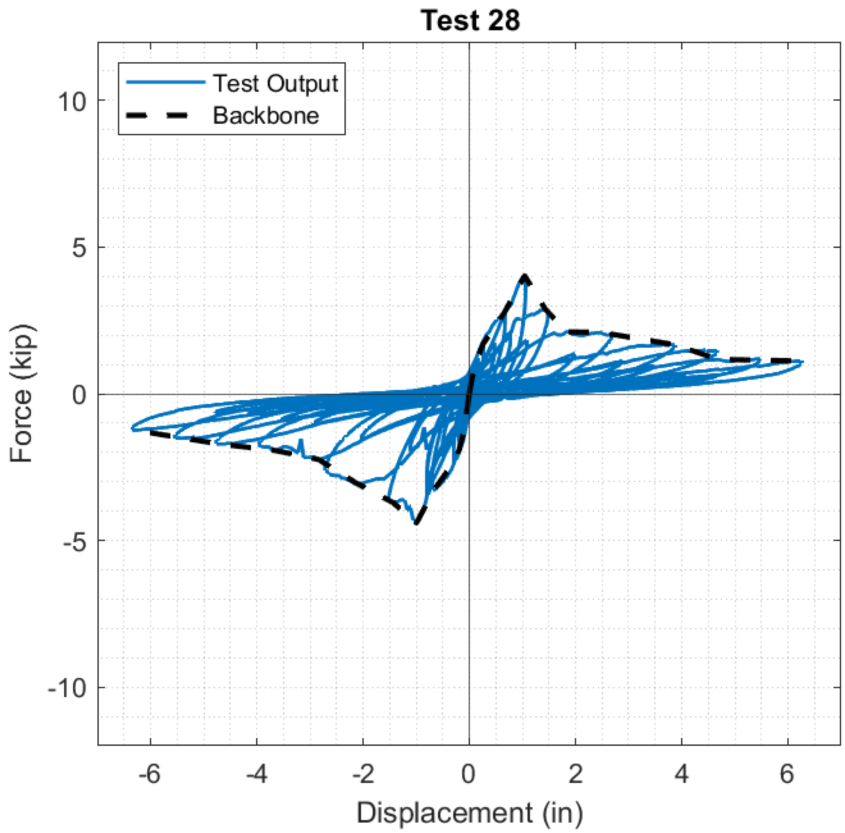

Test 25
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)



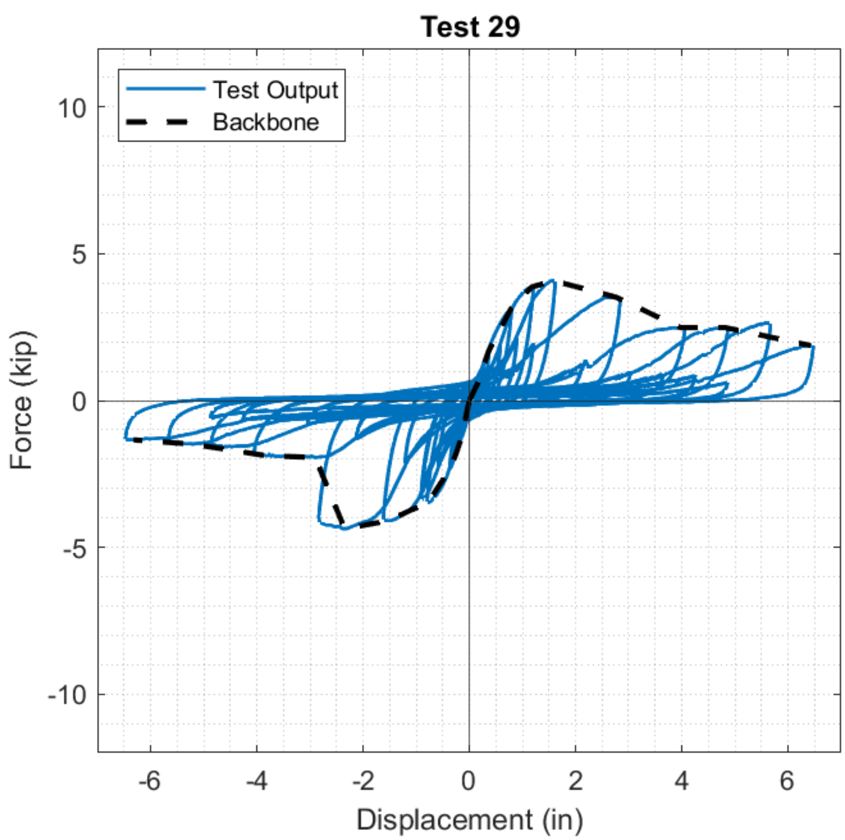
Test 26
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6



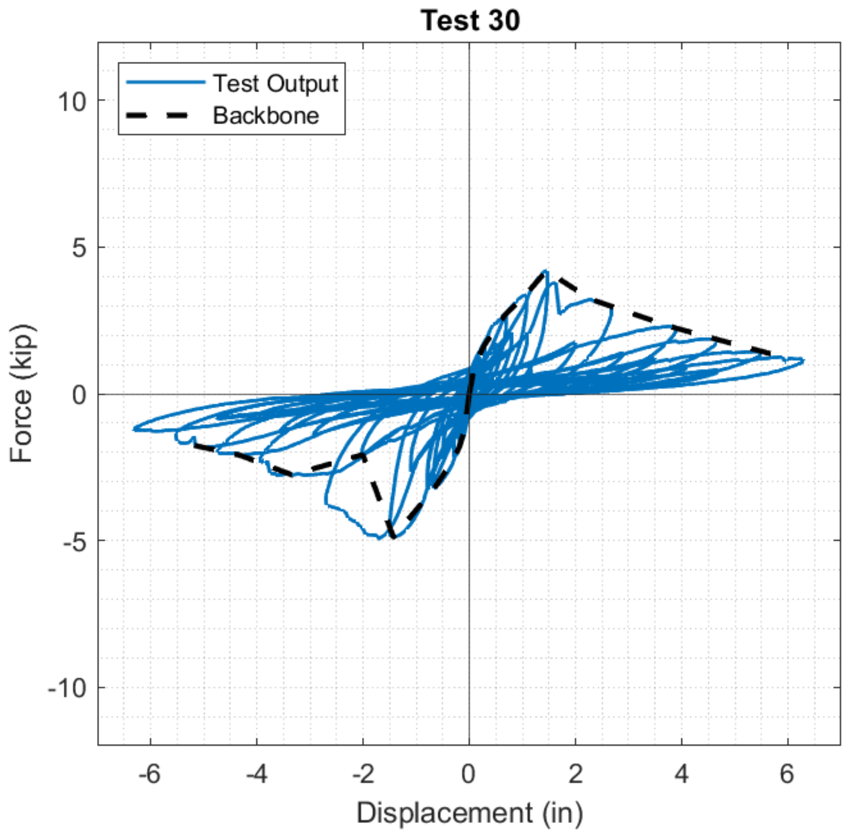
Test 27
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6

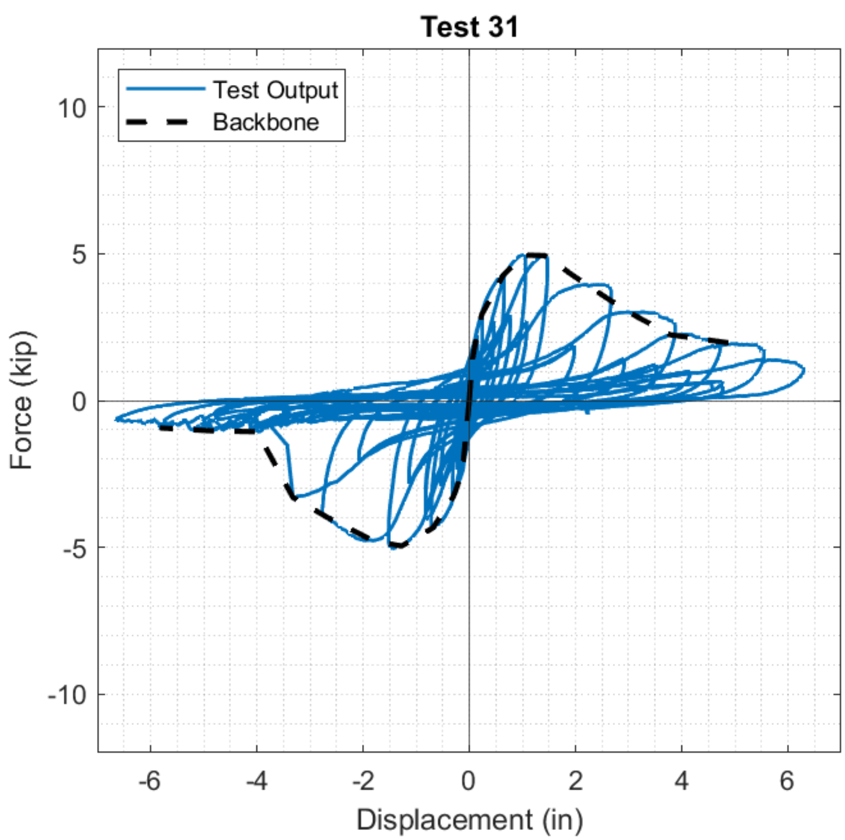


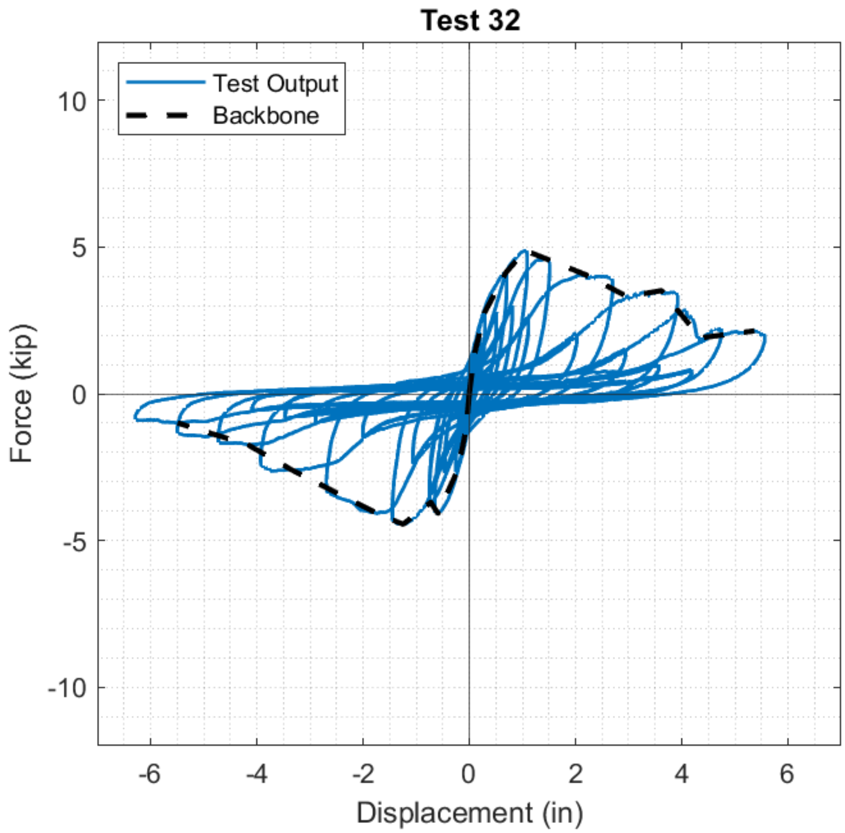
Test 28
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)

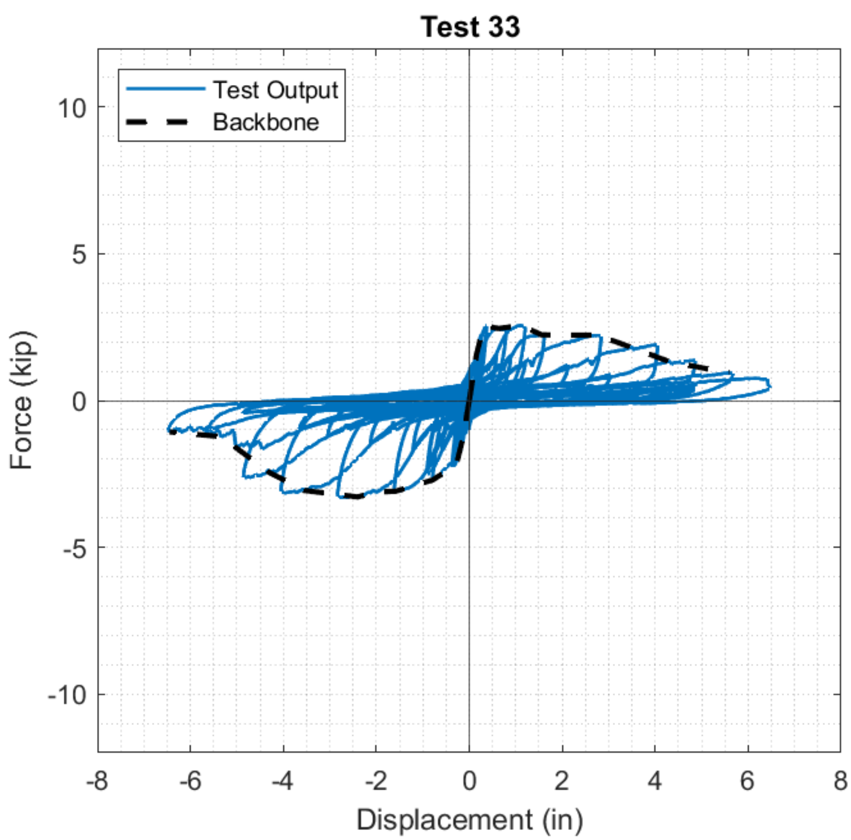


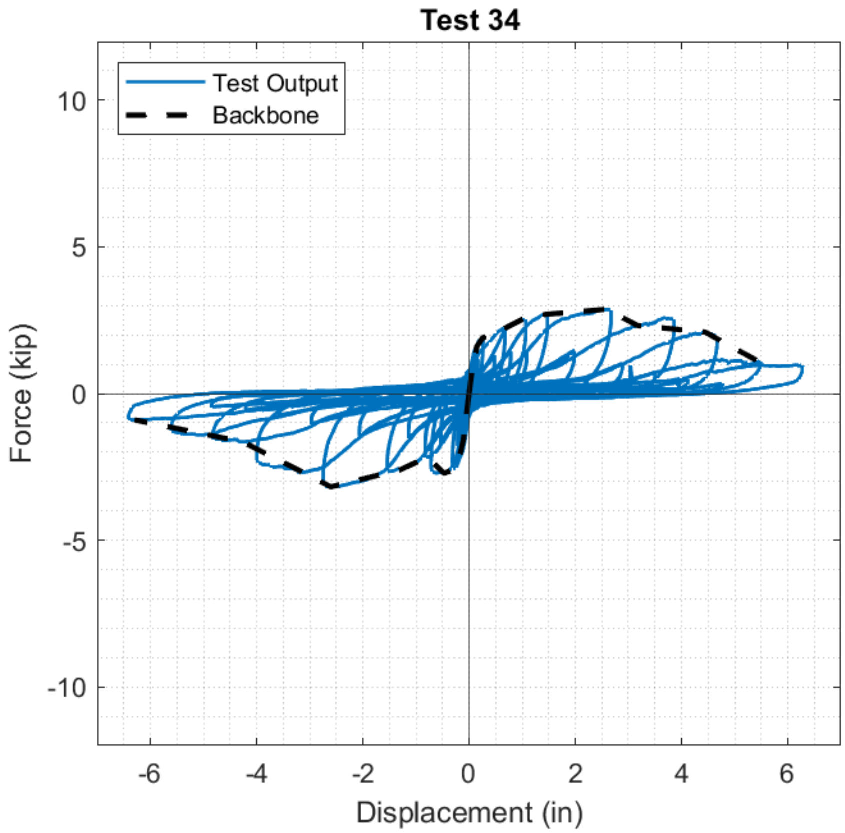
Test 29
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)

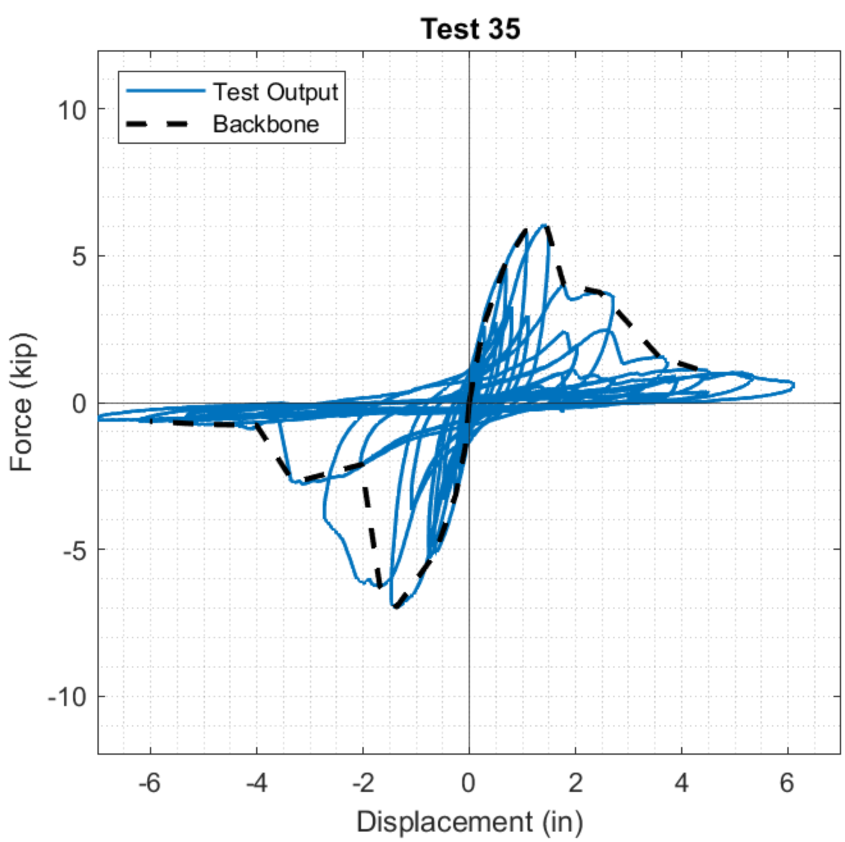

Test 30
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6

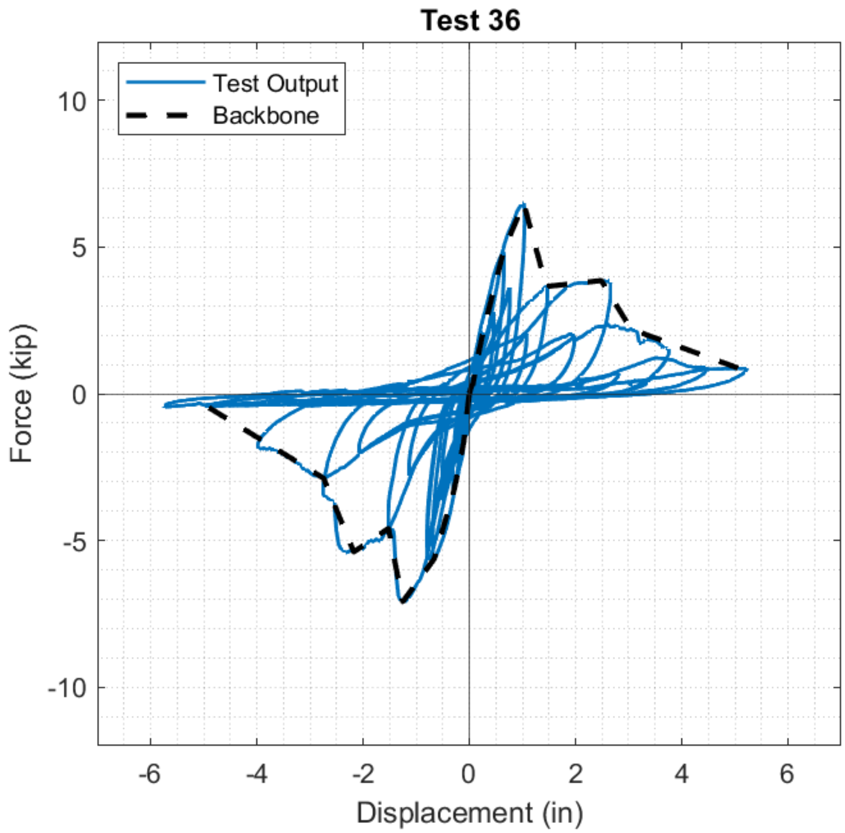

Test 31
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6


Test 32
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6


Test 33
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-8
-6
-4
-2
0
2
4
6
8


Test 34
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6

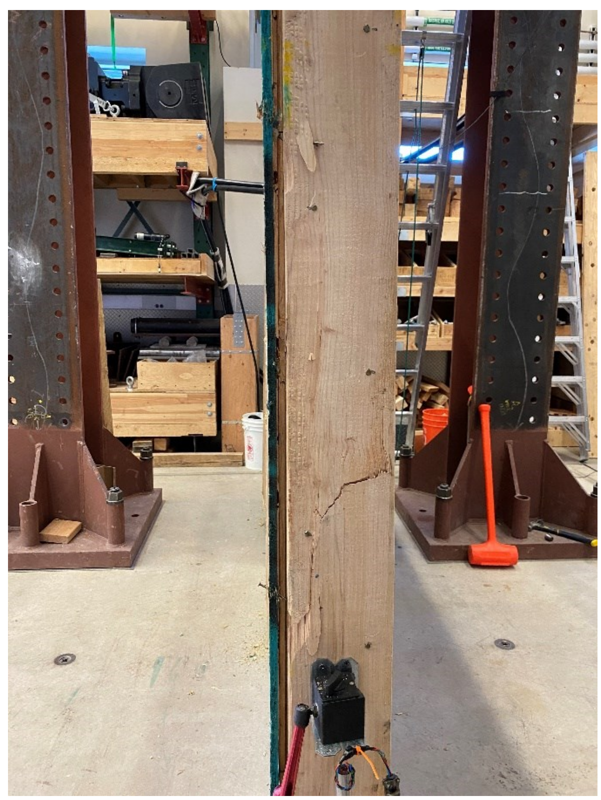

Test 35
Test Output
Backbone
Force (kip)
Displacement (in)
10
5
0
-5
-10
-6
-4
-2
0
2
4
6



Test 36
Test Output
Backbone
10
5
0
-5
-10
Force (kip)
-6
-4
-2
0
2
4
6
Displacement (in)

Appendix D: Photos of Experimental Testing


Norbord
APA
RATED SHEATHING
24/16
EXPOSURE 1
THICKNESS 0.418 IN
424




SIMPSON
Strong-Tie



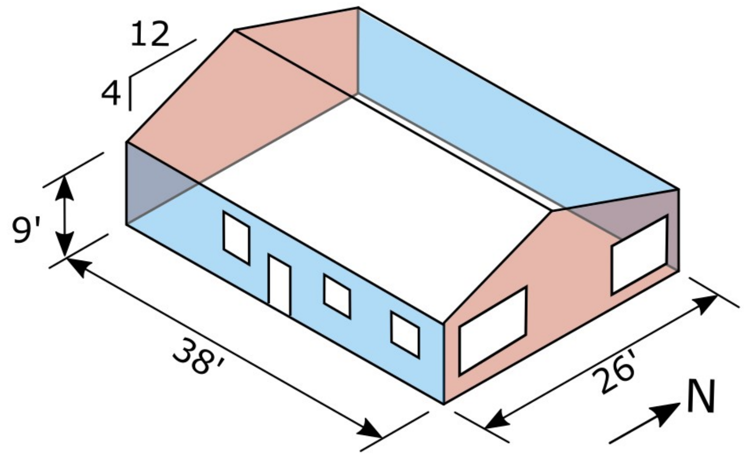

Appendix E: Design Calculations of Archetype Houses

One-Story Structure located in Anchorage, AK. Risk Category II, Soil type C, Wind Exposure C. Calculations using ASCE 7-22.

This house is similar to FEMA P2139 SFD1 (two story singly family). FEMA house is 32'x48':

Table 3-5 Short-Period Wood Light-Frame Building Archetype Configurations and Seismic Design Criteria

Archetype ID	Configuration: No. of Stories	Configuration: Wall Aspect Ratio	Seismic Design Criteria: Seismic Code	Design Period (1) $T = C_u T_a$ (sec)	Seismic Design Category (SDC)	MCE_R Design Parameter, S_{MS} (g)	Response Modification Coefficient (R)	Seismic Response Coefficient, C_s (g)
SFD1	1	High	ASCE 7	0.25	D	1.5	6.5	0.154



Find Ground Snow Load

$p_g := 50\ psf$ https://seaak.net/alaska-snow-loads (50 yr MRI)

Flat Roof Snow Load

$p_f = 0.7\ C_e \cdot C_t \cdot I_s \cdot p_g$ (7.3-1)

$C_e := 1.0$ Table 7.3-1 - Partially Exposed, Exposure C
$C_t := 1.1$ Table 7.3-2 - Cold Roof
$I_s := 1.0$ Table 1.5-2 - Risk Cat II

$p_f := 0.7\ C_e \cdot C_t \cdot I_s \cdot p_g = 38.5\ psf$

Sloped Roof

$C_s := 1.0$ Figure 7.4-1, 3 on 12 non-slippery roof with $C_t = 1.1$

$p_s := C_s \cdot p_f = 38.5\ psf$

Find Dead Loads

Roof:

Composition Shingles	2.0 psf		
Roof Wrap/tar paper	0.5 psf		
Second Lay Shingles	2.0 psf		
5/8" CDX Plywood	3.0 psf		From FEMA P-2139 Single Family Dwelling (SFD)
Engineered Trusses at 24" o.c.	5.0 psf		
18" Insulation	2.0 psf		
Vapor Barrier	0.5 psf		
5/8" Sheetrock	2.2 psf	$q_{roof} := 20\ psf$	$26\ psf$

Exterior Wall

Siding	2.0 psf		
1/2" CDX Plywood or OSB	2.5 psf		
House Wrap	0.5 psf		
2x6 Studs at 16" o.c.	1.5 psf		
R21 Fiberglass Insulation	1.5 psf		
6mil Clear Poly Vapor Barrier	0.5 psf		
5/8" Sheetrock finished	2.2 psf	$q_{wall} := 12\ psf$	$16\ psf$

Determine the seismic weight

Roof

$W_{roof\_r} := (q_{roof})\ (38\ ft)\ (26\ ft) = 20\ kip$

$W_{snow\_r} := 0.2\ p_f\ (38\ ft)\ (26\ ft) = 7.6\ kip$

$W_{walls\_r} := q_{wall}\ (4.5\ ft)\ (4 \cdot 26\ ft + 3 \cdot 38\ ft) = 11.8\ kip$ (include interior walls)

$W_r := W_{roof\_r} + W_{snow\_r} + W_{walls\_r} = 39.1\ kip$

Total Seismic Weight

$W := W_r = 39\ kip$

$W_{Dead} := W - W_{snow\_r} = 32\ kip$

Find Base Shear

Seismic design parameters from Chapters 11 of ASCE 7:

Site class = D — *11.4.2*
Risk category = II — *Table 1.5-1*
Importance factor: $I_e := 1.00$ — *Table 1.5-2*

$S_{DS} := 1.1$ — *ASCE 7 Hazard Tool*
$S_{D1} := 1.25$

Seismic Design Category D

Determine Response Modification Coefficient

Assume "Light Frame Wood Walls sheathed with Wood Panels"
$R := 6.5$ — *Table 12.2-1*

Period Approximation

$C_t := 0.02$ — *Table 12.8-2*
$x := 0.75$ (All other systems)
$h_n := 11\ ft$

$$T_a := C_t \cdot \left(\frac{h_n}{ft}\right)^x \cdot sec = 0.12\ s$$ *12.8-7*

$T_L := 16\ s$ — *Figure 22-13*
$T_a < T_L = 1$

Find Seismic Response Coefficient

$$C_{s.min} := \max\left(\frac{S_{D1}}{\frac{T_a}{s} \cdot \left(\frac{R}{I_e}\right)}, 0.044\ S_{DS} \cdot I_e, 0.01\right) = 1.59$$ *12.8-3*
12.8-5

$$C_{sa} := \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = 0.17$$ *12.8-2*

$$C_s := min\left(C_{s.min}, C_{sa}\right) = 0.17$$

Base Shear: $V := C_s \cdot W = 6.6\ kip$

Apply Seismic Load Combinations

$$E_v := 0.2 \cdot S_{DS} \cdot W_{Dead} = 7\ kip$$

$E_{h\_roof} := 1.3 \cdot V = 8.6\ kip$ where $q = 1.3$

Determine Wind Loads using Chapter 28 of ASCE 7-22

Table 28.2-1.

Steps to Determine Wind Loads on MWFRS Low-Rise Buildings.

Step 1: Determine risk category of building; see Table 1.5-1.
Step 2: Determine the basic wind speed, V, for applicable risk category; see Figure 26.5-1.
Step 3: Determine wind load parameters: • Wind directionality factor, K_d; see Section 26.6 and Table 26.6-1. • Exposure Category (B, C, or D); see Section 26.7. • Topographic factor, K_{zt}; see Section 26.8 and Figure 26.8-1. • Ground elevation factor, K_e; see Section 26.9 and Table 26.9-1. • Enclosure classification; see Section 26.12. • Internal pressure coefficient, (GC_{pi}); see Section 26.13 and Table 26.13-1.
Step 4: Determine velocity pressure exposure coefficient, K_z or K_h; see Table 26.10-1.
Step 5: Determine velocity pressure, q_z or q_h, from Equation (26.10-1).
Step 6: Determine external pressure coefficient, (GC_{pf}), for each load case using Section **28.3.2** for flat and gable roofs. **User Note:** See Commentary Figure C28.3-2 for guidance on hip roofs.
Step 7: Calculate wind pressure, p, from Equation (**28.3-1**).

Step 1

Building is Risk Cat II
Envelope is Partially Enclosed

Step 2

$V := 127\ mph$ ASCE 7-22 Hazard Tool

Step 3

$K_d := 0.85$ ASCE 7-22 Table 26.6-1

Exposure C

$K_{zt} := 1.0$ No large hill from topography (Section 26.8)

$K_e := 1.0$ Sea Level altitude (Table 26.9-1)

$GC_{pi} := 0.55$ Partially Exposed (Table 26.13-1)

Step 4

$K_h := 0.94$ Exposure C, 25 ft elevation altitude (ASCE 7-22 Table 26.10-1)

Step 5

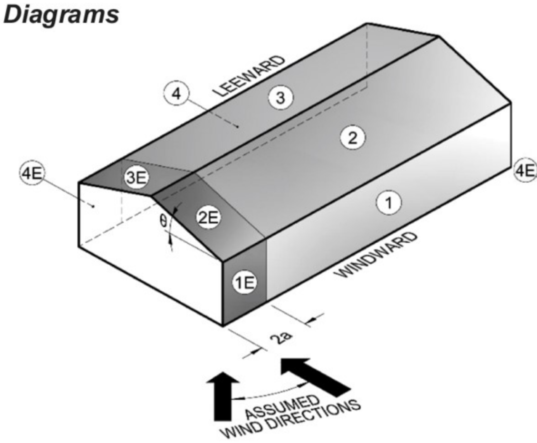
$$q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot \left(\frac{V}{mph}\right)^2 psf = 38.8\ psf$$

Step 6

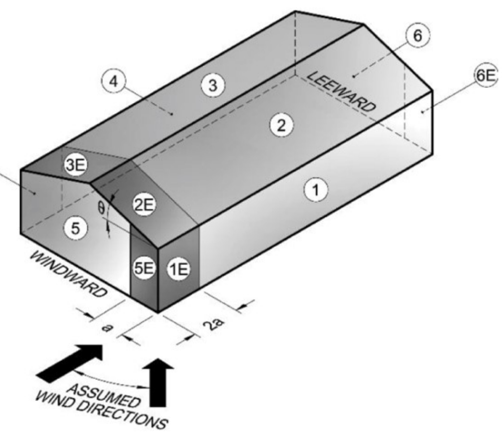
$$\theta := \text{atan}\left(\frac{4}{12}\right) = 18.4\ deg$$

--> Use 20 degrees (conservative)

Basic Load Cases Diagrams



Load Case 1



Load Case 2

Load Case 1

Roof Angle θ (degrees)	Building Surface							
	1	2	3	4	1E	2E	3E	4E
0–5	0.40	–0.69	–0.37	–0.29	0.61	–1.07	–0.53	–0.43
20	0.53	–0.69	–0.48	–0.43	0.80	–1.07	–0.69	–0.64
30–45	0.56	0.21	–0.43	–0.37	0.69	0.27	–0.53	–0.48
90	0.56	0.56	–0.37	–0.37	0.69	0.69	–0.48	–0.48

Load Case 1 (N/S): $a := \max(4\ ft, min(0.1 \cdot 38\ ft, 0.4 \cdot 26\ ft)) = 4\ ft$

$GC_{pf\_1} := 0.53$ $GC_{pf\_1E} := 0.80$

$GC_{pf\_2} := -0.69$ $GC_{pf\_2E} := -1.07$

$GC_{pf\_3} := -0.48$ $GC_{pf\_3E} := -0.69$

$GC_{pf\_4} := -0.43$ $GC_{pf\_4E} := -0.64$

Step 7

$p_{14} := q_h \cdot K_d \cdot (GC_{pf\_1} - GC_{pf\_4}) = 32\ psf$

$p_{14E} := q_h \cdot K_d \cdot (GC_{pf\_1E} - GC_{pf\_4E}) = 48\ psf$ $p_{wall1} := q_h \cdot K_d \cdot (GC_{pf\_1E} + GC_{pi}) = 45\ psf$

$$p_{23} := \frac{4}{12.6} \cdot q_h \cdot K_d \cdot (GC_{pf\_2} - GC_{pi}) - \frac{4}{12.6} \cdot q_h \cdot K_d \cdot (GC_{pf\_3} - GC_{pi}) = -2.2\ psf$$

$$p_{23E} := \frac{4}{12.6} \cdot q_h \cdot K_d \cdot (GC_{pf\_2E} - GC_{pi}) - \frac{4}{12.6} \cdot q_h \cdot K_d \cdot (GC_{pf\_3E} - GC_{pi}) = -4.0\ psf$$

Find Total Diaphragm Force - North-South

Roof Diaphragm (Ignore Roof Vertical Loads)

$q_r := p_{14} \cdot 5\ ft = 158\ plf$

$q_{rE} := p_{14E} \cdot 5\ ft = 238\ plf$

$F_r := q_{rE} \cdot 2\ a + q_r \cdot (38\ ft - 2\ a) = 6.7\ kip$ < $E_{h\_roof} = 8.6\ kip$ SEISMIC GOVERNS

Largest Wall Load:

$p_{1E} := q_h \cdot K_d \cdot (GC_{pf\_1E} + GC_{pi}) = 44.5\ psf$ (Ultimate Level)

$p_{1Ea} := 0.6\ q_h \cdot K_d \cdot (GC_{pf\_1E} + GC_{pi}) = 26.7\ psf$ (Service Level)

Load Case 2

Roof Angle θ (degrees)	Building Surface											
	1	2	3	4	5	6	1E	2E	3E	4E	5E	6E
0–90	–0.45	–0.69	–0.37	–0.45	0.40	–0.29	–0.48	–1.07	–0.53	–0.48	0.61	–0.43

Load Case 2 (Direction 1): $a := \max(4\ ft, min(0.1 \cdot 26\ ft, 0.4 \cdot 26\ ft)) = 4\ ft$

$GC_{pf\_1} := -0.45$ $GC_{pf\_1E} := -0.48$

$GC_{pf\_2} := -0.69$ $GC_{pf\_2E} := -1.07$

$GC_{pf\_3} := -0.37$ $GC_{pf\_3E} := -0.53$

$GC_{pf\_4} := -0.45$ $GC_{pf\_4E} := -0.48$

$GC_{pf\_5} := 0.40$ $GC_{pf\_5E} := 0.61$

$GC_{pf\_6} := -0.29$ $GC_{pf\_6E} := -0.43$

$p_{56} := q_h \cdot K_d \cdot (GC_{pf\_5} - GC_{pf\_6}) = 22.8\ psf$

$p_{56E} := q_h \cdot K_d \cdot (GC_{pf\_5E} - GC_{pf\_6E}) = 34.3\ psf$ $p_{wall2} := q_h \cdot K_d \cdot (GC_{pf\_5E} + GC_{pi}) = 38\ psf$

Find Total Diaphragm Force - (E/W)

Roof Diaphragm: Wall height varies from 8ft to 14ft

$q_r := p_{56} \cdot 6.5\ ft = 148\ plf$

$q_{rE} := p_{56E} \cdot 3\ ft = 103\ plf$

$F_{rw} := q_{rE} \cdot a + q_r \cdot (26\ ft - a) = 3.7\ kip$ < $E_{h\_roof} = 8.6\ kip$ SEISMIC GOVERN

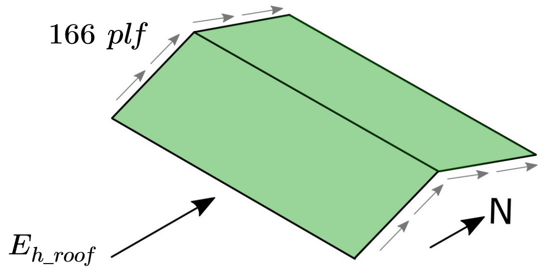
Largest Wall Load:

$p_{5E} := q_h \cdot K_d \cdot (GC_{pf\_5E} + GC_{pi}) = 38.3\ psf$ (Ultimate Level)

$p_{5Ea} := 0.6\ q_h \cdot K_d \cdot (GC_{pf\_5E} + GC_{pi}) = 23.0\ psf$ (Service Level)

Determine Upper Roof Diaphragm Forces

$E_{h\_roof} = 8.6\ kip$



North / South Direction

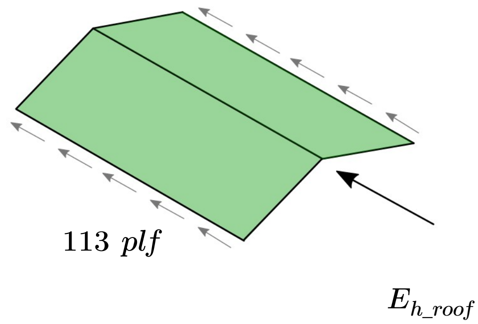
Roof Diaphragm Shear:

$$v_{roof1} := \frac{E_{h\_roof}}{2 \cdot 26\ ft} = 166\ plf \quad \text{(case 1)}$$

East / West Direction

Roof Diaphragm Shear:

$$v_{roof2} := \frac{E_{h\_roof}}{2 \cdot 38\ ft} = 113\ plf \quad \text{(case 3)}$$



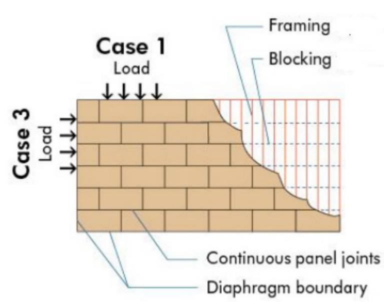
Roof Diaphragm Design

SDPWS 2021 Table 4.2C

Unblocked Wood Structural Panel Diaphragms[1,2,3,4,6]

Sheathing Grade	Common Nail Size[5] Length (in.) x Shank diameter (in.) x Head diameter (in.)	Minimum Nail Bearing Length in Framing Member, ℓ_m (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	6 in. Nail Spacing at diaphragm boundaries and supported panel edges					
					Case 1			Cases 2,3,4,5,6		
					v_n (plf)	G_a (kips/in) OSB	G_a (kips/in) PLY	v_n (plf)	G_a (kips/in) OSB	G_a (kips/in) PLY
Structural I	6d (2 x 0.113 x 0.266)	1-1/4	5/16	2	460	9.0	7.0	350	6.0	4.5
				3	520	7.0	6.0	390	4.5	4.0
	8d (2-1/2 x 0.131 x 0.281)	1-3/8	3/8	2	670	8.5	7.0	505	6.0	4.5
				3	740	7.5	6.0	560	5.0	4.0
	10d (3 x 0.148 x 0.312)	1-1/2	15/32	2	800	14	10	600	9.5	7.0
				3	895	12	9.0	670	8.0	6.0

Out-of-plane Loads: $p_s = 38.5\ psf$ (Service)



Assume 24" o.c. Supports (trusses)

Use 32/16 OSB APA Rated Sheathing
75 psf for L/240 at 24"o.c. (APA Q225 p. 6)
100 psf for L/180 at 24"o.c.

32/16 is **15/32 Sheathing** (see Table 11 of APA D510)

Check roof aspect ratio: $\frac{26\ ft}{38\ ft} = 0.68$ < 3.0, SDPWS 2021 Table 4.2.2

Use 8d nails at 6" o.c. at all panel edges with 3" supports (unblocked)

From SDPWS 4.1.4: $\phi := 0.50$

$v_{n1} := 740\ plf \qquad v_{n3} := 560\ plf$

$\phi v_{n1} := \phi \cdot v_{n1} = 370\ plf \quad > \quad v_{roof2} = 113\ plf$ OK

$\phi v_{n3} := \phi \cdot v_{n3} = 280\ plf \quad > \quad v_{roof1} = 166\ plf$ OK

Roof Chord Design

$$w := \frac{E_{h\_roof}}{38\ ft} = 227\ plf$$

$$M_{roof} := \frac{w \cdot (38\ ft)^2}{8} = 41\ kip \cdot ft$$

$$T_{roof} := \frac{M_{roof}}{24\ ft} = 1.7\ kip$$

$\frac{T_{roof}}{279\ lbf} = 6.1$ nails Ultimate strength of 10d nails in Doug-Fir that is part of a diaphragm $(C_{di} := 1.1)$

Determine Shear Wall Forces:

Out-of-plane Loads: $p_{wall1} = 44.5\ psf$

Assume studs are 16" o.c.

Use 32/16 OSB APA Rated Sheathing
61 psf for L/240 at 16"o.c. (APA Q225 p. 6)

32/16 is 15/32 Sheathing (see Table 11 of APA D510)

Span Rating[(b)]	Load Governed By[(c)]	Parallel to Supports Span Center-to-Center of Supports (inches)		
		12	16	24
24/0	L/360	48	18	
	L/240	72	27	
	L/180	96	36	
	Bending	81	45	
	Shear	248	179	
32/16	L/360	109	41	14
	L/240	163	61	21
	L/180	218	82	28
	Bending	138	77	28
	Shear	314	228	141

Table 4.3A

Wood-based Panels [4]

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Nail Bearing Length in Framing Member or Blocking, ℓ_m (in.)	Nail Type & Size [9] Length (in.) x Shank diameter (in.) x Head diameter (in.)	Panel Edge Nail Spacing (in.) 6 v_n (plf)	6 G_a (kips/in.) OSB	6 G_a (kips/in.) PLY	4 v_n (plf)	4 G_a (kips/in.) OSB	4 G_a (kips/in.) PLY	3 v_n (plf)	3 G_a (kips/in.) OSB	3 G_a (kips/in.) PLY	2 v_n (plf)	2 G_a (kips/in.) OSB	2 G_a (kips/in.) PLY
Wood Structural Panels - Structural I [4,5]	5/16	1-1/4	6d common nail (2 x 0.113 x 0.266) [8]	560	13	10	840	18	13	1090	23	16	1430	35	22
	3/8 [2]	1-3/8	8d common nail (2-1/2 x 0.131 x 0.281) [8]	645	19	14	1010	24	17	1290	30	20	1710	43	24
	7/16 [2]			715	16	13	1105	21	16	1415	27	19	1875	40	24
	15/32			785	14	11	1205	18	14	1540	24	17	2045	37	23
	15/32	1-1/2	10d common nail (3 x 0.148 x 0.312) [8,10]	950	22	16	1430	29	20	1860	36	22	2435	51	28
Wood Structural Panels - Sheathing [4,5]	5/16	1-1/4	6d common nail (2 x 0.113 x 0.266) [8]	505	13	9.5	755	18	12	980	24	14	1260	37	18
	3/8			560	11	8.5	840	15	11	1090	20	13	1430	32	17
	3/8 [2]	1-3/8	8d common nail (2-1/2 x 0.131 x 0.281) [8]	615	17	12	895	25	15	1150	31	17	1485	45	20
	7/16 [2]			670	15	11	980	22	14	1260	28	17	1640	42	21
	15/32			730	13	10	1065	19	13	1370	25	15	1790	39	20
	15/32	1-1/2	10d common nail (3 x 0.148 x 0.312) [8,10]	870	22	14	1290	30	17	1680	37	19	2155	52	23
	19/32			950	19	13	1430	26	16	1860	33	18	2435	48	22
Plywood Siding	5/16	1-1/4	6d galv.[7] casing nail (2 x 0.099 x 0.142)	390	13		590	16		770	17		1010	21	
	3/8	1-3/8	8d galv.[7] casing nail (2-1/2 x 0.113 x 0.155)	450	16		670	18		870	20		1150	22	

8d nails at 6" o.c. at edges with studs 16" o.c. (footnote 2): $v_{n6} := 730\ plf$ $\quad G_{a6} := 13\ \dfrac{kip}{in}$

8d nails at 4" o.c. at edges with studs 16" o.c. (footnote 2): $v_{n4} := 1065\ plf$

8d nails at 3" o.c. at edges with studs 16" o.c. (footnote 2): $v_{n3} := 1370\ plf$

8d nails at 2" o.c. at edges with studs 16" o.c. (footnote 2): $v_{n2} := 1790\ plf$

Shear Wall Strength:

Use 8d nails at 6"o.c. on one side of wall:

$$\phi V_n := \phi \cdot v_{n6} \cdot 12\ ft = 4.4\ kip \quad > \quad V_r := 0.5 \cdot E_{h\_roof} = 4.3\ kip$$

Find Strength Ratio:

$\phi V_{n8} := \phi \cdot v_{n6} \cdot 8\ ft = 2.9\ kip$ 8 foot wall

% Strength of 1 x 8 ft walls: $\dfrac{\phi V_n}{1 \cdot \phi V_{n8}} = 1.5$

Hold-down Uplift:

$$T := \frac{0.5\ (E_{h\_roof} \cdot 11\ ft)}{12\ ft} = 3.9\ kip$$ SDPWS (4.3-8)

Per general notes, multiply allowable loads in catalog by 1.43 to get LRFD values

Use HDU11-SDS2.5 with 4x6 Doug-Fir post. $T_{allowable} := 9.35\ kip$

Shear Wall Deflection:

From SDPWS 2021 - 4.3.4 - Deflection

$$\delta_{sw} = \frac{8 v h^3}{EAb} + \frac{vh}{1000 G_a} + \frac{h \Delta_a}{b} \qquad (4.3\text{-}1)$$

$h := 9\ ft$ $\quad b := 12\ ft$ $\quad G_{a6} = 13\ \frac{kip}{in}$

$v := \frac{0.5 \cdot V_r}{8\ ft} = 269\ plf$ Shear force applied to (1) 8ft walls

$E_{anc} := 40000\ \frac{lbf}{in}$ Vertical anchor stiffness based on string-pot measurement at end chords

$\Delta_a := \frac{v \cdot b}{E_{anc}} = 0.081\ in$

$E := 1400\ ksi$ End Post Modulus (No. 2 SPF)

$A := 3\ in \cdot 5.5\ in = 17\ in^2$ End Post Cross Section

$$\delta_{sw8} := \frac{8\ v \cdot (8\ ft)^3}{E \cdot A \cdot 8\ ft} + \frac{v \cdot 8\ ft}{1000 \cdot G_{a6}} + \frac{8\ ft \cdot \Delta_a}{8\ ft} = 0.137\ in$$ 8x8 foot wall deflection

$\frac{8\ v \cdot (8\ ft)^3}{E \cdot A \cdot 8\ ft} = 0.07\ in$ $\quad \frac{v \cdot 8\ ft}{1000 \cdot G_{a6}} = 0.000\ in$ $\quad \frac{8\ ft \cdot \Delta_a}{8\ ft} = 0.081\ in$

Per SDPWS 2021 - 4.3.4.2

$$v_{design} := \frac{0.5 \cdot V_r}{12\ ft} = 179\ plf$$

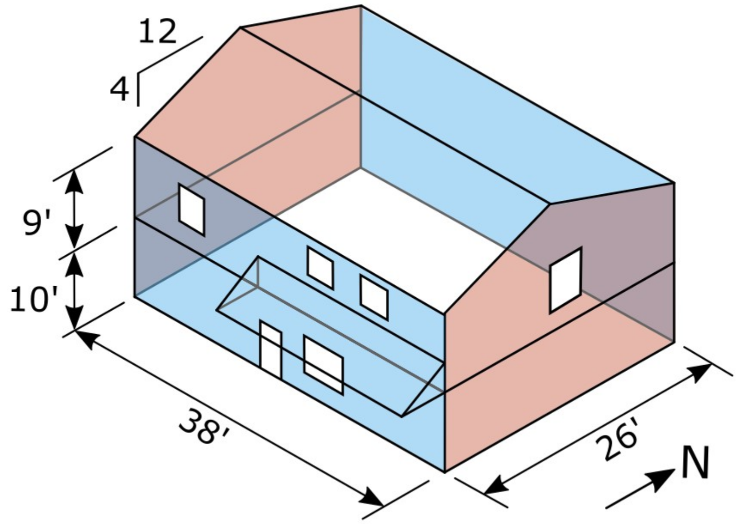
$$\delta_{sw} := \frac{8\ v_{design} \cdot h^3}{E \cdot A \cdot b} + \frac{v_{design} \cdot h}{1000 \cdot G_{a6}} + \frac{h \cdot v_{design}}{E_{anc}} = 0.086\ in$$ Actual wall deflection

Find Stiffness Ratio:

%Stiffness of 8 ft wall: $\frac{\delta_{sw8}}{\delta_{sw}} = 1.59$

Two-Story Structure located in Anchorage, AK. Risk Category II, Soil type C, Wind Exposure C. Calculations using ASCE 7-22.

This house meets the requirements of a structure in a D_2 seismic zone designed per the IRC



Find Ground Snow Load

$p_g := 50\ psf$ https://seaak.net/alaska-snow-loads (50 yr MRI)

Flat Roof Snow Load

$p_f = 0.7\ C_e \cdot C_t \cdot I_s \cdot p_g$ (7.3-1)

$C_e := 1.0$ Table 7.3-1 - Partially Exposed, Exposure C
$C_t := 1.1$ Table 7.3-2 - Cold Roof
$I_s := 1.0$ Table 1.5-2 - Risk Cat II

$p_f := 0.7\ C_e \cdot C_t \cdot I_s \cdot p_g = 38.5\ psf$

Sloped Roof

$C_s := 1.0$ Figure 7.4-1, 3 on 12 non-slippery roof with $C_t = 1.1$

$p_s := C_s \cdot p_f = 38.5\ psf$

Dead Loads - See SF3 Design

$q_{roof} := 20\ psf$

$q_{floor} := 15\ psf$

$q_{wall} := 12\ psf$

Determine the seismic weight

Roof

$W_{roof\_r} := (q_{roof})\ (38\ ft)\ (26\ ft) = 20\ kip$
$W_{snow\_r} := 0.2\ p_f\ (38\ ft)\ (26\ ft) = 7.6\ kip$
$W_{walls\_r} := q_{wall}\ (4.5\ ft)\ (4 \cdot 26\ ft + 3 \cdot 38\ ft) = 11.8\ kip$ (include interior walls)

$W_r := W_{roof\_r} + W_{snow\_r} + W_{walls\_r} = 39.1\ kip$

2nd Floor

$W_{floor2} := (q_{floor})\ (38\ ft)\ (26\ ft) = 14.8\ kip$
$W_{walls2} := (q_{wall})\ (9.5\ ft)\ (4 \cdot 26\ ft + 3 \cdot 38\ ft) = 24.9\ kip$

$W_2 := W_{floor2} + W_{walls2} = 39.7\ kip$

Total Seismic Weight

$W := W_r + W_2 = 79\ kip$

$W_{Dead} := W - W_{snow\_r} = 71\ kip$

Find Base Shear

Seismic design parameters from Chapters 11 of ASCE 7:

Site class = D *11.4.2*
Risk category = II *Table 1.5-1*
Importance factor: $I_e := 1.00$ *Table 1.5-2*

$S_{DS} := 1.1$ *ASCE 7 Hazard Tool*
$S_{D1} := 1.25$

Seismic Design Category D

Determine Response Modification Coefficient

Assume "Light Frame Wood Walls sheathed with Wood Panels"
$R := 6.5$ *Table 12.2-1*

Period Approximation

$C_t := 0.02$ $x := 0.75$ (All other systems) *Table 12.8-2*
$h_n := 24\ ft$

$T_a := C_t \cdot \left(\frac{h_n}{ft}\right)^x \cdot sec = 0.22\ s$ *12.8-7*
$T_L := 16\ s$ *Figure 22-13*
$T_a < T_L = 1$

Find Seismic Response Coefficient

$$C_{s.min} := \max\left(\frac{S_{D1}}{\frac{T_a}{s}\cdot\left(\frac{R}{I_e}\right)}, 0.044\ S_{DS}\cdot I_e, 0.01\right) = 0.89$$ *12.8-3* *12.8-5*

$$C_{sa} := \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = 0.169$$ *12.8-2*

$$C_s := min\left(C_{s.min}, C_{sa}\right) = 0.169$$

Base Shear: $V := C_s \cdot W = 13.3\ kip$

Calculate the seismic force, Fx, at each level

From ASCE7-16, 12.8.3 $C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ (12.8-12)

$T_a = 0.22\ s$ therefore $k := 1.0$

$W_r = 39.1\ kip$

$W_2 = 39.7\ kip$

$h_r := 21\ ft$ $h_2 := 10\ ft$

$$C_{vr} := \frac{W_r\cdot\left(h_r\right)^k}{W_r\cdot\left(h_r\right)^k + W_2\cdot\left(h_2\right)^k} = 0.67$$

$F_{roof} := C_{vr}\cdot V = 9\ kip$

$$C_{v2} := \frac{W_2\cdot\left(h_2\right)^k}{W_r\cdot\left(h_r\right)^k + W_2\cdot\left(h_2\right)^k} = 0.33$$

$F_2 := C_{v2}\cdot V = 4.3\ kip$

Apply Seismic Load Combinations

$E_v := 0.2\cdot S_{DS}\cdot W_{Dead} = 16\ kip$

$E_{h\_roof} := 1.3\cdot F_{roof} = 11.7\ kip$ where $q = 1.3$

$E_{h\_2} := 1.3\cdot F_2 = 5.6\ kip$ where $q = 1.3$

$F_r := E_{h\_roof}$

$F_2 := E_{h\_2}$

$V_2 := F_r + F_2 = 17.3\ kip$

Determine Wind Loads using Chapter 28 of ASCE 7-22

See SF3 - Seismic Governs

Roof and Floor Diaphragm Forces

Building mass and layout are the same as SF3, so diaphragm designs are same as SF3

Determine Shear Wall Forces:

Out-of-plane Loads: $p_{wall1} := 45\ psf$

Assume studs are 16" o.c.

Use 32/16 OSB APA Rated Sheathing
61 psf for L/240 at 16"o.c. (APA Q225 p. 6)

32/16 is 15/32 Sheathing (see Table 11 of APA D510)

Span Rating[b]	Load Governed By[c]	Parallel to Supports Span Center-to-Center of Supports (inches) 12	16	24
24/0	L/360	48	18	
	L/240	72	27	
	L/180	96	36	
	Bending	81	45	
	Shear	248	179	
32/16	L/360	109	41	14
	L/240	163	61	21
	L/180	218	82	28
	Bending	138	77	28
	Shear	314	228	141

<u>Table 4.3A</u>

Wood-based Panels [4]

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Nail Bearing Length in Framing Member or Blocking, ℓ_m (in.)	Nail Type & Size [9] Length (in.) x Shank diameter (in.) x Head diameter (in.)	6: v_n (plf)	6: G_a (kips/in.) OSB	6: G_a PLY	4: v_n (plf)	4: G_a (kips/in.) OSB	4: G_a PLY	3: v_n (plf)	3: G_a (kips/in.) OSB	3: G_a PLY	2: v_n (plf)	2: G_a (kips/in.) OSB	2: G_a PLY
Wood Structural Panels - Structural I [4,5]	5/16	1-1/4	6d common nail (2 x 0.113 x 0.266) [8]	560	13	10	840	18	13	1090	23	16	1430	35	22
	3/8 [2]	1-3/8	8d common nail (2-1/2 x 0.131 x 0.281) [8]	645	19	14	1010	24	17	1290	30	20	1710	43	24
	7/16 [2]			715	16	13	1105	21	16	1415	27	19	1875	40	24
	15/32			785	14	11	1205	18	14	1540	24	17	2045	37	23
	15/32	1-1/2	10d common nail (3 x 0.148 x 0.312) [8,10]	950	22	16	1430	29	20	1860	36	22	2435	51	28
Wood Structural Panels - Sheathing [4,5]	5/16	1-1/4	6d common nail (2 x 0.113 x 0.266) [8]	505	13	9.5	755	18	12	980	24	14	1260	37	18
	3/8			560	11	8.5	840	15	11	1090	20	13	1430	32	17
	3/8 [2]	1-3/8	8d common nail (2-1/2 x 0.131 x 0.281) [8]	615	17	12	895	25	15	1150	31	17	1485	45	20
	7/16 [2]			670	15	11	980	22	14	1260	28	17	1640	42	21
	15/32			730	13	10	1065	19	13	1370	25	15	1790	39	20
	15/32	1-1/2	10d common nail (3 x 0.148 x 0.312) [8,10]	870	22	14	1290	30	17	1680	37	19	2155	52	23
	19/32			950	19	13	1430	26	16	1860	33	18	2435	48	22
Plywood Siding	5/16	1-1/4	6d galv.[7] casing nail (2 x 0.099 x 0.142)	390	13		590	16		770	17		1010	21	
	3/8	1-3/8	8d galv.[7] casing nail (2-1/2 x 0.113 x 0.155)	450	16		670	18		870	20		1150	22	

8d nails at 6" o.c. at edges with studs 16" o.c. (footnote 2): $v_{n6} := 730\ plf$ $\quad G_{a6} := 13\ \frac{kip}{in}$

Perforated Shear Wall Strength:

Use the Perforated Wall method to determine the wall shear and hold-down tension.

$V_n = v_n C_o \Sigma b_i$ (4.3-5)

where:

v_n = nominal unit shear capacity, lbs/ft

C_o = shear capacity adjustment factor from Table 4.3.5.6 or calculated using the following equation:

$C_o = A_{wall} / (3A_o + A_{fhs}) \leq 1.0$ (4.3-6)

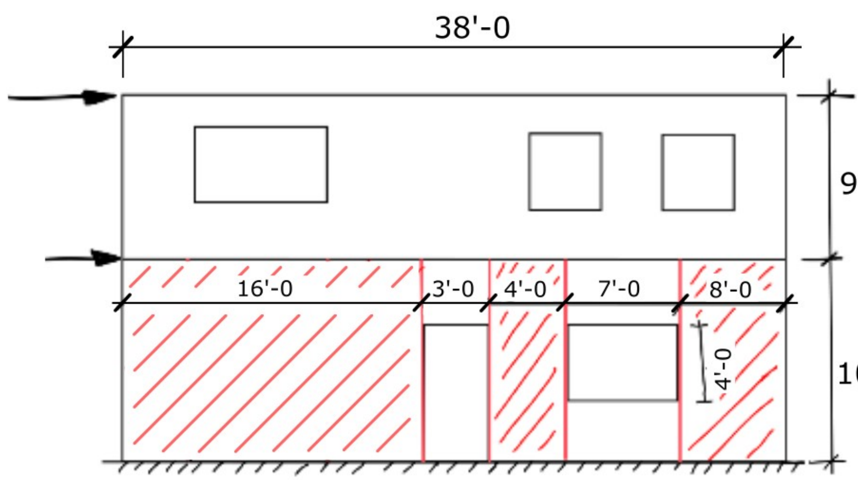
where:

A_{fhs} = total area sheathed with full-height sheathing, ft², regardless of whether individual wall segments meet the aspect ratio limits in 4.3.3.4.

A_o = total area of openings in the perforated shear wall where individual opening areas are calculated as the opening width times the clear opening height, ft².

A_{wall} = total area of a perforated shear wall equal to the length of the perforated shear wall times its height, ft²

Σb_i = sum of perforated shear wall segment lengths b_i, ft. Lengths of perforated shear wall segments with aspect ratios greater than 2:1 shall be adjusted in accordance with 4.3.3.4.



$16\ ft + 3\ ft + 4\ ft + 7\ ft + 8\ ft = 38\ ft$

South Wall - Level 1

$L_{tot} := 38\ ft$ $\quad h := 10\ ft$ $\quad \phi := 0.5$

$A_{wall} := L_{tot} \cdot h = 380\ ft^2$

$A_{fhs} := (16\ ft + 4\ ft + 8\ ft) \cdot h = 280\ ft^2$ Area of full height walls

$A_o := 7\ ft \cdot 3\ ft + 7\ ft \cdot 4\ ft = 49\ ft^2$ Opening area

$$C_o := min\left(\frac{A_{wall}}{(3 \cdot A_o + A_{fhs})}, 1.0\right) = 0.89$$

Check Segment Aspect Ratios (4.3.3.4):

$b_1 := 4.0\ ft$ Smallest full-height wall segments

$\frac{h}{b_1} = 2.5$ > 2.0 must be reduced (unblocked wood structural panel)
< 3.5 - Can be included in b_i

$$\frac{2 \cdot b_1}{h} \cdot b_1 = 3.2\ ft$$

$\Sigma b_i := 16\ ft + 3.2\ ft + 8\ ft = 27\ ft$

Use 8d nails at 6"o.c. on one side of wall:

$\phi V_n := \phi \cdot v_{n6} \cdot C_o \cdot \Sigma b_i = 8.8\ kip$ > $0.5 \cdot V_2 = 8.7\ kip$

Find Strength Ratio:

$\phi V_{n8} := \phi \cdot v_{n6} \cdot 8\ ft = 2.9\ kip$ 8 foot wall

% Strength of (4) 8 ft walls: $\dfrac{\phi V_n}{4 \cdot \phi V_{n8}} = 0.76$

Hold-down Uplift:

$$T := \frac{0.5\ \left(E_{h\_roof} \cdot 19\ ft + E_{h\_roof} \cdot 9\ ft\right)}{1.43\ C_o \cdot \Sigma b_i} = 4.7\ kip$$ SDPWS (4.3-8)

Per general notes, multiply allowable loads in catalog by 1.43 to get LRFD values

Use HDU11-SDS2.5 with 4x6 Doug-Fir post. $T_{allowable} := 9.35\ kip$

Perforated Shear Wall Deflection:

From SDPWS 2021 - 4.3.4 - Deflection

$v := \dfrac{0.5 \cdot V_2}{4 \cdot 8\ ft} = 271\ plf$ Shear force applied to (4) 8ft walls

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

$h = 10\ ft$ $b := 8\ ft$ $G_{a6} = 13\ \dfrac{kip}{in}$

$E_{anc} := 40000\ \dfrac{lbf}{in}$ Vertical anchor stiffness based on string-pot measurement at end chords

$\Delta_a := \dfrac{v \cdot b}{E_{anc}} = 0.054\ in$

$E := 1400\ ksi$ End Post Modulus (No. 2 SPF)

$A := 3\ in \cdot 5.5\ in = 17\ in^2$ End Post Cross Section

$$\delta_{sw8} := \frac{8\ v \cdot h^3}{E \cdot A \cdot b} + \frac{v \cdot h}{1000 \cdot G_{a6}} + \frac{h \cdot \Delta_a}{b} = 0.266\ in$$

8 foot wall deflection

$$\frac{8\ v \cdot h^3}{E \cdot A \cdot b} = 0.141\ in \qquad \frac{v \cdot h}{1000 \cdot G_{a6}} = 0.000\ in \qquad \frac{h \cdot \Delta_a}{b} = 0.068\ in$$

Per SDPWS 2021 - 4.3.4.2

$$v_{max} = \frac{V}{C_o \sum b_i} \qquad (4.3\text{-}9)$$

$$v_{max} := \frac{0.5 \cdot V_2}{26\ ft} = 333\ plf \qquad b = 8\ ft$$

$$\delta_{sw} := \frac{8\ v_{max} \cdot h^3}{E \cdot A \cdot \Sigma b_i} + \frac{v_{max} \cdot h}{1000 \cdot G_{a6}} + \frac{h \cdot v_{max}}{E_{anc}} = 0.135\ in$$

Perforated wall deflection

$$\frac{8\ v_{max} \cdot h^3}{E \cdot A \cdot \Sigma b_i} = 0.051\ in \qquad \frac{v_{max} \cdot h}{1000 \cdot G_{a6}} = 0.000\ in \qquad \frac{v_{max} \cdot h}{E_{anc}} = 0.083\ in$$

Find Stiffness Ratio:

%Stiffness of 8 ft wall: $\frac{\delta_{sw8}}{\delta_{sw}} = 1.98$

East Wall - Level 1

$L_{tot} := 26\ ft$

$h := 10\ ft$

Use full length wall (no windows)

$\phi V_n := \phi \cdot v_{n6} \cdot 26\ ft = 9.5\ kip \quad > \quad 0.5 \cdot V_2 = 8.7\ kip$

Use 8d nails at 6"o.c. on one side of wall

Find Strength Ratio:

$\phi V_{n8} := \phi \cdot v_{n6} \cdot 8\ ft = 2.9\ kip$ 8 foot wall

% Strength of 8 ft walls: $\dfrac{\phi V_n}{3 \cdot \phi V_{n8}} = 1.083$

Hold-down Uplift:

$$T := \frac{0.5\ \left(E_{h\_roof} \cdot 19\ ft + E_{h\_roof} \cdot 9\ ft\right)}{1.43\ C_o \cdot \Sigma b_i} = 4.7\ kip$$ SDPWS (4.3-8)

Per general notes, multiply allowable loads in catalog by 1.43 to get LRFD values

Use HDU11-SDS2.5 with 4x6 Doug-Fir post. $T_{allowable} := 9.35\ kip$

Perforated Shear Wall Deflection:

From SDPWS 2021 - 4.3.4 - Deflection

$v := \dfrac{0.5 \cdot V_2}{3 \cdot 8\ ft} = 361\ plf$ Shear force applied to (3) 8ft walls

$$\delta_{sw8} := \frac{8\ v \cdot (8\ ft)^3}{E \cdot A \cdot 8\ ft} + \frac{v \cdot 8\ ft}{1000 \cdot G_{a6}} + \frac{8\ ft \cdot v}{E_{anc}} = 0.169\ in$$ 8x8 ft wall deflection

Per SDPWS 2021 - 4.3.4.2 $b := L_{tot} = 26\ ft$

$v_{max} := \dfrac{0.5 \cdot V_2}{b} = 333\ plf$ (eqn 4.3-9)

$$\delta_{sw} := \frac{8\ v_{max} \cdot h^3}{E \cdot A \cdot b} + \frac{v_{max} \cdot h}{1000 \cdot G_{a6}} + \frac{h \cdot v_{max}}{E_{anc}} = 0.137\ in$$ wall deflection

Find Stiffness Ratio:

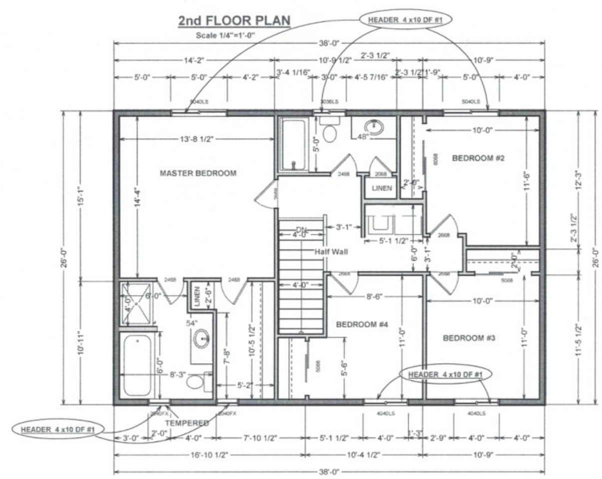
%Stiffness of 8 ft wall: $\frac{\delta_{sw8}}{\delta_{sw}} = 1.23$

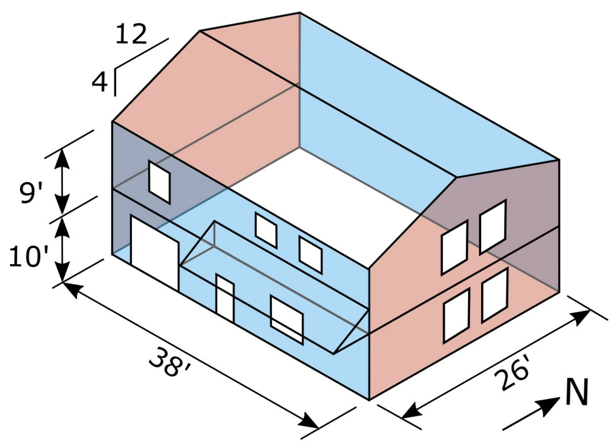
Two-Story Structure located in Anchorage, AK. Risk Category II, Soil type C, Wind Exposure C. Calculations using ASCE 7-22.

This house is similar to FEMA P2139 SFD2 (two story singly family). FEMA house is 32'x48':

Table 3-5 Short-Period Wood Light-Frame Building Archetype Configurations and Seismic Design Criteria

Archetype ID	Configuration: No. of Stories	Configuration: Wall Aspect Ratio	Seismic Design Criteria: Seismic Code	Design Period (1) $T = C_u T_a$ (sec)	Seismic Design Category (SDC)	MCE_R Design Parameter, S_{MS} (g)	Response Modification Coefficient (R)	Seismic Response Coefficient, C_s (g)
SFD2	2	High	ASCE 7	0.25	D	1.5	6.5	0.154





Find Ground Snow Load

$p_g := 50\ psf$ https://seaak.net/alaska-snow-loads (50 yr MRI)

Flat Roof Snow Load

$p_f = 0.7\ C_e \cdot C_t \cdot I_s \cdot p_g$ (7.3-1)

$C_e := 1.0$ Table 7.3-1 - Partially Exposed, Exposure C
$C_t := 1.1$ Table 7.3-2 - Cold Roof
$I_s := 1.0$ Table 1.5-2 - Risk Cat II

$p_f := 0.7\ C_e \cdot C_t \cdot I_s \cdot p_g = 38.5\ psf$

Sloped Roof

$C_s := 1.0$ Figure 7.4-1, 3 on 12 non-slippery roof with $C_t = 1.1$

$p_s := C_s \cdot p_f = 38.5\ psf$

Find Dead Loads

Roof:

Composition Shingles	2.0 psf		
Roof Wrap/tar paper	0.5 psf		
Second Lay Shingles	2.0 psf		
5/8" CDX Plywood	3.0 psf		From FEMA P-2139 Single Family Dwelling (SFD)
Engineered Trusses at 24" o.c.	5.0 psf		
18" Insulation	2.0 psf		
Vapor Barrier	0.5 psf		
5/8" Sheetrock	2.2 psf	$q_{roof} := 20\ psf$	$26\ psf$

Floor:

Carpet or Flooring	1.0 psf		
3/4" T&G Plywood	4.0 psf		
12" BCI I-joists at 16" o.c.	3.0 psf		
R21 Fiberglass Insulation	1.5 psf		
psf6mil Clear Poly Vapor Barrier	0.5 psf		
5/8" Sheetrock finished	2.2 psf	$q_{floor} := 15\ psf$	$29\ psf$

Exterior Wall

Siding	2.0 psf		
1/2" CDX Plywood or OSB	2.5 psf		
House Wrap	0.5 psf		
2x6 Studs at 16" o.c.	1.5 psf		
R21 Fiberglass Insulation	1.5 psf		
6mil Clear Poly Vapor Barrier	0.5 psf		
5/8" Sheetrock finished	2.2 psf	$q_{wall} := 12\ psf$	$16\ psf$

Determine the seismic weight

Roof

$W_{roof\_r} := \left(q_{roof}\right) \left(38\ ft\right) \left(26\ ft\right) = 20\ kip$

$W_{snow\_r} := 0.2\ p_f \left(38\ ft\right) \left(26\ ft\right) = 7.6\ kip$

$W_{walls\_r} := q_{wall} \left(4.5\ ft\right) \left(4 \cdot 26\ ft + 3 \cdot 38\ ft\right) = 11.8\ kip$ (include interior walls)

$W_r := W_{roof\_r} + W_{snow\_r} + W_{walls\_r} = 39.1\ kip$

2nd Floor

$$W_{floor2} := (q_{floor})\ (38\ ft)\ (26\ ft) = 14.8\ kip$$
$$W_{walls2} := (q_{wall})\ (9.5\ ft)\ (4 \cdot 26\ ft + 3 \cdot 38\ ft) = 24.9\ kip$$

$$W_2 := W_{floor2} + W_{walls2} = 39.7\ kip$$

Total Seismic Weight

$$W := W_r + W_2 = 79\ kip$$

$$W_{Dead} := W - W_{snow\_r} = 71\ kip$$

Find Base Shear

Seismic design parameters from Chapters 11 of ASCE 7:

Site class = D — *11.4.2*
Risk category = II — *Table 1.5-1*
Importance factor: $I_e := 1.00$ — *Table 1.5-2*

$S_{DS} := 1.1$ — *ASCE 7 Hazard Tool*
$S_{D1} := 1.25$

Seismic Design Category D

Determine Response Modification Coefficient

Assume "Light Frame Wood Walls sheathed with Wood Panels"

$R := 6.5$ — *Table 12.2-1*

Period Approximation

$C_t := 0.02$ — *Table 12.8-2*
$x := 0.75$ (All other systems)
$h_n := 21\ ft$

$$T_a := C_t \cdot \left(\frac{h_n}{ft}\right)^x \cdot sec = 0.2\ s$$ *12.8-7*

$T_L := 16\ s$ — *Figure 22-13*
$T_a < T_L = 1$

Find Seismic Response Coefficient

$$C_{s.min} := \max\left(\frac{S_{D1}}{\frac{T_a}{s} \cdot \left(\frac{R}{I_e}\right)}, 0.044\ S_{DS} \cdot I_e, 0.01\right) = 0.98$$

12.8-3
12.8-5

$$C_{sa} := \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = 0.169$$ *12.8-2*

$$\boxed{C_s} := min\left(C_{s.min}, C_{sa}\right) = 0.169$$

Base Shear: $V := C_s \cdot W = 13.3\ kip$

Calculate the seismic force, Fx, at each level

From ASCE7-16, 12.8.3 $C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ (12.8-12)

$T_a = 0.20\ s$ therefore $k := 1.0$

$W_r = 39.1\ kip$
$W_2 = 39.7\ kip$

$h_r := 20.5\ ft$ $h_2 := 10\ ft$

$$C_{vr} := \frac{W_r \cdot \left(h_r\right)^k}{W_r \cdot \left(h_r\right)^k + W_2 \cdot \left(h_2\right)^k} = 0.67$$

$F_{roof} := C_{vr} \cdot V = 8.9\ kip$

$$C_{v2} := \frac{W_2 \cdot \left(h_2\right)^k}{W_r \cdot \left(h_r\right)^k + W_2 \cdot \left(\left(h_2\right)^k\right)} = 0.33$$

$F_2 := C_{v2} \cdot V = 4.4\ kip$

Apply Seismic Load Combinations

$E_v := 0.2 \cdot S_{DS} \cdot W_{Dead} = 16\ kip$

$E_{h\_roof} := 1.3 \cdot F_{roof} = 11.6\ kip$ where $q = 1.3$

$E_{h\_2} := 1.3 \cdot F_2 = 5.7\ kip$ where $q = 1.3$

Determine Wind Loads using Chapter 28 of ASCE 7-22

Table 28.2-1.

Steps to Determine Wind Loads on MWFRS Low-Rise Buildings.

Step 1: Determine risk category of building; see Table 1.5-1.
Step 2: Determine the basic wind speed, V, for applicable risk category; see Figure 26.5-1.
Step 3: Determine wind load parameters: • Wind directionality factor, K_d; see Section 26.6 and Table 26.6-1. • Exposure Category (B, C, or D); see Section 26.7. • Topographic factor, K_{zt}; see Section 26.8 and Figure 26.8-1. • Ground elevation factor, K_e; see Section 26.9 and Table 26.9-1. • Enclosure classification; see Section 26.12. • Internal pressure coefficient, (GC_{pi}); see Section 26.13 and Table 26.13-1.
Step 4: Determine velocity pressure exposure coefficient, K_z or K_h; see Table 26.10-1.
Step 5: Determine velocity pressure, q_z or q_h, from Equation (26.10-1).
Step 6: Determine external pressure coefficient, (GC_{pf}), for each load case using Section **28.3.2** for flat and gable roofs. **User Note:** See Commentary Figure C28.3-2 for guidance on hip roofs.
Step 7: Calculate wind pressure, p, from Equation (**28.3-1**).

Step 1

Building is Risk Cat II

Envelope is Partially Enclosed

Step 2

$V := 127\ mph$ ASCE 7-22 Hazard Tool

Step 3

$K_d := 0.85$ ASCE 7-22 Table 26.6-1

Exposure C

$K_{zt} := 1.0$ No large hill from topography (Section 26.8)

$K_e := 1.0$ Sea Level altitude (Table 26.9-1)

$GC_{pi} := 0.55$ Partially Exposed (Table 26.13-1)

Step 4

$K_h := 0.94$ Exposure C, 25 ft elevation altitude (ASCE 7-22 Table 26.10-1)

Step 5

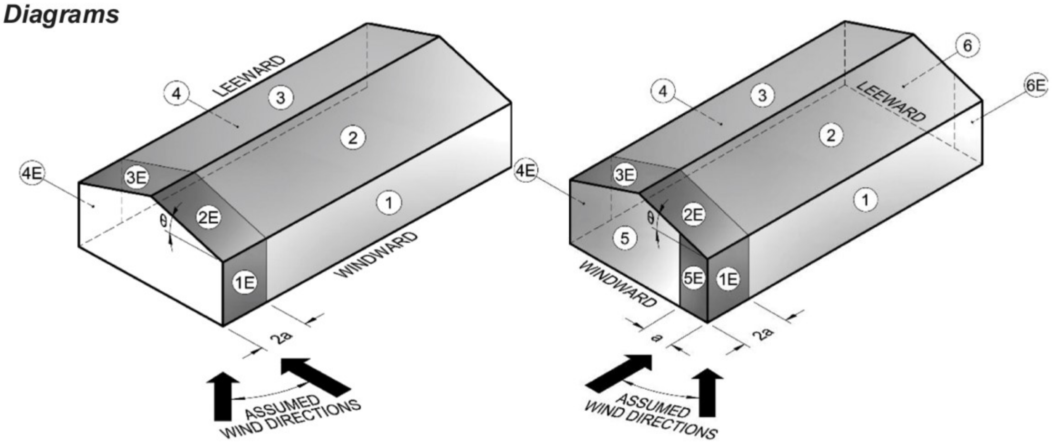
$$q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot \left(\frac{V}{mph}\right)^2 psf = 38.8\ psf$$

Step 6

$$\theta := \operatorname{atan}\left(\frac{4}{12}\right) = 18.4\ deg$$

--> Use 20 degrees (conservative)

Basic Load Cases Diagrams



Load Case 1 **Load Case 2**

***Load Case* 1**

Roof Angle θ (degrees)	Building Surface							
	1	2	3	4	1E	2E	3E	4E
0–5	0.40	−0.69	−0.37	−0.29	0.61	−1.07	−0.53	−0.43
20	0.53	−0.69	−0.48	−0.43	0.80	−1.07	−0.69	−0.64
30–45	0.56	0.21	−0.43	−0.37	0.69	0.27	−0.53	−0.48
90	0.56	0.56	−0.37	−0.37	0.69	0.69	−0.48	−0.48

Load Case 1 (N/S): $a := \max\left(4\ ft, min\left(0.1 \cdot 38\ ft, 0.4 \cdot 26\ ft\right)\right) = 4\ ft$

$GC_{pf\_1} := 0.53$ $\qquad GC_{pf\_1E} := 0.80$

$GC_{pf\_2} := -0.69$ $\qquad GC_{pf\_2E} := -1.07$

$GC_{pf\_3} := -0.48$ $\qquad GC_{pf\_3E} := -0.69$

$GC_{pf\_4} := -0.43$ $\qquad GC_{pf\_4E} := -0.64$

Step 7

$p_{14} := q_h \cdot K_d \cdot \left(GC_{pf\_1} - GC_{pf\_4}\right) = 32\ psf$

$p_{14E} := q_h \cdot K_d \cdot \left(GC_{pf\_1E} - GC_{pf\_4E}\right) = 48\ psf$ $\qquad p_{wall1} := q_h \cdot K_d \cdot \left(GC_{pf\_1E} + GC_{pi}\right) = 45\ psf$

$$p_{23} := \frac{4}{12.6} \cdot q_h \cdot K_d \cdot \left(GC_{pf\_2} - GC_{pi}\right) - \frac{4}{12.6} \cdot q_h \cdot K_d \cdot \left(GC_{pf\_3} - GC_{pi}\right) = -2.2\ psf$$

$$p_{23E} := \frac{4}{12.6} \cdot q_h \cdot K_d \cdot \left(GC_{pf\_2E} - GC_{pi}\right) - \frac{4}{12.6} \cdot q_h \cdot K_d \cdot \left(GC_{pf\_3E} - GC_{pi}\right) = -4.0\ psf$$

Find Total Diaphragm Force - North-South

Roof Diaphragm (Ignore Roof Vertical Loads)

$q_r := p_{14} \cdot 5\ ft = 158\ plf$

$q_{rE} := p_{14E} \cdot 5\ ft = 238\ plf$

$F_{rw} := q_{rE} \cdot 2\ a + q_r \cdot \left(38\ ft - 2\ a\right) = 6.7\ kip$ < $F_r := E_{h\_roof} = 11.6\ kip$ SEISMIC GOVERNS

Floor Diaphragm (Ignore Roof Loads)

$q_2 := p_{14} \cdot 9\ ft = 285\ plf$

$q_{2E} := p_{14E} \cdot 9\ ft = 428\ plf$

$F_{2w} := q_{2E} \cdot 2\ a + q_2 \cdot \left(38\ ft - 2\ a\right) = 12\ kip$ > $\boxed{F_2} := E_{h\_2} = 5.7\ kip$ WIND GOVERNS

$V_{2w} := F_r + F_2 = 17.3\ kip$ < $V_2 := E_{h\_roof} + E_{h\_2} = 17.3\ kip$

WIND Creates larger forces, but Seismic will govern when ϕ is included

Largest Wall Load:

$p_{1E} := q_h \cdot K_d \cdot \left(GC_{pf\_1E} + GC_{pi}\right) = 44.5\ psf$ (Ultimate Level)

$p_{1Ea} := 0.6\ q_h \cdot K_d \cdot \left(GC_{pf\_1E} + GC_{pi}\right) = 26.7\ psf$ (Service Level)

Load Case 2

Roof Angle θ (degrees)	Building Surface											
	1	2	3	4	5	6	1E	2E	3E	4E	5E	6E
0–90	–0.45	–0.69	–0.37	–0.45	0.40	–0.29	–0.48	–1.07	–0.53	–0.48	0.61	–0.43

Load Case 2 (Direction 1): $a := \max\left(4\ ft, min\left(0.1 \cdot 26\ ft, 0.4 \cdot 26\ ft\right)\right) = 4\ ft$

$GC_{pf\_1} := -0.45$
$GC_{pf\_2} := -0.69$
$GC_{pf\_3} := -0.37$
$GC_{pf\_4} := -0.45$
$GC_{pf\_5} := 0.40$
$GC_{pf\_6} := -0.29$

$GC_{pf\_1E} := -0.48$
$GC_{pf\_2E} := -1.07$
$GC_{pf\_3E} := -0.53$
$GC_{pf\_4E} := -0.48$
$GC_{pf\_5E} := 0.61$
$GC_{pf\_6E} := -0.43$

$p_{56} := q_h \cdot K_d \cdot \left(GC_{pf\_5} - GC_{pf\_6}\right) = 22.8\ psf$

$p_{56E} := q_h \cdot K_d \cdot \left(GC_{pf\_5E} - GC_{pf\_6E}\right) = 34.3\ psf$ $p_{wall2} := q_h \cdot K_d \cdot \left(GC_{pf\_5E} + GC_{pi}\right) = 38\ psf$

Find Total Diaphragm Force - (E/W)

Roof Diaphragm: Wall height varies from 8ft to 14ft

$q_r := p_{56} \cdot 6.5\ ft = 148\ plf$
$q_{rE} := p_{56E} \cdot 3\ ft = 103\ plf$

$F_{rw} := q_{rE} \cdot a + q_r \cdot \left(26\ ft - a\right) = 3.7\ kip$ < $E_{h\_roof} = 11.6\ kip$ SEISMIC GOVERN

Floor Diaphragm (Ignore Roof Loads)

$q_{fl} := p_{56} \cdot 9\ ft = 205\ plf$
$q_{flE} := p_{56E} \cdot 9\ ft = 309\ plf$

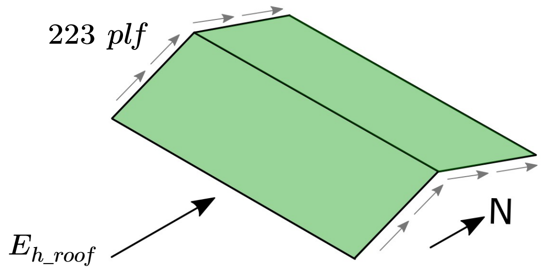
$F_{2w} := q_{flE} \cdot a + q_{fl} \cdot \left(26\ ft - a\right) = 5.7\ kip$ < $E_{h\_2} = 5.7\ kip$ SEISMIC GOVERNS

Largest Wall Load:

$p_{5E} := q_h \cdot K_d \cdot \left(GC_{pf\_5E} + GC_{pi}\right) = 38.3\ psf$ (Ultimate Level)

$p_{5Ea} := 0.6\ q_h \cdot K_d \cdot \left(GC_{pf\_5E} + GC_{pi}\right) = 23.0\ psf$ (Service Level)

Determine Upper Roof Diaphragm Forces

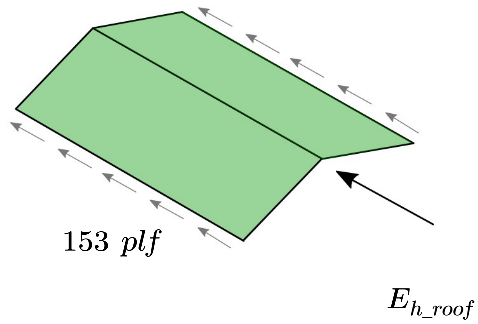


$E_{h\_roof} = 11.6\ kip$

North / South Direction

Roof Diaphragm Shear:

$v_{roof1} := \dfrac{E_{h\_roof}}{2 \cdot 26\ ft} = 223\ plf$ (case 1)



East / West Direction

Roof Diaphragm Shear:

$v_{roof2} := \dfrac{E_{h\_roof}}{2 \cdot 38\ ft} = 153\ plf$ (case 3)

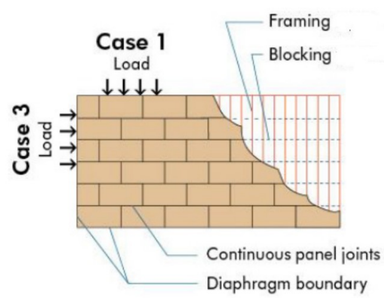
Roof Diaphragm Design

SDPWS 2021 Table 4.2C

Unblocked Wood Structural Panel Diaphragms[1,2,3,4,6]

Sheathing Grade	Common Nail Size[5] Length (in.) x Shank diameter (in.) x Head diameter (in.)	Minimum Nail Bearing Length in Framing Member, ℓ_m (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	6 in. Nail Spacing at diaphragm boundaries and supported panel edges					
					Case 1			Cases 2,3,4,5,6		
					v_n (plf)	G_a (kips/in)		v_n (plf)	G_a (kips/in)	
						OSB	PLY		OSB	PLY
Structural I	6d (2 x 0.113 x 0.266)	1-1/4	5/16	2	460	9.0	7.0	350	6.0	4.5
				3	520	7.0	6.0	390	4.5	4.0
	8d (2-1/2 x 0.131 x 0.281)	1-3/8	3/8	2	670	8.5	7.0	505	6.0	4.5
				3	740	7.5	6.0	560	5.0	4.0
	10d (3 x 0.148 x 0.312)	1-1/2	15/32	2	800	14	10	600	9.5	7.0
				3	895	12	9.0	670	8.0	6.0

Out-of-plane Loads: $p_s = 38.5\ psf$ (Service)



Assume 24" o.c. Supports (trusses)

Use 32/16 OSB APA Rated Sheathing
75 psf for L/240 at 24"o.c. (APA Q225 p. 6)
100 psf for L/180 at 24"o.c.

32/16 is **15/32 Sheathing** (see Table 11 of APA D510)

Check roof aspect ratio: $\dfrac{26\ ft}{38\ ft} = 0.68$ < 3.0, SDPWS 2021 Table 4.2.2

Use 8d nails at 6" o.c. at all panel edges with 2" supports (unblocked)

From SDPWS 4.1.4: $\phi := 0.50$

$v_{n1} := 670\ plf$ $\quad v_{n3} := 505\ plf$

$\phi v_{n1} := \phi \cdot v_{n1} = 335\ plf$ > $v_{roof2} = 153\ plf$ OK

$\phi v_{n3} := \phi \cdot v_{n3} = 253\ plf$ > $v_{roof1} = 223\ plf$ OK

Roof Chord Design

$$w := \frac{E_{h\_roof}}{38\ ft} = 305\ plf$$

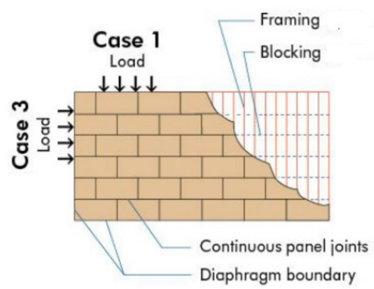
$$M_{roof} := \frac{w \cdot (38\ ft)^2}{8} = 55\ kip \cdot ft$$

$$T_{roof} := \frac{M_{roof}}{24\ ft} = 2.3\ kip$$

$\dfrac{T_{roof}}{279\ lbf} = 8.2$ nails

Ultimate strength of 10d nails in Doug-Fir that is part of a diaphragm ($C_{di} := 1.1$)

Floor Diaphragm Design



Out-of-plane Loads: $p_{live} := 40\ psf$

Assume 19.2" o.c. Supports (joists)

Use 24/16 OSB APA Rated Sheathing
70 psf for L/360 at 19.2"o.c. (APA Q225 p. 6)

24/16 is 7/16 Sheathing (see Table 11 of APA D510)

Use 8d nails at 6" o.c. at all panel edges with 2" wide supports (unblocked)

Diaphragm Loads

North/South (case 1): $v_{floor2} := \dfrac{F_2}{2 \cdot 26\ ft} = 110\ plf$

East/West (case 3): $v_{floor1} := \dfrac{E_{h\_2}}{2 \cdot 38\ ft} = 75\ plf$

From SDPWS 4.1.4: $\phi := 0.50$

$v_{n1} := 670\ plf$ $\quad v_{n3} := 505\ plf$

$\phi v_{n1} := \phi \cdot v_{n1} = 335\ plf$ > $v_{floor2} = 110\ plf$ OK

$\phi v_{n3} := \phi \cdot v_{n3} = 253\ plf$ > $v_{floor1} = 75\ plf$ OK

Floor Chord Design

$$w := \frac{F_2}{38\ ft} = 151\ plf$$

$$M_{floor} := \frac{w \cdot (38\ ft)^2}{8} = 27\ kip \cdot ft$$

$$T_{floor} := \frac{M_{floor}}{25\ ft} = 1.1\ kip$$

$$\frac{T_{floor}}{279\ lbf} = 3.9$$ nails

Ultimate strength of 10d nails in Doug-Fir that is part of a diaphragm ($C_{di} := 1.1$)

Determine Shear Wall Forces:

Out-of-plane Loads: $p_{wall1} = 44.5\ psf$

Assume studs are 16" o.c.

Use 32/16 OSB APA Rated Sheathing
61 psf for L/240 at 16"o.c. (APA Q225 p. 6)

32/16 is 15/32 Sheathing (see Table 11 of APA D510)

Span Rating(b)	Load Governed By(c)	Parallel to Supports Span Center-to-Center of Supports (inches) 12	16	24
24/0	L/360	48	18	
	L/240	72	27	
	L/180	96	36	
	Bending	81	45	
	Shear	248	179	
32/16	L/360	109	41	14
	L/240	163	61	21
	L/180	218	82	28
	Bending	138	77	28
	Shear	314	228	141

Table 4.3A

Wood-based Panels [4]

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Nail Bearing Length in Framing Member or Blocking, ℓ_m (in.)	Nail Type & Size [9] Length (in.) x Shank diameter (in.) x Head diameter (in.)	Panel Edge Nail Spacing (in.) 6 v_n (plf)	6 G_a (kips/in.) OSB	6 G_a PLY	4 v_n (plf)	4 G_a OSB	4 G_a PLY	3 v_n (plf)	3 G_a OSB	3 G_a PLY	2 v_n (plf)	2 G_a OSB	2 G_a PLY
Wood Structural Panels - Structural I [4,5]	5/16	1-1/4	6d common nail (2 x 0.113 x 0.266) [8]	560	13	10	840	18	13	1090	23	16	1430	35	22
	3/8 [2]	1-3/8	8d common nail (2-1/2 x 0.131 x 0.281) [8]	645	19	14	1010	24	17	1290	30	20	1710	43	24
	7/16 [2]			715	16	13	1105	21	16	1415	27	19	1875	40	24
	15/32			785	14	11	1205	18	14	1540	24	17	2045	37	23
	15/32	1-1/2	10d common nail (3 x 0.148 x 0.312) [8,10]	950	22	16	1430	29	20	1860	36	22	2435	51	28
Wood Structural Panels - Sheathing [4,5]	5/16	1-1/4	6d common nail (2 x 0.113 x 0.266) [8]	505	13	9.5	755	18	12	980	24	14	1260	37	18
	3/8			560	11	8.5	840	15	11	1090	20	13	1430	32	17
	3/8 [2]	1-3/8	8d common nail (2-1/2 x 0.131 x 0.281) [8]	615	17	12	895	25	15	1150	31	17	1485	45	20
	7/16 [2]			670	15	11	980	22	14	1260	28	17	1640	42	21
	15/32			730	13	10	1065	19	13	1370	25	15	1790	39	20
	15/32	1-1/2	10d common nail (3 x 0.148 x 0.312) [8,10]	870	22	14	1290	30	17	1680	37	19	2155	52	23
	19/32			950	19	13	1430	26	16	1860	33	18	2435	48	22
Plywood Siding	5/16	1-1/4	6d galv.[7] casing nail (2 x 0.099 x 0.142)	390	13		590	16		770	17		1010	21	
	3/8	1-3/8	8d galv.[7] casing nail (2-1/2 x 0.113 x 0.155)	450	16		670	18		870	20		1150	22	

8d nails in OSB at 3" o.c. at edges with studs 16" o.c. (footnote 2):

$v_{n3} := 1370\ plf \qquad G_{a3} := 25\ \frac{kip}{in}$

Perforated Shear Wall Strength:

Use the Perforated Wall method to determine the wall shear and hold-down tension.

$V_n = v_n\, C_o\, \Sigma b_i$ (4.3-5)

where:

v_n = nominal unit shear capacity, lbs/ft

C_o = shear capacity adjustment factor from Table 4.3.5.6 or calculated using the following equation:

$C_o = A_{wall} / (3A_o + A_{fhs}) \leq 1.0$ (4.3-6)

where:

A_{fhs} = total area sheathed with full-height sheathing, ft², regardless of whether individual wall segments meet the aspect ratio limits in 4.3.3.4.

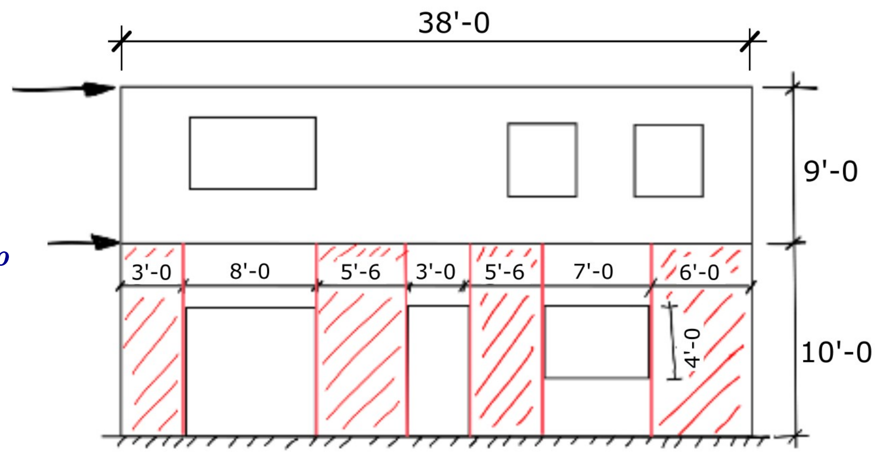
A_o = total area of openings in the perforated shear wall where individual opening areas are calculated as the opening width times the clear opening height, ft².

A_{wall} = total area of a perforated shear wall equal to the length of the perforated shear wall times its height, ft²

Σb_i = sum of perforated shear wall segment lengths b_i, ft. Lengths of perforated shear wall segments with aspect ratios greater than 2:1 shall be adjusted in accordance with 4.3.3.4.

$0.5\ F_r = 5.8\ kip$

$0.5\ F_2 = 2.9\ kip$



$3.0\ ft + 8\ ft + 5.5\ ft + 3\ ft + 5.5\ ft + 7\ ft + 6\ ft = 38\ ft$

South Wall - Level 1

$L_{tot} := 38\ ft \qquad h := 10\ ft \qquad \phi = 0.50$

$A_{wall} := L_{tot} \cdot h = 380\ ft^2$

$A_{fhs} := (3.0\ ft + 5.5\ ft + 5.5\ ft + 6\ ft) \cdot h = 200\ ft^2$

$A_o := 8\ ft \cdot 8\ ft + 3\ ft \cdot 7\ ft + 7\ ft \cdot 4\ ft = 113\ ft^2$

$$C_o := min\left(\frac{A_{wall}}{(3 \cdot A_o + A_{fhs})}, 1.0\right) = 0.71$$

Check Segment Aspect Ratios (4.3.3.4):

$b_1 := 3.0\ ft$ Smallest full-height wall segments

$\frac{h}{b_1} = 3.3$ > 2.0 must be reduced (unblocked wood structural panel)
< 3.5 - Can be included in b_i

$\frac{2 \cdot b_1}{h} \cdot b_1 = 1.8\ ft$

$$\Sigma b_i := 1.8\ ft + 5.5\ ft + 5.5\ ft + 6\ ft = 19\ ft$$

Use 8d nails at 3"o.c. on one side of wall:

$$\phi V_n := \phi \cdot v_{n3} \cdot C_o \cdot \Sigma b_i = 9.1\ kip \qquad > \qquad 0.5 \cdot V_2 = 8.7\ kip$$

Find Strength Ratio:

$$\phi V_{n8} := \phi \cdot v_{n3} \cdot 8\ ft = 5.5\ kip$$ 8 foot wall

%Strength of 3 x 8 ft walls: $\dfrac{\phi V_n}{3 \cdot \phi V_{n8}} = 0.55$

Hold-down Uplift:

$$T := \frac{0.5\ \left(E_{h\_roof} \cdot 19\ ft + E_{h\_roof} \cdot 9\ ft\right)}{1.43\ C_o \cdot \Sigma b_i} = 8.6\ kip$$ SDPWS (4.3-8)

Per general notes, multiply allowable loads in catalog by 1.43 to get LRFD values

Use HDU11-SDS2.5 with 4x6 Doug-Fir post. $T_{allowable} := 9.35\ kip$

Perforated Shear Wall Deflection:

From SDPWS 2021 - 4.3.4 - Deflection

$$v := \frac{0.5 \cdot V_2}{3 \cdot 8\ ft} = 361\ plf$$ Shear force applied to (3) 8ft walls

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

$h = 10\ ft$ $\quad b := 8\ ft$ $\quad G_{a3} = 25\ \dfrac{kip}{in}$

$E_{anc} := 40000\ \dfrac{lbf}{in}$ Vertical anchor stiffness based on string-pot measurement at end chords

$$\Delta_a := \frac{v \cdot b}{E_{anc}} = 0.072\ in$$

$E := 1400\ ksi$ End Post Modulus (No. 2 SPF)

$A := 3\ in \cdot 5.5\ in = 17\ in^2$ End Post Cross Section

$$\delta_{sw8} := \frac{8\ v \cdot h^3}{E \cdot A \cdot b} + \frac{v \cdot 8\ ft}{1000 \cdot G_{a3}} + \frac{8\ ft}{b} \cdot \Delta_a = 0.26\ in$$ 8x8 foot wall deflection

$$\frac{8\ v \cdot h^3}{E \cdot A \cdot b} = 0.188\ in \qquad \frac{v \cdot 8\ ft}{1000 \cdot G_{a3}} = 0.0001\ in \qquad \frac{h \cdot v}{E_{anc}} = 0.090\ in$$

Per SDPWS 2021 - 4.3.4.2

$$v_{max} = \frac{V}{C_o \sum b_i} \qquad (4.3\text{-}9)$$

$C_o \cdot \Sigma b_i = 13\ ft$

$0.5 \cdot V_2 = 8669\ lbf$

$$v_{max} := \frac{0.5 \cdot V_2}{C_o \cdot \Sigma b_i} = 654\ plf$$

$$\delta_{sw} := \frac{8\ v_{max} \cdot h^3}{E \cdot A \cdot \Sigma b_i} + \frac{v_{max} \cdot h}{1000 \cdot G_{a3}} + \frac{h \cdot v_{max}}{E_{anc}} = 0.308\ in$$

Perforated wall deflection

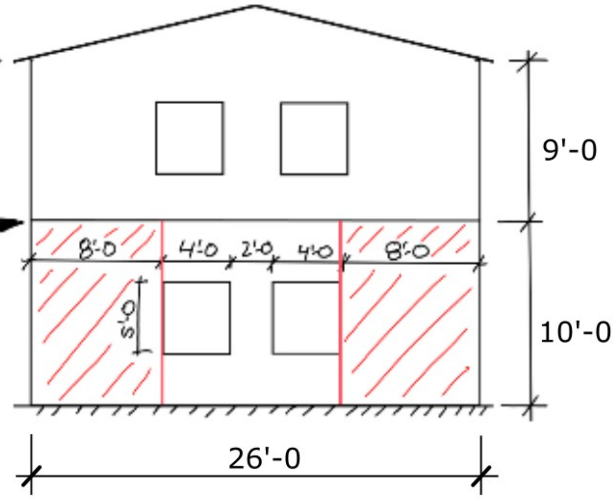
$$\frac{8\ v_{max} \cdot h^3}{E \cdot A \cdot \Sigma b_i} = 0.145\ in \qquad \frac{v_{max} \cdot h}{1000 \cdot G_{a3}} = 0.000\ in \qquad \frac{v_{max} \cdot h}{E_{anc}} = 0.164\ in$$

Find Stiffness Ratio:

% Stiffness of (3) 8 ft walls: $\frac{\delta_{sw}}{\delta_{sw8}} = 1.19$

East Wall - Level 1

$0.5\ F_r = 5.8\ kip$



$0.5\ F_2 = 2.9\ kip$

$L_{tot} := 26\ ft$

$h := 10\ ft$

$A_{wall} := L_{tot} \cdot h = 260\ ft^2$

$A_{fhs} := 16\ ft \cdot h = 160\ ft^2$

$A_o := 2 \cdot 4\ ft \cdot 5\ ft = 40\ ft^2$

$$C_o := min\left(\frac{A_{wall}}{(3 \cdot A_o + A_{fhs})}, 1.0\right) = 0.93$$

Check Segment Aspect Ratios (4.3.3.4):

$b_1 := 8\ ft$ Smallest full-height wall segments

$\frac{h}{b_1} = 1.3$ < 2.0 for unblocked wood structural panel walls, no adjustment

$\Sigma b_i := 8\ ft + 8\ ft = 16\ ft$

$\phi V_n := \phi \cdot v_{n3} \cdot C_o \cdot \Sigma b_i = 10.2\ kip$ > $0.5 \cdot V_2 = 8.7\ kip$

Use 8d nails at 3"o.c. on one side of wall

Find Strength Ratio:

$\phi V_{n8} := \phi \cdot v_{n3} \cdot 8\ ft = 5.5\ kip$ 8 foot wall

%Strength of 2 x 8 ft walls: $\frac{\phi V_n}{2 \cdot \phi V_{n8}} = 0.929$

Hold-down Uplift:

$$T := \frac{0.5\ \left(E_{h\_roof} \cdot 19\ ft + E_{h\_roof} \cdot 9\ ft\right)}{1.43\ C_o \cdot \Sigma b_i} = 7.6\ kip$$ SDPWS (4.3-8)

Per general notes, multiply allowable loads in catalog by 1.43 to get LRFD values

Use HDU11-SDS2.5 with 4x6 Doug-Fir post. $T_{allowable} := 9.35\ kip$

Perforated Shear Wall Deflection:

From SDPWS 2021 - 4.3.4 - Deflection

$$v := \frac{0.5 \cdot V_2}{2 \cdot 8\ ft} = 542\ plf$$

Shear force applied to 2x8ft walls

$$\delta_{sw8} := \frac{8\ v \cdot h^3}{E \cdot A \cdot b} + \frac{v \cdot 8\ ft}{1000 \cdot G_{a3}} + \frac{h \cdot v}{E_{anc}} = 0.417\ in$$

8x8 foot wall deflection

Per SDPWS 2021 - 4.3.4.2

$$v_{perf} := \frac{0.5 \cdot V_2}{C_o \cdot \Sigma b_i} = 584\ plf \qquad \text{(eqn 4.3-9)}$$

$$\delta_{sw} := \frac{8\ v_{perf} \cdot h^3}{E \cdot A \cdot \Sigma b_i} + \frac{v_{perf} \cdot h}{1000 \cdot G_{a3}} + \frac{h \cdot v_{perf}}{E_{anc}} = 0.298\ in$$

Perforated wall deflection

Find Stiffness Ratio:

%Stiffness of 8 ft wall: $\frac{\delta_{sw8}}{\delta_{sw}} = 1.40$

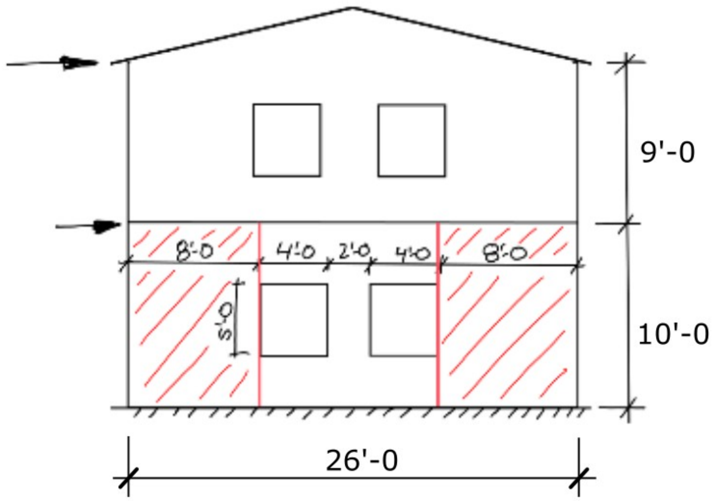
East Wall - Level 1

Two Wall Segments (not perforated), 4 holddowns

$b := 8\ ft$

$h := 10\ ft$

$\frac{h}{b} = 1.3$ < 2.0 for unblocked wood structural panel walls



$\phi V_n := \phi \cdot v_{n3} \cdot 2 \cdot b = 11.0\ kip$ > $0.5 \cdot V_2 = 8.7\ kip$

Use 8d nails at 3"o.c. on one side of wall

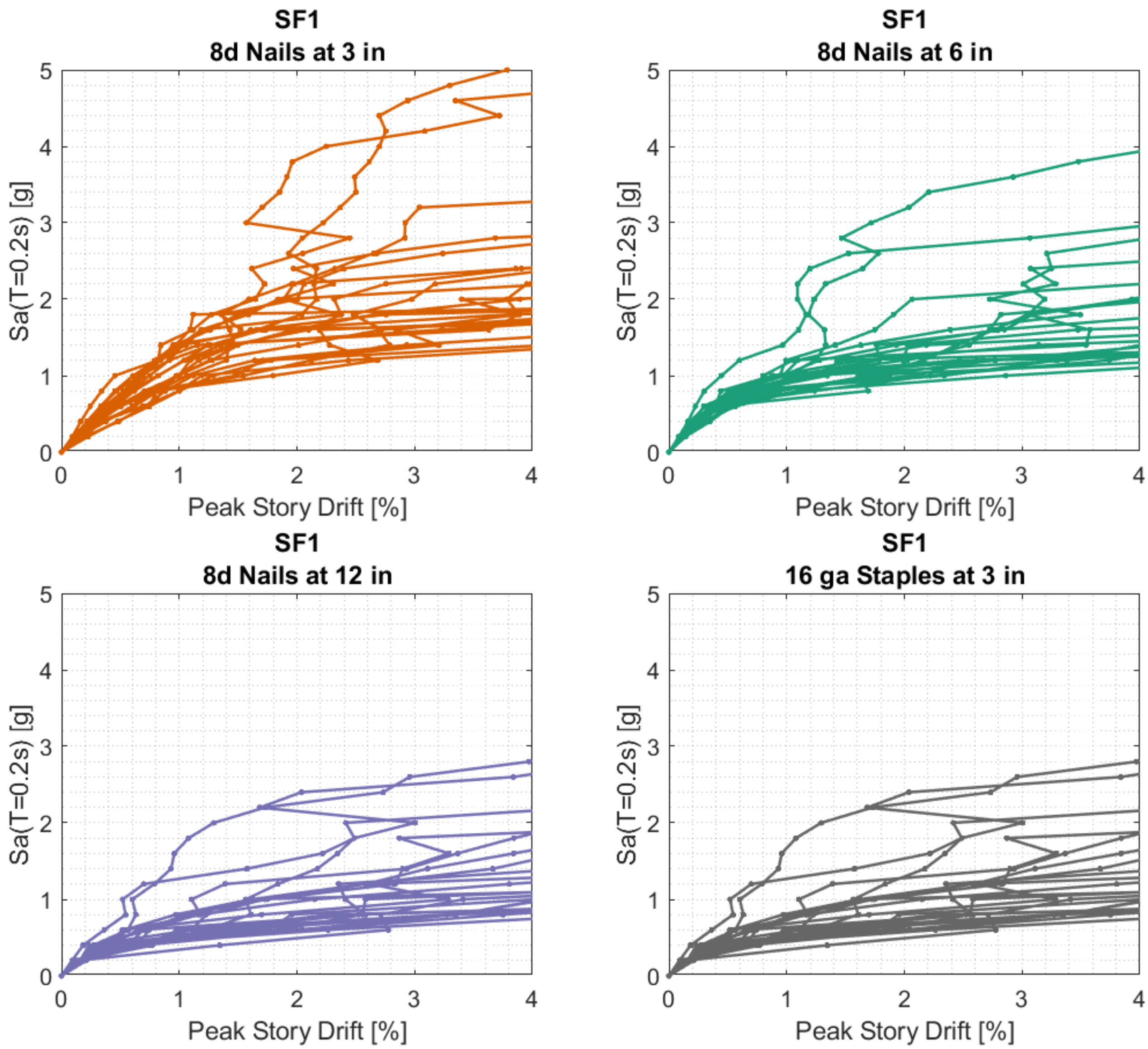
Hold-down Uplift:

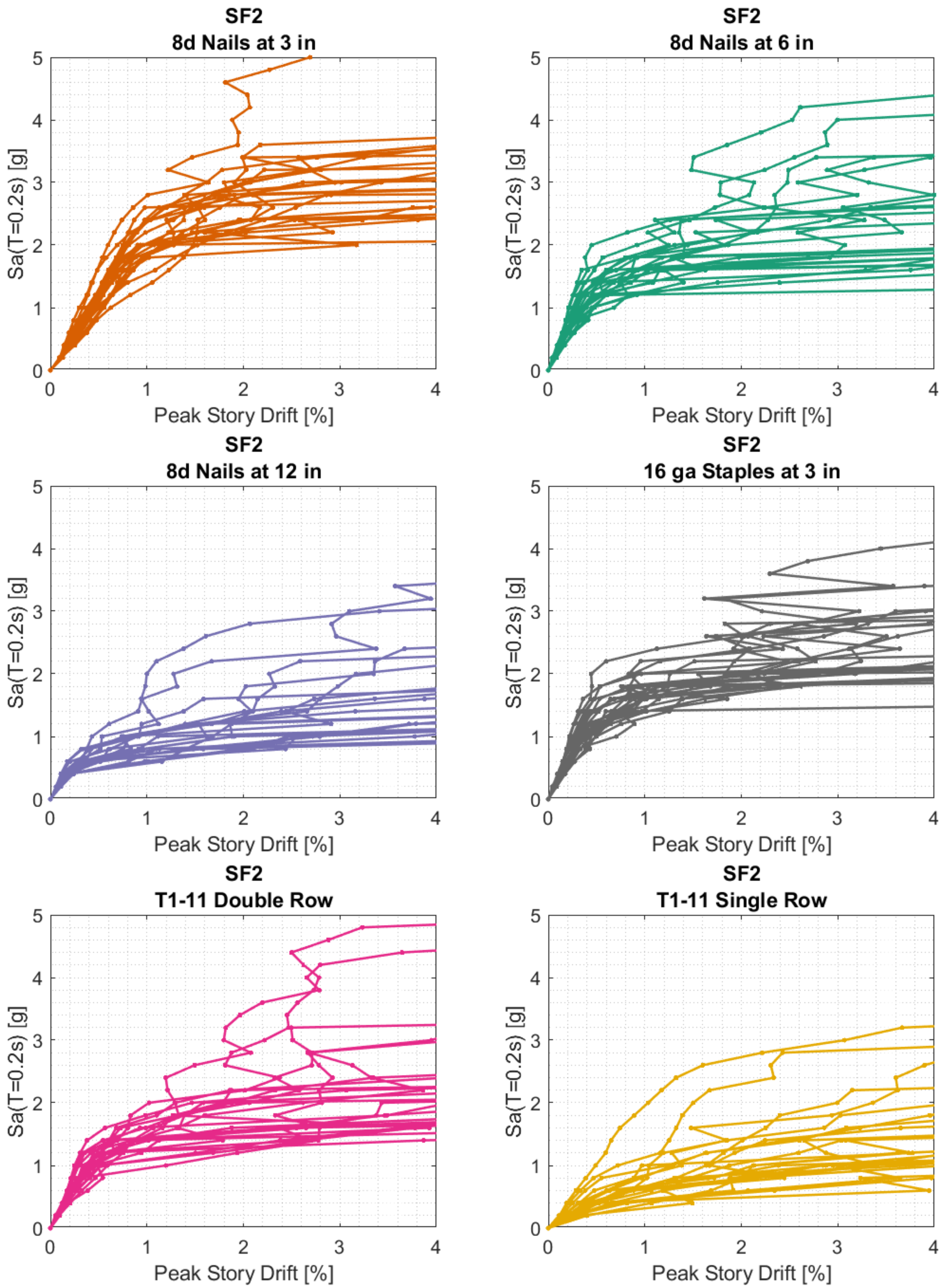
$$T := \frac{0.5 \cdot 0.5 \left(E_{h\_roof} \cdot 18\ ft + E_{h\_roof} \cdot 9\ ft\right)}{1.43\ C_o \cdot \Sigma b_i} = 3.7\ kip$$ SDPWS (4.3-8)

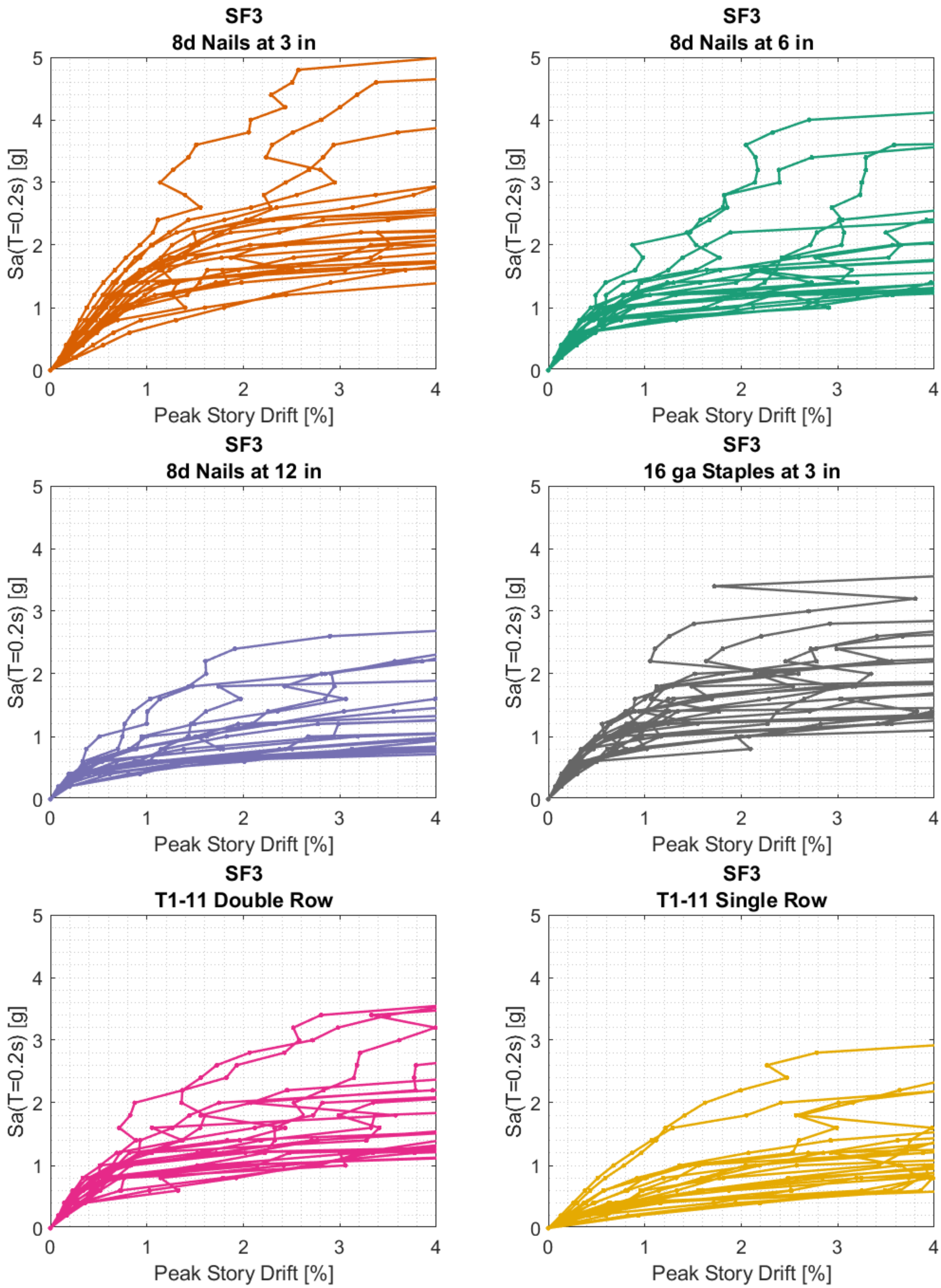
Per general notes, multiply allowable loads in catalog by 1.43 to get LRFD values

Use HDU8-SDS2.5 with (2) 2x6 SPF post. $T_{allowable} := 5.8\ kip$

Appendix F: Incremental Dynamic Analysis Output Plots


SF1
8d Nails at 3 in
SF1
8d Nails at 6 in
SF1
8d Nails at 12 in
SF1
16 ga Staples at 3 in
Sa(T=0.2s) [g]
Peak Story Drift [%]
0
1
2
3
4
5


SF2
8d Nails at 3 in
SF2
8d Nails at 6 in
SF2
8d Nails at 12 in
SF2
16 ga Staples at 3 in
SF2
T1-11 Double Row
SF2
T1-11 Single Row
Sa(T=0.2s) [g]
Peak Story Drift [%]
0
1
2
3
4
5


SF3
8d Nails at 3 in
SF3
8d Nails at 6 in
SF3
8d Nails at 12 in
SF3
16 ga Staples at 3 in
SF3
T1-11 Double Row
SF3
T1-11 Single Row
Sa(T=0.2s) [g]
Peak Story Drift [%]
0
1
2
3
4
5

Appendix G: Workshop Flyer

UAA College of Engineering
UNIVERSITY of ALASKA ANCHORAGE

FEMA

Reducing Seismic Risk in Residential Construction

Friday, December 2, 2022 | 8 am – 12 pm

UAA Engineering and Industry Building Room 217

Breakfast Included!

Earn CEUs!

Presenters:
Scott Hamel, PE, PhD
Polly Murray, PhD

Workshop Description

We will conduct a lab demonstration of a full-scale wall under cyclic racking load, present results from our study of *seismic risk* associated with incorrectly designed and/or constructed wood frame houses; provide instruction on proper seismic house detailing; and discuss which structural elements have the biggest effect on seismic risk. This study was motivated by the Mw 7.1 earthquake in southcentral Alaska on November 30, 2018, which caused more severe and widespread damage in residential structures *outside* of the Building Safety Service Area, where building codes are not enforced.

Workshop Topics

- Lab demonstration of full-scale wall
- Structural systems in residential construction
- Impact of design and construction decisions on seismic risk

Register Here:
https://forms.gle/u2am9N2cpePfmCd27

Questions?
Email Polly Murray at bbmurray@alaska.edu