

Agenda Item VII.A.

State of Oregon

Board memo

Building Codes Division

February 3, 2010

To: Building Codes Structures Board

From: Patrick Allen
Administrator, Oregon Building Codes Division

Subject: Board Concurrence – Oregon amendment relative to anchorage of bolts in concrete supporting wood plate sills/walls. IBC 1908.1.16 [ACI 318 D3.3.5].

Action requested:

A board motion noting concurrence with the hearings officer's findings regarding adoption of code requirements specific to anchorage of bolts in concrete supporting wood plate sills/walls. IBC 1908.1.16 [ACI 318 D3.3.5].

In testimony before the public hearings officer, *Mr. Vandehey* of the Structural Engineers Association of Oregon (SEAO) spoke to recent research and testing conducted by the Structural Engineers Association of California (SEAOC) on *the anchorage of bolts in concrete supporting wood plate sills/walls*. In considering adoption of the 2010 Oregon Structural Specialty Code (OSSC), SEAO is requesting consideration of an additional amendment based on the following narrative. The hearings officer is recommending adoption of the same. Accordingly, I am requesting that the Building Codes Structures Board consider for adoption the Oregon amendments noted below.

Background:

SEAO became aware of the SEAOC testing well after Building Code Division (BCD) had commenced with the 2010 OSSC code development process.

SEAOC testing results have shown that Appendix D of ACI 318, as referenced by the OSSC, is unduly conservative by the application of several factors. Two sensitive assumptions that affect the ACI Appendix D calculation are; 1) the ductility parameter and 2) the cracked concrete parameter.

In application, a fairly heavily loaded shear wall that would have traditionally required two anchors per stud bay now requires eight anchors per stud bay, which physically do not fit. The SEAOC test data show that the yield strength of the wood sill plate connection governs over the strength of the concrete in the subject connections.

This component testing was necessary to determine the specifics of the connection behavior, particularly the large amount of yielding the bolts achieve above the concrete surface and the beneficial clamping effect due to the square plate washer.

The ductility parameter of IBC 1908.1.16 [ACI 318 D3.3.5] alone requires **a 60 percent reduction** to the connection capacity in concrete if the attachment to concrete is not ductile at the concrete design strength. The resultant low concrete capacity values indicate that a failure of the connection is expected to occur in the concrete long before it occurs in the anchor bolt or the wood sill plate, which is counter-intuitive.

In light of their research, SEAOC has submitted a code change to the State of California in conjunction with that state's adoption of the 2009 International Building Code (IBC). ***On behalf of SEAOC, Mr. Vandehey is requesting that the State of Oregon pursue a similar code changes as noted below.*** The changes mirror those recently approved by the International Codes Council's (ICC), Structural Committee for inclusion in the *2012 International Building Code*.

The ICC Structural Committee's amendment justification states in part:

“This proposal revises the determination of anchor bolt capacity under Appendix D of ACI 318, in recognition that both lab tests and field experience show that failure of the wood sill plate controls the capacity. In these instances, there is no need for laborious concrete strength calculations. It also reformats the proposal as new Exception 3 and places the sill plate anchor details in new Section 2305.1.2.” (et al)

In support of his amendment request, Mr. Vandehey submitted two reference documents:

1. *“Report on laboratory testing of anchor bolts connecting wood sill plates to concrete with minimum edge distances.”* Issued by SEAOC.
2. *“SEAOC Blue Book – Seismic Design Recommendations – Anchor Bolts in Light-Frame Construction at Small Edge Distances.”* Issued by SEAOC.

Both documents are attached for reference.

PROPOSED OREGON AMENDMENTS TO THE 2010 OSSC

Add the following modifications to the 2010 OSSC (2009 IBC):

1908.1.9 ACI 318, Section D.3.3. Modify ACI 318, Sections D3.3.4 and D3.3.5, to read as follows:

D.3.3.4 – Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with D.5.1 and D.6.1, unless either D.3.3.5 or D.3.3.6 is satisfied.

Exceptions:

1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.4.

2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.4.

3. In light-frame wood structure bearing or non-bearing walls, for the design of anchors used to attach wood sill plates to foundations or foundation stem walls, it shall be permitted to take the allowable in-plane shear strength of the anchors in accordance with Section 2305.1.2 of the International Building Code.

D.3.3.5 – Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.3.

Exceptions:

1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5.

2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.5.

Add new section as follows to OSSC 2305 GENERAL DESIGN OF REQUIREMENTS FOR LATERAL-FORCE-RESISTING SYSTEMS:

2305.1.2 Sill plate anchor bolts. For sill plates of 2x or 3x nominal thickness, the allowable lateral design for shear parallel to the grain of sill plate anchor bolts is permitted to be determined using the lateral design value for a bolt attaching a wood sill plate to concrete, as specified in AF&PA NDS Table 11E, provided the anchor bolts comply with all of the following:

1. The maximum anchor nominal diameter is 5/8 inches (16 mm);

2. Anchors are embedded into concrete a minimum of 7 inches (178 mm);

3. Anchors are located a minimum of 1-3/4 inches (45 mm) from the edge of the concrete parallel to the length of the wood sill plate; and

4. Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the wood sill plate.

Options:

- Pass a motion concurring with the hearings officer's recommendation, with the finding that the added cost, if any, is necessary to the health and safety of the occupants or the public or necessary to conserve scarce resources.
- Suggest additional code language and pass a motion with the finding that the added cost, if any, is necessary to the health and safety of the occupants or the public or necessary to conserve scarce resources.

Recommendation:

Pass a motion concurring with the Hearings Officer's recommendation, with the finding that the added cost, if any, is necessary to the health and safety of the occupants or the public or necessary to conserve scarce resources.

Report on laboratory testing of anchor bolts connecting wood sill plates to concrete with minimum edge distances

W. Andrew Fennell, CE, SECB, CPEng.
Scientific Construction Laboratories, Inc.

Kevin S. Moore, CE, SE.
Certus Consulting, Inc.

Philip Line, PE
American Wood Council / AF&PA

Thomas D. Van Dorpe, CE, SE, CBO.
Van Dorpe, Chou and Associates, Inc.

Gary L. Mochizuki, CE, SE.
Structural Solutions, Inc.

Thomas A. Voss, CE.
Scientific Construction Laboratories, Inc.

Abstract:

The 2006 International Building Code (IBC-06) is the “Model Code” for the 2007 California Building Code (CBC-07). IBC-06 references ACI 318-05 Appendix D for the determination of anchor bolt capacity (in single-shear) when attaching wood sill plates to concrete foundations. Many practicing engineers and building officials are currently mystified by the low anchor bolt capacities obtained from the application of Appendix D equations for wood framed construction in seismic design categories D, E and F.

In the absence of available test data, members of the 2008-2009 SEAOC Seismology Committee undertook a study of typical anchor bolted connections to establish a basis for evaluating design capacities while better understanding the behavior of this basic connection.

Initial experiments aided the development of the test set-up and protocol. The main tests were performed at the Tyrell Gilb Research Laboratory in Stockton California. All tests were single-bolt tests in wood sill plates connected to concrete with standard cast-in-place steel anchor bolts. A total of 28 tests were performed; twenty-four primary tests and four “bonus” tests. All tests were conducted with the load applied parallel to the free edge of the concrete. Additional testing was performed concurrently on the concrete to detect flaws and delaminations (if any) that may have formed.

The test program showed that the wood sill plate to concrete connection using cast-in-place steel anchor bolts is ductile and that design capacities (both past and present are conservative). The program has also served as a basis for additional testing programs and publications.

Introduction:

Seismic force resisting systems for wood framed buildings typically comprise wood structural panel shear walls with anchor bolts located at the edge of foundations. These connections often have specified edge distance of 1-3/4" from the bolt center line to the face of the concrete slab or footing.

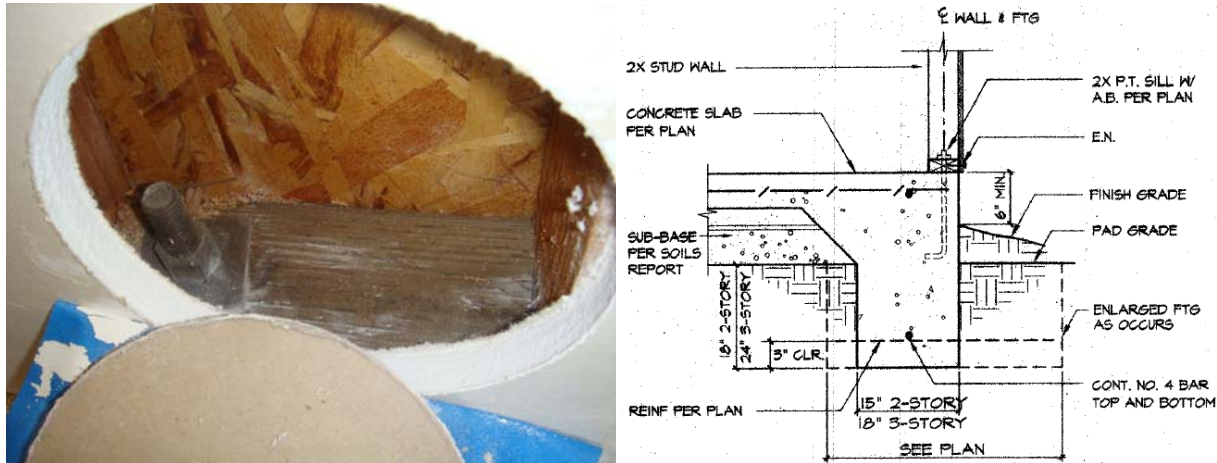


Image 1 – View of an “as-built” anchor bolt installation per plan detail with minimum edge distance of approximately 1-3/4”. Anchor shown is 5/8” nominal diameter with a 2”x2” plate washer per CBC-2001. Note CBC-2007 requires 3”x3”x0.229” plate washer.

Engineers have historically anticipated the controlling failure of this connection to occur between the anchor bolts and the wood sill plate. However, design capacities for break-out strength in shear determined in accordance with ACI 318-05 Appendix D are greatly reduced and less than the wood to concrete connection design capacity for small edge distances. ACI 318-05 provides benefit to break-out design capacity where connections are ductile but application of “ductile” provisions to the wood to concrete connection is not clearly defined within ACI 318-05.

Lacking specific test data to substantiate the reduced design capacities for anchors in concrete in a typical wood to concrete connection loaded parallel to the edge, SEAOC’s Seismology Committee initiated a project with the following primary goals:

- Establish test data for the connection capacity when loaded parallel to the edge.
- Determine whether the connection exhibits “ductile” behavior.
- Propose rational design capacities for the connection based on test results.

All tests were single-bolt tests in wood sill plates connected to concrete with standard cast-in-place steel anchor bolts. A total of 28 tests were performed; 24 primary tests and four “bonus” tests. Additional testing was performed concurrently to detect flaws and delaminations (if any) that may have formed within the concrete during the testing.

Test Specimens:

Figure 1 shows a typical cross section of test specimens specified. Material properties are tabulated below. Material properties for each test are included in **Table-1 (primary tests)** and **Table-2 (“bonus” tests)**.

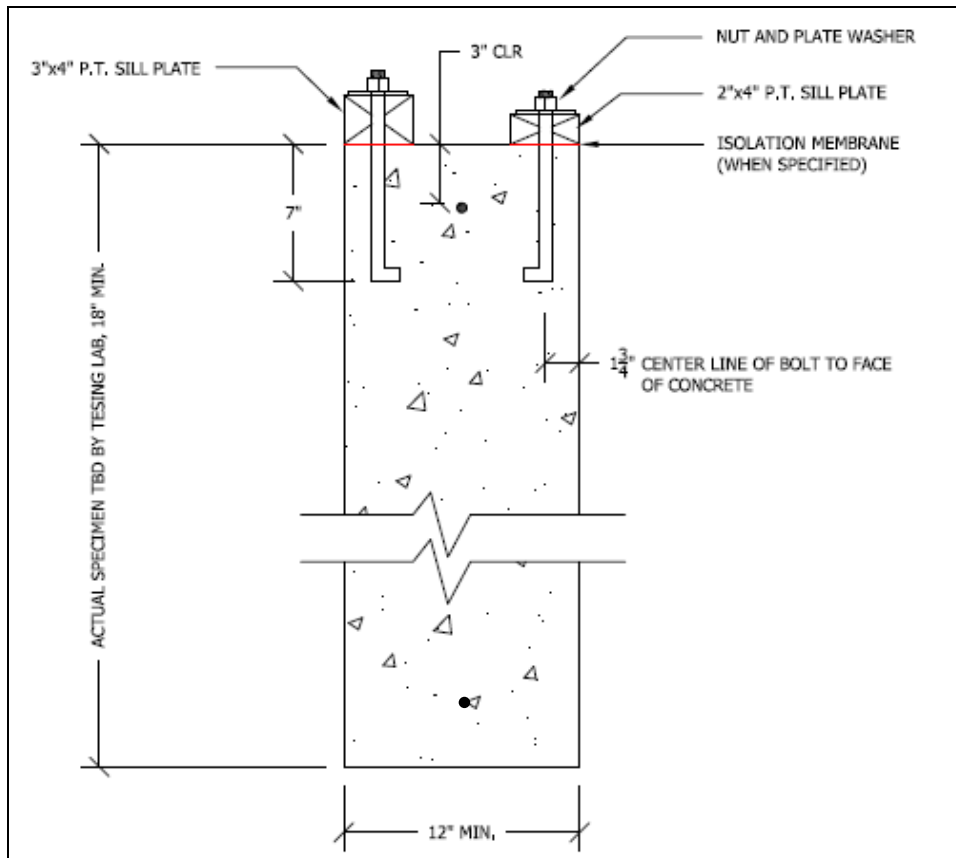


Figure 1 – Typical section for test specimens. Not to scale.

The anchors tested (parallel to the concrete face) were cast-in-place in two rows as shown in **Figure 1**. Interaction between the two rows of anchors and effects on test results was shown to be a non-issue during initial experiments with anchors cast down each side of a narrow concrete section.

For all tests, nominal bolt diameter was 5/8” and concrete compressive strength was between 2500psi and 3000psi. All concrete specimens were tested in the “uncracked” condition. Wood sill plates were 2x4, 3x4, 2x6 and 3x6 Douglas fir, incised and preservative treated. Anchor bolts centered in the wide face of 4x nominal width and 6x nominal width members resulted in target edge distances (measured from center-line of the anchor to the face of the concrete foundation) of 1-3/4” and 2-3/4”, respectively.

Component descriptions follow:

Concrete – a compressive strength (f'_c) of 2500psi to 3000psi was specified for the tests to represent typical light-frame construction. Test cylinders for f'_c ranged from 2550 psi (on 11/12/08) to 2710psi (on 11/19/08). Modulus of Elasticity (MOE) was measured on 11/11/08 at SCL and reported as 3.61×10^6 psi.

A single 60ksi #4 reinforcing bar was run top and bottom as shown in **Figure 1**. The reinforcement was placed 3" from the top and bottom and located centrally in the 12" wide test specimen.

Wood sill plates – were of nominal 2x4, 3x4, 2x6 and 3x6 sizes. All material was pressure preservative treated (PT). The following properties for each specimen are reported in **Table 1**; lumber species, lumber grade, moisture content and preservative treatment. Sill plate stock was tested in "as received" condition. The material procured was specified as "PT, DF, #2 or better".

Anchor bolts – were bare steel ASTM A307 L-bolts, 5/8" nominal diameter (0.559" actual) with rolled threads. Shear stress, F_y , for the bolts was determined by test and found to be 40ksi. Plate washers were 3"x3" square by 0.229" thick with a 3/4" diameter hole (not slotted).

Embedment – anchor bolts were embedded 7" and were held in place by bolt holders during casting. No reinforcement was placed at the bolt locations.

Edge distance – minimum edge distance of 1-3/4" was specified for 2x4 and 3x4 tests. For tests on 2x6 and 3x6 material, a 2-3/4" edge distance was specified. Actual clear cover measurements were taken with a pachometer after the forms were removed. See **Table 1** for specified (and actual) edge distances reported per test.

Bolt hole – the wood sill plate hole for the anchor bolt was oversized by a nominal 1/16". Unless otherwise noted in **Table 1**, each test was run with 11/16" bored hole. The hole was centered on the wide face of the wood sill plate.

Anchor bolt nut tightness – All tests (except 1 bonus test) were run with the nut "finger tight + 1/4 turn". This was thought to be typical of an in-service condition where the sill plate has undergone some dimensional change (due to changes in moisture content).

Membrane – an isolation membrane was installed as noted in Table 1. The membrane comprised two layers of 10-mil polyethylene sheeting (0.010"). Lithium grease was sprayed between the two plies to approximate an idealized "frictionless" plane. Tests utilizing the membrane are designed "nf" for non-friction, (i.e. 1-D-1-nf). The effect of friction was evaluated for 2x4 and 3x4 sill plates where the specified edge distance was 1-3/4".

Test ID's Test Date	Plate Test. UON 5/8" A/B (0.559") 3"x3"x0.229" washer	Edge Distance: Nominal, Actual.	Loading Protocol	Moisture Content, PT Lumber species & grade.
1-A-1-f 289 11/12/08	2x4 sill plate.	1.75" 1.9"	Monotonic 250#/sec.	9.1 % to 9.7 %, Borate. DF-Standard & Better.
1-A-2-f 290 11/12/08	2x4 sill plate.	1.75" 1.8"	Monotonic 250#/sec.	8.4 %, Borate. DF-Standard & Better.
2-A-1-f 293 11/12/08	2x4 sill plate.	1.75" 1.9"	Cyclic SEAOC @ 50#/sec.	7.9 % to 8.5 %, Borate. DF-Standard & Better.
2-A-2-f 294 11/12/08	2x4 sill plate.	1.75" 1.7"	Cyclic (0.2 Hz) SEAOC-Modified.	7.5 % to 8.1 %, Borate. DF-Standard & Better.
1-C-1-nf 291 11/12/08	2x4 sill plate.	1.75" 1.9"	Monotonic 250#/sec.	6.5 % to 8.1 %, Borate. DF-Standard & Better.
1-C-2-nf 292 11/12/08	2x4 sill plate.	1.75" 1.9"	Monotonic 250#/sec.	7.0 % to 8.3 %, Boarate. DF-Standard & Better.
2-C-1-nf 295 11/12/08	2x4 sill plate.	1.75" 1.8"	Cyclic (0.2 Hz) SEAOC-Modified.	9.1 % to 10.2 %, Borate. DF-Standard & Better.
2-C-2-nf 296 11/13/08	2x4 sill plate.	1.75" 1.9"	Cyclic (0.2 Hz) SEAOC-Modified.	7.5 % to 8.1 %, Borate. DF-Standard & Better.
1-B-1-f 298 11/13/08	3x4 sill plate.	1.75" 1.9"	Monotonic 0.75"/min.	12.1 % to 13.0 %, Borate. DF-Standard & Better.
1-B-2-f 299 11/13/08	3x4 sill plate.	1.75" 1.8"	Monotonic 0.75"/min.	12.5 %, Boarate. DF-Standard & Better.
2-B-1-f 304 11/14/08	3x4 sill plate.	1.75" 2.0"	Cyclic (0.2 Hz) SEAOC-Modified.	10.6 % to 12.3 %, Borate. DF-Standard & Better.
2-B-2-f 305 11/14/08	3x4 sill plate.	1.75" 1.7"	Cyclic (0.2 Hz) SEAOC-Modified.	10.7 % to 11.8 %, Borate. DF-Standard & Better.
1-D-1-nf 300 11/13/08	3x4 sill plate.	1.75" 1.9"	Monotonic 0.75"/min.	10.1 % to 12.4 %, Borate. DF-Standard & Better.
1-D-2-nf 301 11/13/08	3x4 sill plate.	1.75" 1.8"	Monotonic 0.75"/min.	11.2 %, Boarate. DF-Standard & Better.
2-D-1-nf 306 11/14/08	3x4 sill plate.	1.75" 1.8"	Cyclic (0.2 Hz) SEAOC-Modified.	10.9 % to 11.1 %, Borate. DF-Standard & Better.
2-D-2-nf 307 11/14/08	3x4 sill plate.	1.75" 1.9"	Cyclic (0.2 Hz) SEAOC-Modified.	9.0 % to 9.1 %, Borate. DF-Standard & Better.
4-A-1-f 310 11/14/08	2x6 sill plate.	2.75" 2.6"	Monotonic 0.75"/min.	14.0 % to 17.4 %, Borate. DF-#2.
4-A-2-f 311 11/14/08	2x6 sill plate.	2.75" 2.7"	Monotonic 0.75"/min.	17.6 % to 18.2 %, Borate. DF-#1 or Better.
4-C-1-f 314 11/19/08	2x6 sill plate.	2.75" 2.6"	Cyclic (0.2 Hz) SEAOC-Modified.	14.0 % to 17.4 %, Borate. DF-#2.
4-C-2-f 315 11/19/08	2x6 sill plate.	2.75" 2.4"	Cyclic (0.2 Hz) SEAOC-Modified.	17 %, Borate. DF-#1 or Better.
4-B-1-f 312 11/14/08	3x6 sill plate.	2.75" 2.7"	Monotonic 0.75"/min.	14 %, Borate. DF-#1 or Better.
4-B-2-f 313 11/14/08	3x6 sill plate.	2.75" 2.9"	Monotonic 0.75"/min.	9.2 %, ACQ . DF, Grade N/A.
4-D-1-f 316 11/19/08	3x6 sill plate.	2.75" 2.6"	Cyclic (0.2 Hz) SEAOC-Modified.	10.1 %, Borate. DF-#1 or Better.
4-D-2-f 317 11/19/08	3x6 sill plate.	2.75" 2.7"	Cyclic (0.2 Hz) SEAOC-Modified.	11.6 %, ACQ . DF, Grade N/A.

Table 1 – Summary of Primary Tests

Test ID's Test Date	Plate Test. UON 5/8" A/B (0.559") 3"x3"x0.229" washer	Edge Distance: Nominal, Actual.	Loading Protocol	Moisture Content, PT Lumber species & grade.
Spare 1-f 308 11/14/08	3x4 sill plate. Loose nut O/S hole = 0.75" f	1.75" 1.9"	Cyclic (0.2 Hz) SEAOC-Modified.	10.1 % to 11.8 %, Borate. DF-Standard & Better.
Spare 2-f 309 11/14/08	3x4 sill plate.	1.75" 1.7"	Cyclic (0.2 Hz). SPD (D control). Input error.	8.8 %, Borate. DF-Standard & Better.
Spare SPD-1-f 302 11/13/08	(N) 2x4 sill plate. Used same anchor tested in 2-C-2-nf.	1.75" 1.9" Same as 2-C-2-nf	Cyclic (0.2 Hz) SPD (D control).	9.5 %, Boarate. DF-Standard & Better.
Spare SPD-2-f 303 11/13/08	2x4 sill plate.	1.75" 1.8"	Cyclic (0.2 Hz) SPD (D control).	8.8 %, Borate. DF-Standard & Better.

Table 2 – Summary of “Bonus” Tests.

Test Set-Up and Instrumentation:

The primary tests were conducted at the Tyrell Gilb Research Laboratory in Stockton, California. The laboratory is owned and operated by the Simpson Strong-Tie Company (SSTC) who generously agreed to donate materiel and testing services to this project. The majority of the testing occurred between November 12-14, 2008 (four tests were completed on November 19, 2008). **Image 2** is an annotated image showing the typical set-up for the single-anchor tests.

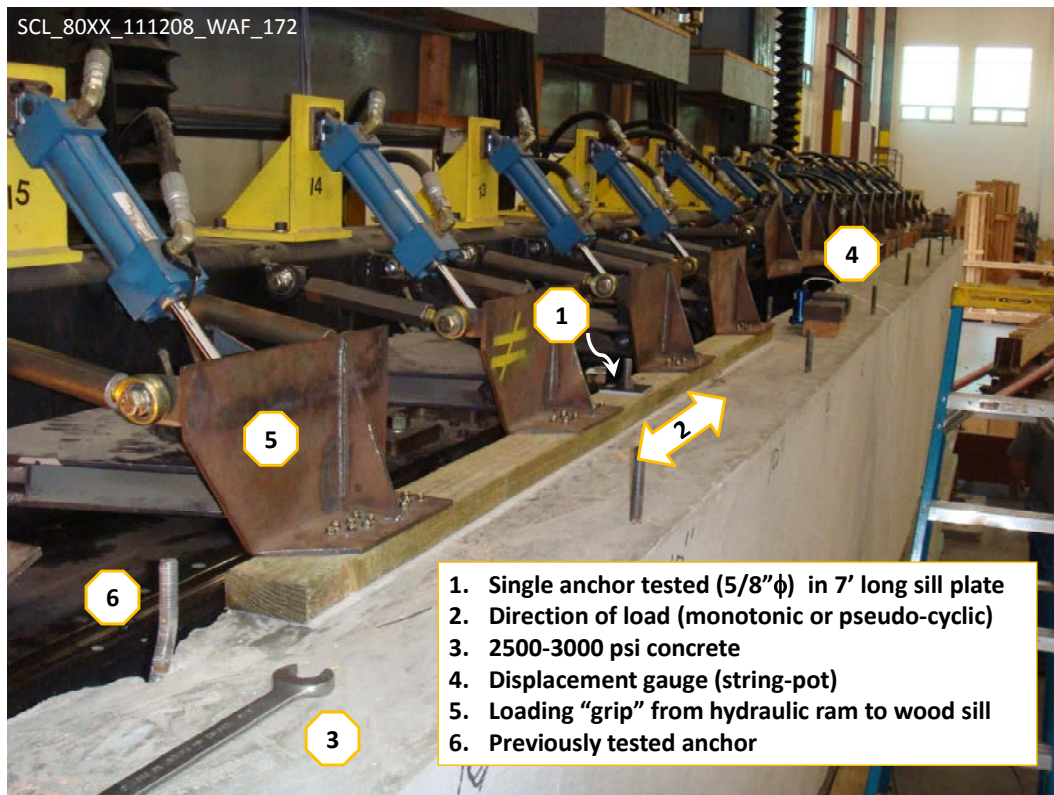


Image 2 – Typical set-up for anchor tests at Tyrell Gilb Research Laboratory in Stockton, CA.

Load rate and frequency - Monotonic tests were run as displacement controlled at a rate of 0.75"/minute. Cyclic tests were run as displacement controlled at a frequency of 0.2Hz (1 cycle every 5 seconds).

Load transfer – each anchor bolt was tested in a 7' long sill plate. Four loading “grips” transferred the shearing force from the loading beam to the sill plate (see **Image 2**). The “grips” were attached to the sill plate with 1-1/2" long SDS Series wood screws. A total of 64 screws were used to transfer the applied load into each test specimen. No vertical load was introduced.

Displacement – Displacement was measured horizontally at two locations; (1) at the loading ram and (2) at the sill plate adjacent to the anchor bolt.

Data acquisition - all loads and displacements were digitally collected by the data acquisition system. For the monotonic tests, data was collected at a rate of 32 readings per second. For the cyclic tests, data was collected 8 times per second.

Documentation – specimen details were documented before and after each test. Video was collected during each test from two camera angles: (1) from the side to observe the face of the concrete and any spalls that formed, and (2) from above to observe the sill plate and anchor bolt behavior.

Pachometer survey – to confirm the clear cover at each anchor location, a *Profometer* re-bar locator (pachometer), manufactured by Proceq Instruments was used.

Impact-echo survey – during each test, impact-echo testing was used to “sound” for internal flaws. From earlier experiments, it was determined that concrete delaminations can form some time before anything is visually observable from the exterior face of the concrete.

Test Plan Development:

Research, test data and/or analysis considered in development of the test plan included the following:

Prior testing of wood bolted to concrete (**Reference 1**) using 1/2" and 3/4" diameter anchor bolts at approximately 8" from edges of the concrete specimen exhibited yielding of the bolt as predicted by the NDS yield limit equations. Observations from the monotonic tests included yielding of the bolt “at the surface of the concrete” followed by rotation of the bolt such that the washer below the nut was pressed into the wood sill plate. Concrete spalling was observed in the vicinity of the bolt. The reported testing; however, did not evaluate a wood sill plate bolted to concrete using minimum edge distances or under cyclic loading.

Comparative data on the capacities of 1/2" and 3/4" diameter bolted connections (wood-to-wood) using various loading protocols (pseudo-cyclic, monotonic, and sequential phased displacement) (**Reference 3**) indicated fastener fatigue was not a likely failure mode and that ultimate strength was not significantly influenced by test method used.

Preliminary experiments conducted during Summer 2008 and performed at Scientific Construction Laboratories (SCL), Inc. provided basic information on connection behavior and test set-up. Specimen configuration (e.g. single 5/8' diameter anchors with 1-3/4" edge distance) was identical to that specified for the Stockton tests. Observations from the preliminary tests indicated the following:

- NDS Yield Mode III_s and IV were the governing yield mode for wood sill plate anchors loaded parallel to the concrete edge for 2x and 3x nominal thickness wood members, respectively.
- Concrete side break-out occurs but usually at relatively large loads and displacements when compared to calculated design capacities.
- Initial nut tightness has an effect on connection performance and that significant friction develops between the concrete and the wood.
- Early stages of concrete side break-out are not visually detectable during the test.

From the preliminary study, friction between the wood sill plate and concrete was considered to be significant. The amount of shear resisted by friction was not known and the amount of friction present in the test may not be present in real applications. These were among the considerations for recommended use of the isolation membrane. The membrane tests (e.g. “nf” in Table 1) were recommended to simulate pure shear to minimize the effect of friction on connection behavior, and conservatively force the majority of the load into the anchor bolt.

The load protocol adopted for the Stockton tests, called the SEAOC Modified load protocol in **Table 1**, was developed by the SEAOC Seismology Committee and the Light-Frame Construction Subcommittee. Initially the loading was designed to be force-controlled, however hydraulic equipment issues developed during the first cyclic test that necessitated a change to a displacement-controlled load protocol.

Peak loads from monotonic tests were used to establish the reference force term, Q_0 , used to prescribe the load steps in the pseudo-cyclic testing. Monotonic tests were run at a sufficiently slow rate to pick up the internal flaws forming within the concrete by using impact-echo testing.

The Pseudo-cyclic tests were based on the CUREE load protocol (See **Reference 2**) but with cycles added at low load levels. **Table 3** shows the CUREE cyclic protocol load steps (varying between $0.5Q_0$ and $1.0Q_0$). Displacements associated with smaller load steps (e.g. 500 lbf, 1000 lbf, 1500 lbf, 2250 lbf, 3000 lbf and 5000 lbf) were taken from monotonic test data to produce the new protocol called SEAOC Modified in Table 1. **Image 3** shows an example plot of the SEAOC Modified displacement-based loading protocol.

Additional loading inputs described in FEMA-461 (See **Reference 7**) were considered.

Spare tests were included in the test plan to provide some redundancy in case of specimen damage or errors in data acquisition. The 24 primary tests ran relatively issue-free leaving four “bonus” tests. Three were run using Sequential Phase Displacement (SPD) load protocol (see **Table 2**). The fourth “bonus” test was run as a cyclic test using SEAOC Modified load protocol with a loose anchor nut and with an over-sized hole in the wood sill plate.

All tests were conducted in “uncracked” concrete. Further discussion of the thinking and opinions behind this decision can be found in the SEAOC Blue Book article on anchor bolts (available from www.SEAOC.org/bluebook).

Monotonic Tests	2x4 f	2x4 nf	3x4 f	3x4 nf	2x6 f	3x6 f	Comment
Monotonic 1	1-A-1-f (289)	1-C-1-f (291)	1-B-1-f (298)	1-D-1-f (300)	4-A-1-f (310)	4-B-1-f (312)	Tests 289-292 run as force-control (250#/s).
Monotonic 2	1-A-2-f (290)	1-C-2-f (292)	1-B-2-f (299)	1-D-2-f (301)	4-A-2-f (311)	4-B-2-f (313)	All other monotonic run as displacement control
Pseudo-Cyclic 1	2-A-1-f (293)	2-C-1-f (295)	2-B-1-f (304)	2-D-1-f (306)	4-C-1-f (314)	4-D-1-f (316)	Test 293 run as force-control (50#/s).
Pseudo-Cyclic 2	2-A-2-f (294)	2-C-2-f (296)	2-B-2-f (305)	2-D-2-f (307)	4-C-2-f (315)	4-D-2-f (317)	These cyclic tests run as displacement control
Qo =	14000#	8000#	14000#	10000#	14000#	16000#	Qo determined from 2 monotonic tests.
# of cycles at +/-	2x4 f (Inches)	2x4 nf (Inches)	3x4 f (Inches)	3x4 nf (Inches)	2x6 f (Inches)	3x6 f (Inches)	Comment
3	0.001	0.045	0.001	0.028	0.044	0.001	Average Δ at 500 # from 2 monotonic tests
3	0.034	0.068	0.001	0.055	0.069	0.002	Average Δ at 1000 # from 2 monotonic tests
3	0.057	0.078	0.014	0.078	0.086	0.036	Average Δ at 1500 # from 2 monotonic tests
3	0.065	0.093	0.080	0.096	0.107	0.053	Average Δ at 2250 # from 2 monotonic tests
3	0.076	0.108	0.104	0.117	0.131	0.065	Average Δ at 3000 # from 2 monotonic tests
3	---	---	---	---	0.231	0.110	Extra cycles. Average Δ at 5000 # from 2 monotonic tests
5	0.206	0.139	0.311	0.219	0.367	0.326	0.5Qo
5	0.442	0.231	0.793	0.551	0.662	0.806	0.7Qo
1	0.614	0.319	1.016	0.941	0.815	0.913	0.8Qo
2	0.294	0.174	0.518	0.322	0.485	0.571	0.6Qo
1	0.834	0.456	1.139	1.732	0.991	1.184	0.9Qo
2	0.395	0.212	0.724	0.301	0.626	0.756	0.675Qo
1	1.500	0.704	1.368	2.053	1.204	1.437	1Qo
2	0.532	0.251	0.886	0.761	0.739	0.913	0.75Qo

Table 3 – Inputs for displacement-controlled cyclic tests. See Image 3 below for a graphical example plot.

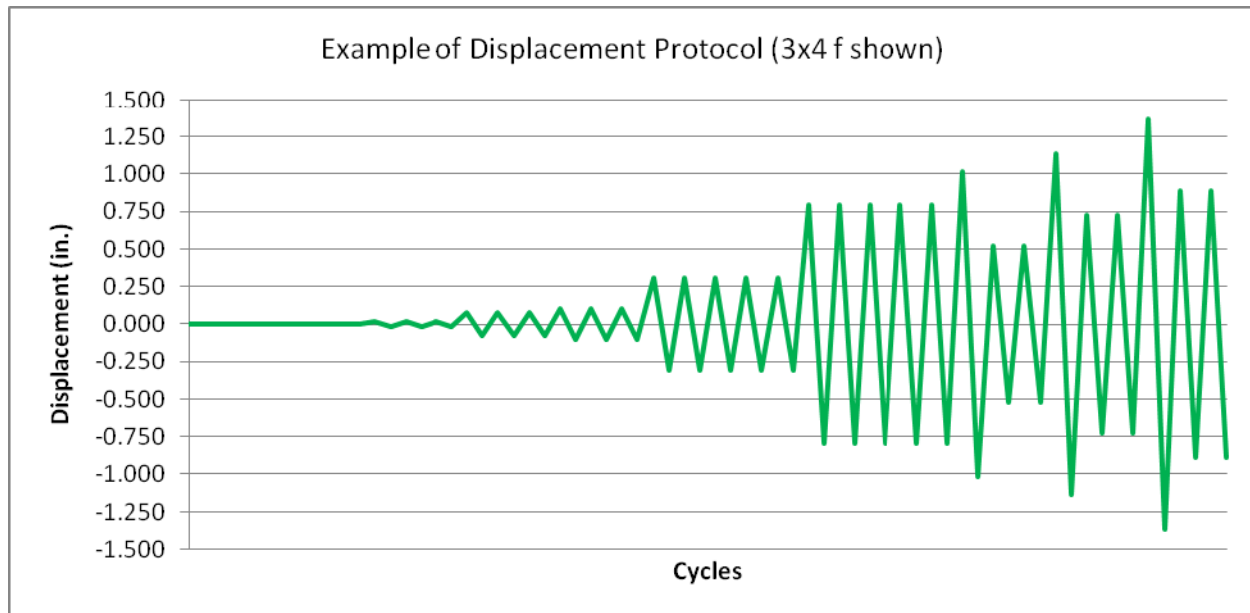


Image 3 – Graphical example plot of inputs for displacement-controlled cyclic tests

Experimental Performance of Test Specimens:

The primary test results have been plotted on **Charts 1-6**. Each chart contains four plots; two identical monotonic tests and two identical cyclic tests. The load and displacement axes are kept constant which results in minor data clipping. This constant scale was deemed better (more useful) for comparison of results than trying to compare plots with varying scales.

In addition to the data plotting, **Appendix Table A** contains detailed observations of each test conducted.

Chart 1 shows Cyclic Tests (293, 294) vs. Monotonic Tests (289, 290). These tests were on 2x4 sill plates with 1-3/4" specified edge distance, loaded parallel to the concrete edge. Friction was allowed to develop below the sill plate.

Chart 2 shows Cyclic Tests (295, 296) vs. Monotonic Tests (291, 292). These tests were on 2x4 sill plates with 1-3/4" specified edge distance, loaded parallel to the concrete edge. An isolation membrane was used between the wood sill plate and concrete to minimize friction.

Chart 3 shows Cyclic Tests (304, 305) vs. Monotonic Tests (298, 299). These tests were on 3x4 sill plates with 1-3/4" specified edge distance, loaded parallel to the concrete edge. Friction was allowed to develop below the sill plate.

Chart 4 shows Cyclic Tests (306, 307) vs. Monotonic Tests (300, 301). These tests were on 3x4 sill plates with 1-3/4" specified edge distance, loaded parallel to the concrete edge. An isolation membrane was used between the wood sill plate and concrete to minimize friction.

Chart 5 shows Cyclic Tests (314, 315) vs. Monotonic Tests (310, 311). These tests were on 2x6 sill plates with 2-3/4" specified edge distance, loaded parallel to the concrete edge. Friction was allowed to develop below the sill plate.

Chart 6 shows Cyclic Tests (316, 317) vs. Monotonic Tests (312, 313). These tests were on 3x6 sill plates with 2-3/4" specified edge distance, loaded parallel to the concrete edge. Friction was allowed to develop below the sill plate.

Image 4 and **Image 5** show examples of pre-test and post-test documentation.

Plots for the "bonus" tests and discussion of observed test behavior for the "bonus" tests is not included at this time.

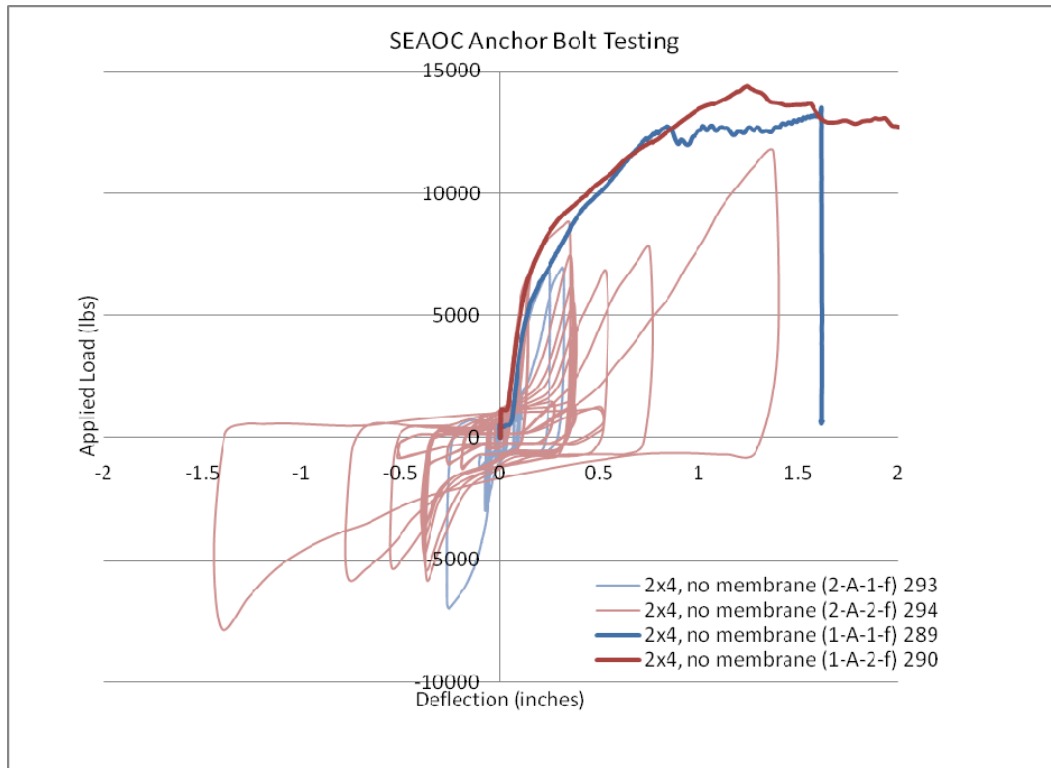


Chart 1 – Cyclic Tests (293, 294) vs Monotonic Tests (289, 290)

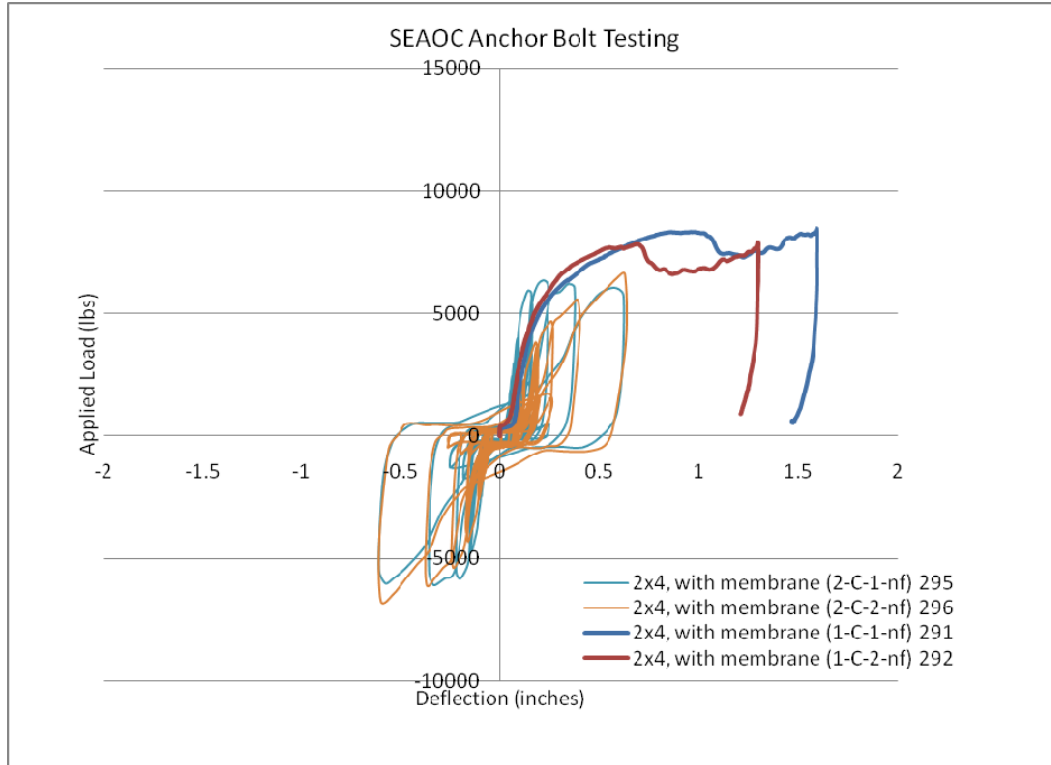


Chart 2 - Cyclic Tests (295, 296) vs Monotonic Tests (291, 292)

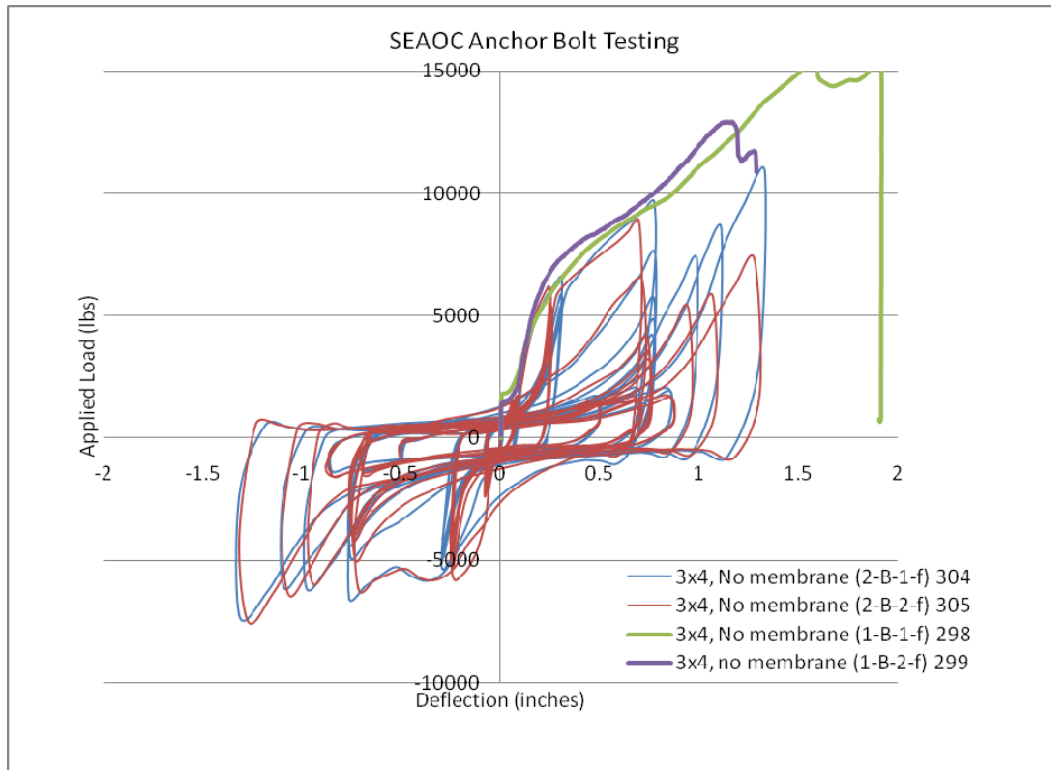


Chart 3 - Cyclic Tests (304, 305) vs Monotonic Tests (298, 299)

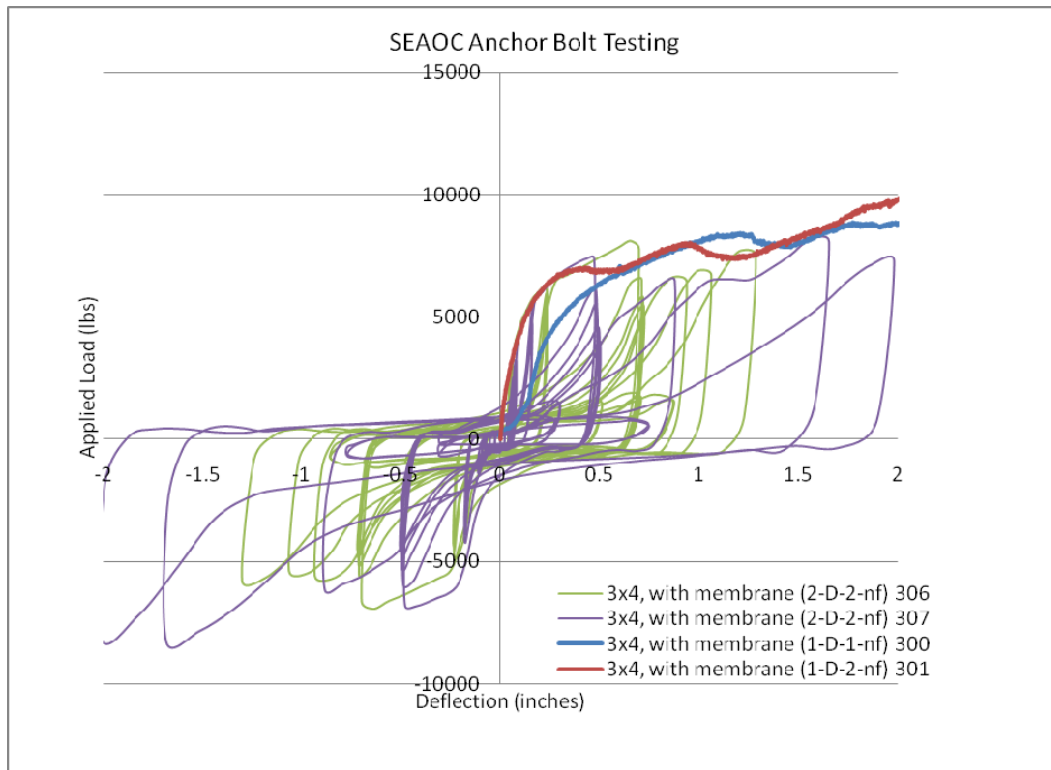


Chart 4 - Cyclic Tests (306, 307) vs Monotonic Tests (300, 301)

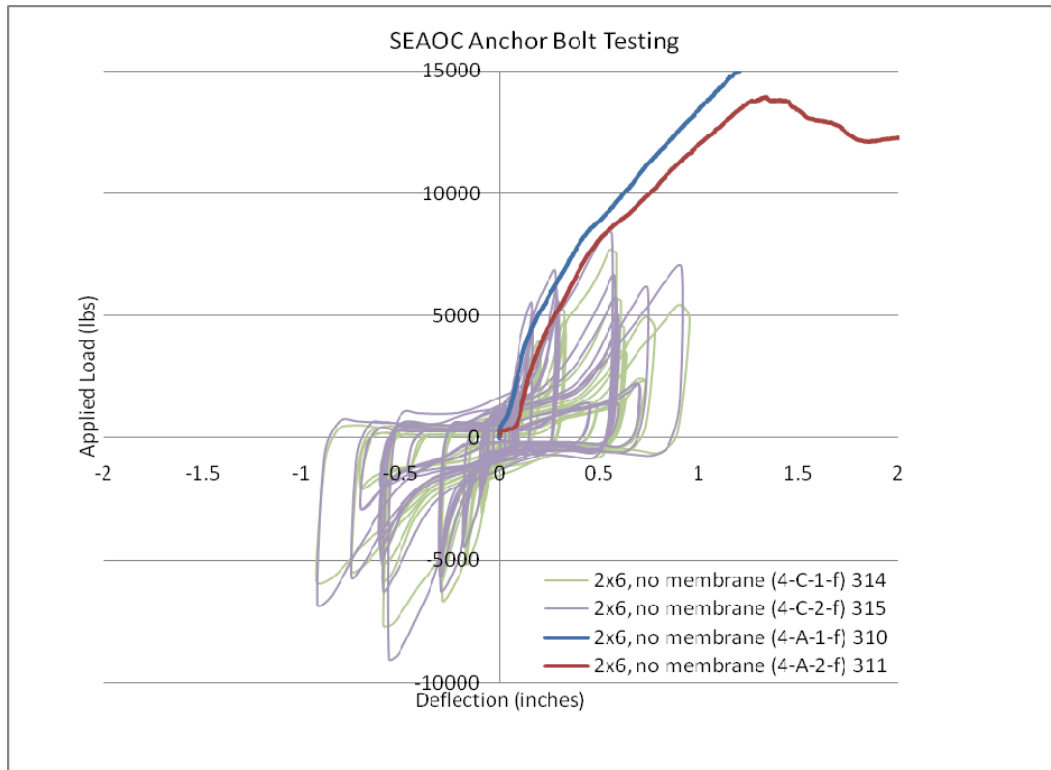


Chart 5 - Cyclic Tests (314, 315) vs Monotonic Tests (310, 311)

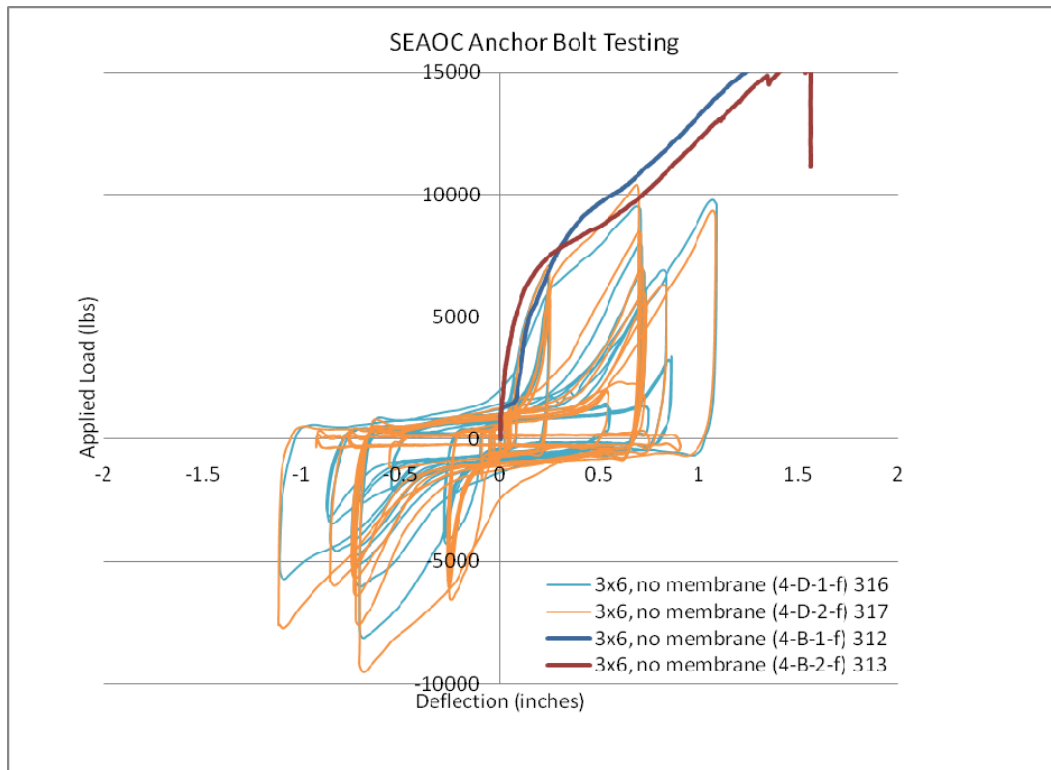
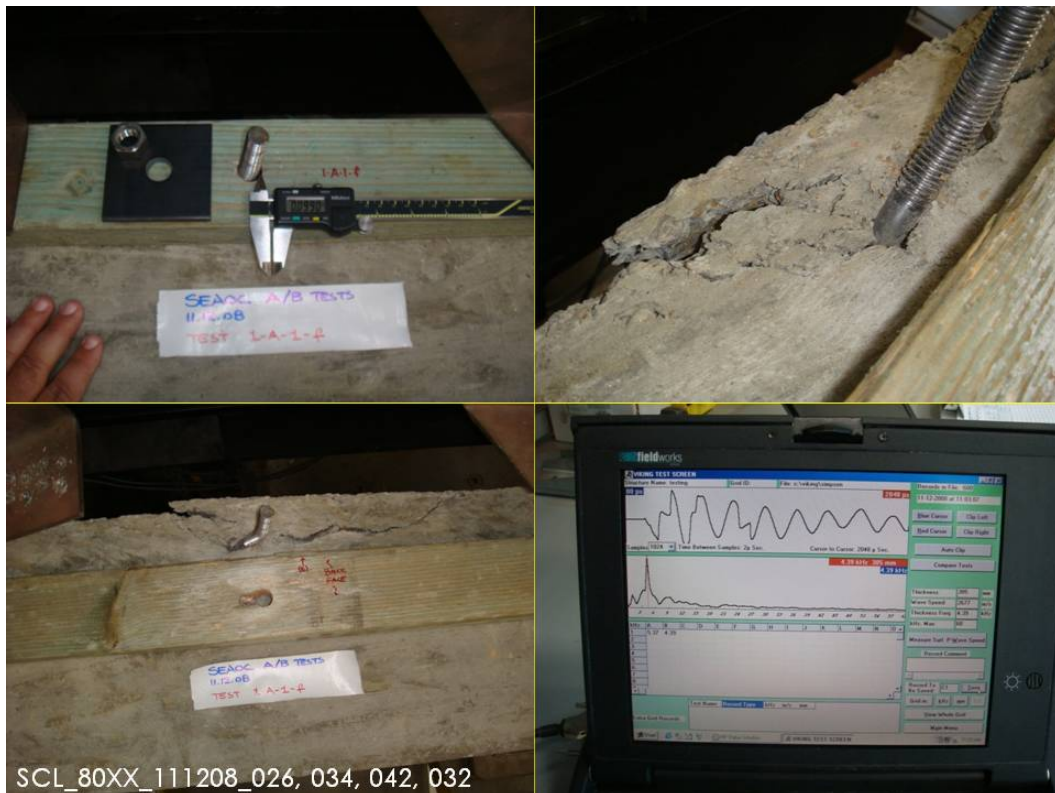


Chart 6 - Cyclic Tests (316, 317) vs Monotonic Tests (312, 313)



Image 4 – Pre-test documentation (typical).



SCL_80XX_111208_026, 034, 042, 032

Image 5 – Post-test documentation (typical). Impact-echo testing shown in lower-right of image.

Experimental Performance of Test Specimens: (continued)

The following plots and images highlight specific observations regarding effect of the membrane on monotonic test results and typical concrete failure for the 1-3/4" edge distance case. See **Appendix Table A** for additional detailed observations of each test conducted.

Effect of friction - Chart 7 shows comparative plots of monotonic tests conducted “with” and “without” the membrane. The membrane created an idealized frictionless plane between the wood sill plate and the surface of the concrete. As shown in Chart 7, the friction effect is negligible at small displacement in the range of the allowable design capacities but has a distinguishable effect at relatively large loads and displacements. The effect of the membrane on load and displacement response in cyclic tests was minor when compared to observations from monotonic tests.

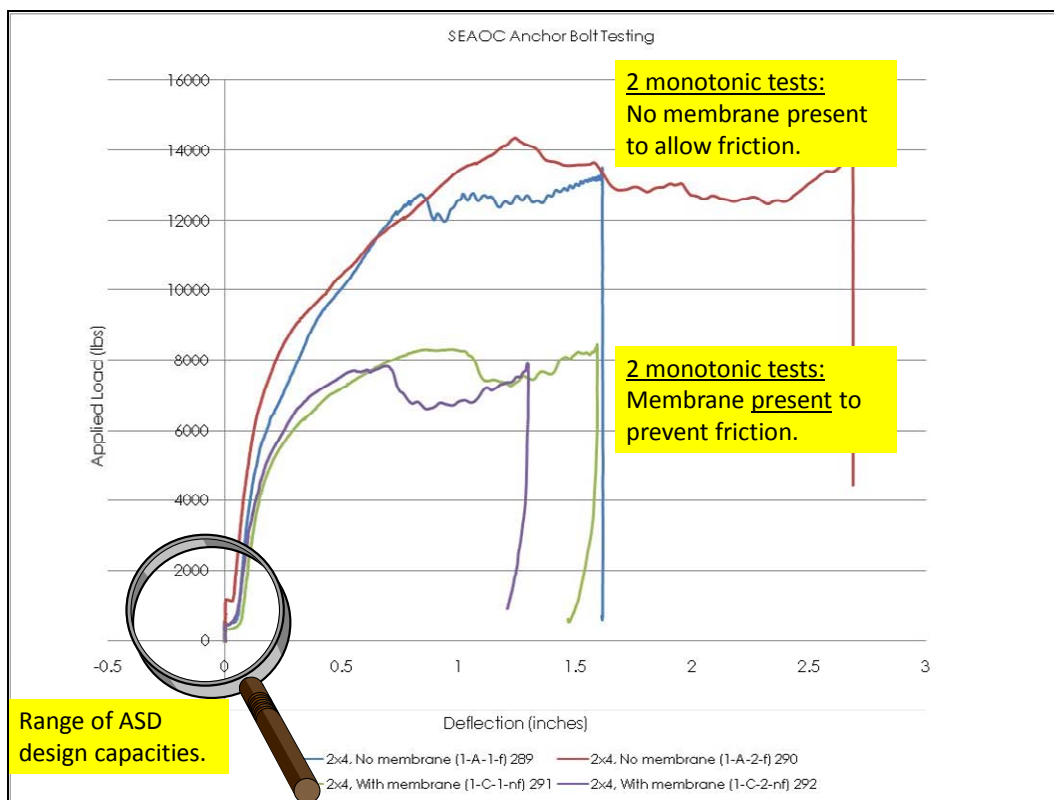


Chart 7 – Comparative plot of monotonic tests with (291, 292) & without (289, 290) membrane.

Concrete side break-out (if it occurred) was detected during the tests using impact-echo method (see **Reference 8**). For each specimen, **Appendix Table A** shows the approximate load and displacement where concrete deterioration was detected. The first stage of deterioration is a series of cracks that form within the concrete propagated from the centre-line of the anchor bolt and angling out towards the outer/free face of the concrete. See **Image 5**. The cracks ultimately reach the outer face and a shallow spall forms. It is important to note that the early stages of concrete deterioration are not always visually apparent. As described in the next section (analytical studies), a strong correlation between the “peak” envelope values with the onset of concrete side break-out was observed.



Image 6 – Composite imagery of “stopped” test (Test 296). See text for description.

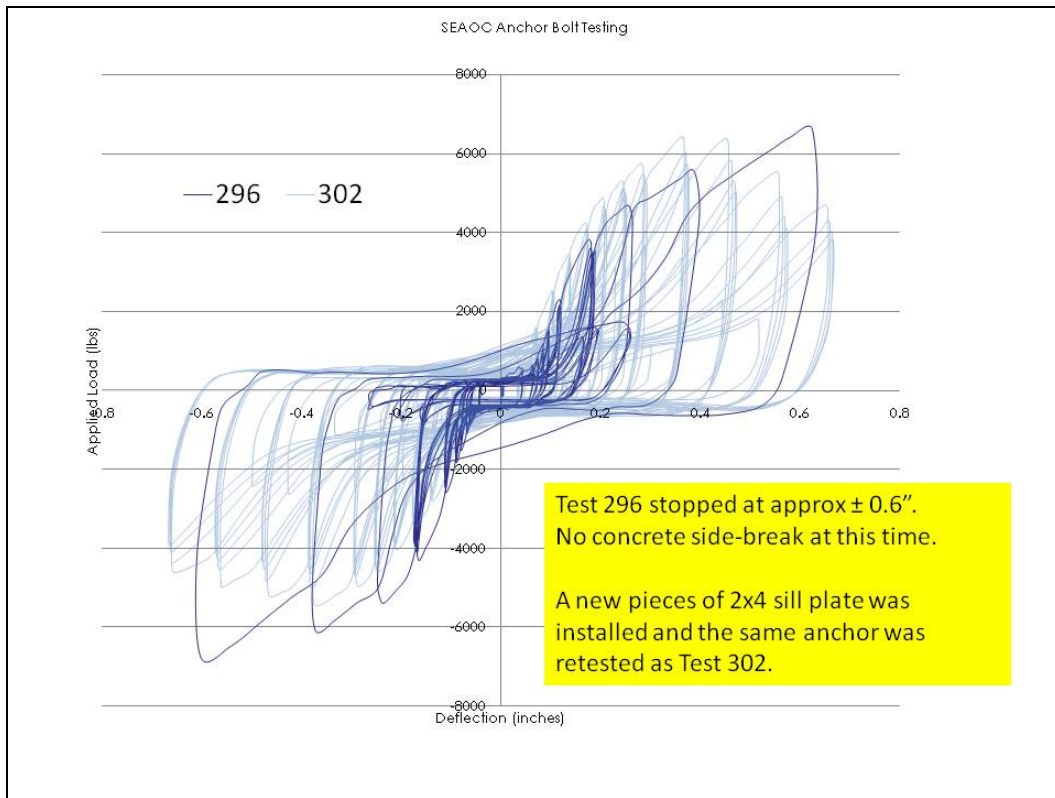


Chart 8 – Composite plot of Tests 296 and 302.

After a number of tests where concrete delaminations were documented at displacements beyond 0.5” it was decided to stop cyclic test 296 at approximately 0.60” displacement. The sill plate was unbolted and documented. The specimen conditions after test 296 are shown in **Image 6**. The concrete remained unflawed as confirmed visually and with impact-echo. The same anchor was then re-tested using one of the spare tests (Test 302). A new piece of sill plate was bolted down and subjected to a SPD protocol. The resulting load-displacement plots for both Tests 296 and 302 are shown on **Chart 8**.

Analytical Studies:

Peak and Ultimate Definitions

All cyclic test data was analyzed in accordance with ASTM E 2126 *Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Walls for Buildings (Reference 6)*. The positive and negative envelop curves for each specimens were combined to produce an average envelope curve used to establish peak load, displacement at peak load, ultimate load, and displacement at ultimate load as summarized in **Table 4**. Graphs of data are provided in **Appendix B**. Example load displacement hysteresis curves are shown in **Charts 1 through 6**. An example average envelope curve is shown in **Figure 2**.

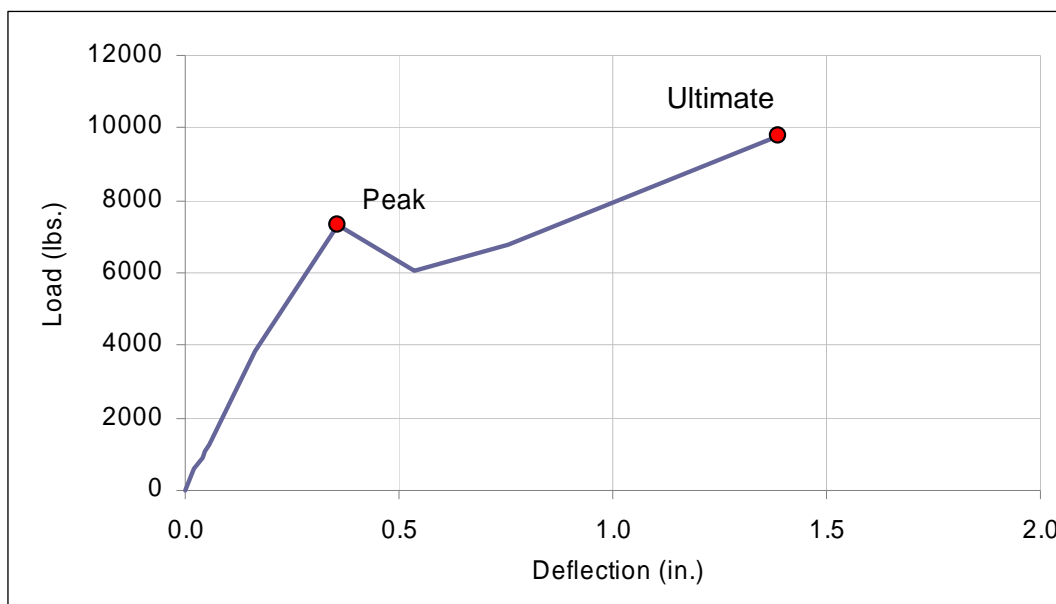


Figure 2 - Average envelope curve from cyclic tests (Specimen 294 shown).

The “saw tooth” pattern shown in **Figure 2** was observed for tests using the SEAOc Modified protocol. For purposes of this study, “Peak” load was assigned to the highest load reached prior to a drop in load level of at least 5% which is a departure from ASTM E2126 where peak is defined as the maximum load. The revised definition of “Peak” used in this report intends to address first signs of noticeable strength loss and also corresponds to first signs of onset of concrete side break-out. Ultimate load, as used in this study, is the last data point with value greater than 0.8 Peak. The “Ultimate” load, as presented in this report, is the load at maximum displacement prior to stopping the test. However, if the load at maximum displacement is smaller than 0.8 Peak load then 0.8 Peak load and the corresponding displacement is reported as “Ultimate”.

Average envelope curves for 1-3/4" edge distance tests are depicted in **Chart 9**.

Average envelope curves for 2-3/4" edge distance tests are depicted in **Chart 10**.

Table 4 (below)– Test Values

Test ID	Sill pate, Load	C _{a1} In.	Peak		Ultimate	
			Load, lbf	Displacement, in.	Load, lbf	Displacement, in.
1-A-1-f 289	2x4, Mono.	1.9	12755	0.84	13519	1.62
1-A-2-f 290	2x4, Mono.	1.8	14367	1.24	14373	2.69
2-A-1-f 293	2x4, Cyclic	1.9	-	-	-	-
2-A-2-f 294	2x4, Cyclic	1.7	7331	0.36	9751	1.39
1-C-1-nf291	2x4, Mono.	1.9	8328	0.96	8465	1.59
1-C-2-nf 292	2x4, Mono.	1.9	7841	0.69	7909	1.30
2-C-1-nf 295	2x4, Cyclic	1.8	6126	0.34	6022	0.58
2-C-2-nf 296	2x4, Cyclic	1.9	6672	0.61	6672	0.61
1-B-1-f 298	3x4, Mono.	1.9	15278	1.59	15380	1.92
1-B-2-f 299	3x4, Mono.	1.8	12950	1.14	11751	1.28
2-B-2-f 304	3x4, Cyclic	2.0	8083	0.71	9064	1.26
2-B-2-f 305	3x4, Cyclic	1.7	7556	0.71	7418	1.28
1-D-1-nf 300	3x4, Mono.	1.9	8416	1.20	9666	2.95
1-D-2-nf 301	3x4, Mono.	1.8	8008	0.94	12468	2.88
2-D-1-nf 306	3x4, Cyclic	1.8	7518	0.65	6729	1.25
2-D-2-nf 307	3x4, Cyclic	1.9	7128	0.45	7693	2.00
4-A-1-f 310	2x6, Mono.	2.6	16342	1.55	13073	2.53
4-A-2-f 311	2x6, Mono.	2.7	13967	1.34	11173	2.23
4-C-1-f 314	2x6, Cyclic	2.6	7657	0.57	6126	0.68
4-C-1-f 315	2x6, Cyclic	2.4	8696	0.56	6957	0.68
4-B-1-f 312	3x6, Mono.	2.7	18791	2.36	18708	2.86
4-B-2-f 313	3x6, Mono.	2.9	15746	1.53	15746	1.53
4-D-1-f 316	3x6, Cyclic	2.6	8835	0.69	7764	1.07
4-D-2-f 317	3x6, Cyclic	2.7	9926	0.69	8529	1.08
Average 2x4 and 3x4, Cyclic, n=7:		1.8	7202	0.5	7621	1.2
Average 2x6 and 3x6, Cyclic, n=4:		2.6	8779	0.6	7344	0.9
Average 2x4 and 3x4, Mono, n= 8:		1.9	10993	1.1	11691	2.0
Average 2x6 and 3x6, Mono, n=4:		2.7	16211	1.7	14675	2.3

In **Figure 2**, the drop in strength and stiffness immediately following peak load coincides with initial detection of concrete damage. This damage is believed to reduce concrete bearing support for the anchor bolt leading to both increased anchor bending stresses and loss of stiffness. As displacement increases, the anchor becomes increasingly stressed in tension in addition to bending and shear. Tension forces are resisted by anchor embedment in concrete and wood bearing under the plate washer to produce a “clamping” of the wood member to the concrete foundation. Increased load resistance, beyond that associated with “Peak” load, was commonly observed at increasing displacement and is attributed to the tensile resistance provided by the anchor after fastener yielding. These increased loads and deflections are associated with “Ultimate” load data in **Table 4** (see **Appendix B**).

From **Table 4**, the following observations can be made:

- Peak loads and displacements from cyclic tests were less than those from monotonic tests
- Peak loads from cyclic tests were not substantially affected by the presence of the membrane

NDS allowable design value

The NDS allowable design value, Z' , for the connections tested are included in **Table 2** and based on the following inputs to the NDS yield limit equations (also referred to as the EYM equations - see **Reference 4**): $D = 0.559$; $F_{yb} = 45000$ psi; $F_{es} = 5600$ psi for $G=0.5$ Douglas Fir; and $F_{em} = 7890$ psi (taken as $3x$ average $f'_c = 2630$ psi). Yield Mode III_s (see **Image 6**) was found to be the controlling yield mode for $2x$ nominal wood sill plates and anchor embedment in concrete of at least 8 diameters in accordance with the following:

$$Z = \frac{k_3 D \ell_s F_{em}}{\left(2 + \frac{F_{em}}{F_{es}}\right) R_d} \quad \text{Eq. 1}$$

Yield mode IV (see **Image 6**) was found to be the controlling yield mode for $3x$ nominal wood sill plates and anchor embedment in concrete of at least 8 diameters in accordance with the following:

$$Z = \frac{D^2}{R_d} \sqrt{\frac{2F_{em}F_{yb}}{3\left(1 + \frac{F_{em}}{F_{es}}\right)}} \quad \text{Eq. 2}$$

where,

$$k_3 = -1 + \sqrt{\frac{2\left(1 + \frac{F_{em}}{F_{es}}\right)}{\frac{F_{em}}{F_{es}}} + \frac{2F_{yb}\left(2 + \frac{F_{em}}{F_{es}}\right)D^2}{3F_{em}\ell_s^2}}$$

$R_d = 3.2$ (3.2 is the reduction term for Yield Mode III_s and IV)

$\ell_s = 1.5$ inch for $2x$ nominal and 2.5 inch for $3x$ nominal (side member dowel bearing length, inches)

Wood to concrete anchor bolt design values in **Table 5** have been adjusted for short term seismic loading by multiplying by the 1.6 load duration factor. The ratio of average Peak cyclic strengths to NDS allowable design values for target 1-3/4" edge distance ranges from 4.6 to 5.9.

The NDS yield value (5% offset yield) is shown in **Table 5** and is calculated from Eq. 1 as previously shown except setting the value of $R_d = 1.0$. The NDS yield value is included in **Table 5** to enable

comparison of the yield limit state basis of the NDS calculation provisions for anchors to test results. The ratio of average Peak cyclic strengths to NDS yield values for target 1-3/4" edge distance ranges from 2.3 to 2.9.

It should be noted that the NDS yield limit equations do not describe ultimate connection failure. Rather, they estimate the load at which inelastic connection behavior begins to occur (i.e.- the "yield point" due to plastic hinge formation in the fastener or deformation of wood fibers in bearing against the fastener or combination thereof). Published connection capacities (Z) are determined by calculating a connection yield point, then applying safety adjustment factors. The theoretical yield point for the anchor connection between the wood sill and the concrete foundation is determined by setting $R_d = 1.0$ as described previously. Connections exhibiting fastener yielding modes often have significantly greater ultimate strength than estimated by the yield point established by the yield limit equations.

Table 5 (below). Anchor bolt values, NDS

Test ID	Sill pate, Load	C_{a1}	NDS		Peak ^a / NDS Allowable	Peak ^a / NDS Yield	Max ^b / NDS Yield
		In.	Allowable, lbf	Yield, lbf			
1-A-1-f 289	2x4, Mono.	1.9	1247	2493	10.8	5.4	5.4
1-A-2-f 290	2x4, Mono.	1.8	1247	2493	11.5	5.8	5.8
2-A-1-f 293	2x4, Cyclic	1.9	1247	2493	-	-	0.0
2-A-2-f 294	2x4, Cyclic	1.7	1247	2493	5.9	2.9	3.9
1-C-1-nf291	2x4, Mono.	1.9	1247	2493	6.7	3.3	3.4
1-C-2-nf 292	2x4, Mono.	1.9	1247	2493	6.3	3.1	3.2
2-C-1-nf 295	2x4, Cyclic	1.8	1247	2493	4.9	2.5	2.5
2-C-2-nf 296	2x4, Cyclic	1.9	1247	2493	5.4	2.7	2.7
1-B-1-f 298	3x4, Mono.	1.9	1549	3097	9.9	4.9	5.0
1-B-2-f 299	3x4, Mono.	1.8	1549	3097	8.4	4.2	4.2
2-B-2-f 304	3x4, Cyclic	2.0	1549	3097	5.2	2.6	2.9
2-B-2-f 305	3x4, Cyclic	1.7	1549	3097	4.9	2.4	2.4
1-D-1-nf 300	3x4, Mono.	1.9	1549	3097	5.4	2.7	3.1
1-D-2-nf 301	3x4, Mono.	1.8	1549	3097	5.2	2.6	4.0
2-D-1-nf 306	3x4, Cyclic	1.8	1549	3097	4.9	2.4	2.4
2-D-2-nf 307	3x4, Cyclic	1.9	1549	3097	4.6	2.3	2.5
4-A-1-f 310	2x6, Mono.	2.6	1247	2493	13.1	6.6	6.6
4-A-2-f 311	2x6, Mono.	2.7	1247	2493	11.2	5.6	5.6
4-C-1-f 314	2x6, Cyclic	2.6	1247	2493	6.1	3.1	3.1
4-C-1-f 315	2x6, Cyclic	2.4	1247	2493	7.0	3.5	3.5
4-B-1-f 312	3x6, Mono.	2.7	1549	3097	12.1	6.1	6.1
4-B-2-f 313	3x6, Mono.	2.9	1549	3097	10.2	5.1	5.1
4-D-1-f 316	3x6, Cyclic	2.6	1549	3097	5.7	2.9	2.9
4-D-2-f 317	3x6, Cyclic	2.7	1549	3097	6.4	3.2	3.2
Average 2x4 and 3x4, Cyclic, n=7:					5.1	2.6	2.8
Average 2x6 and 3x6, Cyclic, n=4:					6.3	3.2	3.2

^a Peak is the peak load from tests, See Table 4.

^b Max represents the maximum of the tested Peak load and tested Ultimate load.

ACI 318-08 nominal concrete breakout strength, $V_{cb||}$

The nominal concrete breakout strength in shear of a single anchor with shear force acting parallel to the edge, denoted as $V_{cb||}$ herein, is of interest. $V_{cb||}$ is taken to be twice that of V_{cb} for shear force acting perpendicular to the edge with $\psi_{ed,v} = 1.0$:

$$V_{cb||} = 2(A_{vc}/A_{vc0}) \psi_{ed,v} \psi_{c,v} \psi_{h,v} V_b \quad \text{Eq. 3}$$

$$V_b = 7 (\ell_e/d_a)^{0.2} (d_a)^{0.5} \lambda (f'_c)^{0.5} (c_{a1})^{1.5} \quad \text{Eq. 4}$$

Where,

$(A_{vc}/A_{vc0}) = 1.0$ (projected concrete failure area not influenced by proximity to corner, fastener spacing, or member thickness)

$\psi_{ed,v} = 1.0$ (value set equal to 1.0 per ACI 318 Appendix D for shear parallel to edge)

$\psi_{c,v} = 1.4$ (uncracked condition assumed),

$\psi_{h,v} = 1.0$ (thickness of member in tests greater than $1.5 c_{a1}$)

V_b = basic concrete breakout strength in shear of a single anchor in cracked concrete, lbf

ℓ_e = 4.5 in. (load bearing length of anchor for shear not to exceed $8d_a$, in.)

d_a = 0.559 in. (outside diameter of anchor, in.)

λ = 1.0 (standard weight concrete)

f'_c = 2630 psi (actual compressive strength of concrete)

c_{a1} = see **Table 6** for actual distance from the center of an anchor to the edge of concrete, in.

Values of $V_{cb||}$ are summarized in **Table 6**. Design strengths are provided for the case where the anchor is ductile and not ductile as follows:

$$\text{Non ductile: } 0.75 (\phi) V_{cb||} \times 0.5 \times 0.7 \quad \text{EQ. 5}$$

$$\text{Ductile: } 0.75 (\phi) V_{cb||} \times 0.7 \quad \text{EQ. 6}$$

Where,

0.75 = factor to account for seismic loading effects on strength

$\phi = 0.7$ (strength reduction factor for shear loads governed by concrete break-out, condition B)

0.5 = factor to account for non-ductile failure per ACI 318-08 Section D.3.3.6. This factor is 0.4 in ACI 318-05.

0.7 = factor to adjust from LRFD to ASD basis.

Nominal break-out capacities $V_{cb||}$ in accordance with ACI 318-08 Appendix D intends to approximate the 5% fractile of concrete break-out strength. To facilitate comparison with mean test values from this study, values of $V_{cb||}$ are adjusted to a mean basis using a nominal to mean ratio of 0.75 (see **Reference 9**). This mean break-out strength, associated with $V_{cb||}$, is denoted as $V_{cb||Avg}$ in **Table 6**. The ratio of the peak cyclic strength to $V_{cb||Avg}$ ranges from 1.7 to 2.2 for 1-3/4" edge distance indicating conservatism in the ACI 318 break-out strength predictions. Values of $V_{cb||}$ and $V_{cb||Avg}$ for 1-3/4" edge distance tests are shown in **Table 6** and depicted in **Chart 9**. Values of $V_{cb||}$ and $V_{cb||Avg}$ for 2-3/4" edge distance tests are shown in **Table 6** and depicted in **Chart 10**.

It should be noted that an assumed nominal to mean ratio equal to 0.75 is associated with a COV=0.15. Assumption of concrete break-out strength COV, other than COV=0.15, will produce different estimates of the mean break-out strength associated with $V_{cb//}$. For example, COV=0.30 results in a nominal to mean ratio of 0.5 and corresponding reductions in the level of conservatism of the ACI 318 break-out strength predictions relative to tested peak strengths.

As previously described, the ultimate load can be in excess of the peak load as tension forces develop in the anchor. The term “Max” in **Table 6** represents the maximum of the tested Peak load and tested Ultimate load to account for cases where increased strength was observed beyond first concrete damage. The ratio of the cyclic maximum values to $V_{cb//Avg}$ ranges from 1.7 to 2.9 for 1-3/4” edge distance.

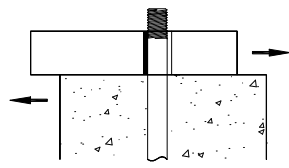
Table 6 (below). Concrete break-out values, ACI 318-08

Test ID	Sill pate, Load	c_{a1} In.	ACI 318 Allowable		ACI 318 Break-out ^c		Peak ^a / $V_{cb//}$	Peak ^a / $V_{cb//Avg}$	Max ^b / $V_{cb//Avg}$
			Non ductile	Ductile	$V_{cb//}$	$V_{cb//Avg}$	$V_{cb//}$	$V_{cb//Avg}$	$V_{cb//Avg}$
1-A-1-f 289	2x4, Mono.	1.9	548	1096	2983	3978	4.5	3.4	3.4
1-A-2-f 290	2x4, Mono.	1.8	505	1011	2751	3668	5.2	3.9	3.9
2-A-1-f 293	2x4, Cyclic	1.9	548	1096	2983	3978	-	-	-
2-A-2-f 294	2x4, Cyclic	1.7	464	928	2525	3367	2.9	2.2	2.9
1-C-1-nf291	2x4, Mono.	1.9	548	1096	2983	3978	2.8	2.1	2.1
1-C-2-nf 292	2x4, Mono.	1.9	505	1011	2751	3668	2.9	2.1	2.2
2-C-1-nf 295	2x4, Cyclic	1.8	505	1011	2751	3668	2.2	1.7	1.7
2-C-2-nf 296	2x4, Cyclic	1.9	548	1096	2983	3978	2.2	1.7	1.7
1-B-1-f 298	3x4, Mono.	1.9	548	1096	2983	3978	5.1	3.8	3.9
1-B-2-f 299	3x4, Mono.	1.8	505	1011	2751	3668	4.7	3.5	3.5
2-B-2-f 304	3x4, Cyclic	2.0	592	1184	3222	4296	2.5	1.9	2.1
2-B-2-f 305	3x4, Cyclic	1.7	464	928	2525	3367	3.0	2.2	2.2
1-D-1-nf 300	3x4, Mono.	1.9	548	1096	2983	3978	2.8	2.1	2.4
1-D-2-nf 301	3x4, Mono.	1.8	464	928	2525	3367	3.2	2.4	3.7
2-D-1-nf 306	3x4, Cyclic	1.8	505	1011	2751	3668	2.7	2.0	2.0
2-D-2-nf 307	3x4, Cyclic	1.9	548	1096	2983	3978	2.4	1.8	1.9
4-A-1-f 310	2x6, Mono.	2.6	877	1755	4775	6368	3.4	2.6	2.6
4-A-2-f 311	2x6, Mono.	2.7	929	1857	5054	6739	2.8	2.1	2.1
4-C-1-f 314	2x6, Cyclic	2.6	877	1755	4775	6368	1.6	1.2	1.2
4-C-1-f 315	2x6, Cyclic	2.4	778	1556	4235	5647	2.1	1.5	1.5
4-B-1-f 312	3x6, Mono.	2.7	929	1857	5054	6739	3.7	2.8	2.8
4-B-2-f 313	3x6, Mono.	2.9	1034	2067	5625	7501	2.8	2.1	2.1
4-D-1-f 316	3x6, Cyclic	2.6	877	1755	4775	6368	1.9	1.4	1.4
4-D-2-f 317	3x6, Cyclic	2.7	929	1857	5054	6739	2.0	1.5	1.5
Average 2x4 and 3x4, Cyclic, n=7:							2.6	1.9	2.1
Average 2x6 and 3x6, Cyclic, n=4:							1.9	1.4	1.4

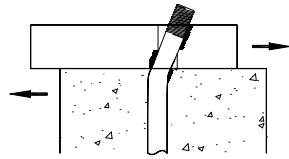
^a Peak is the peak load from tests, See **Table 4**.

^b Max represents the maximum of the tested Peak load and tested Ultimate load.

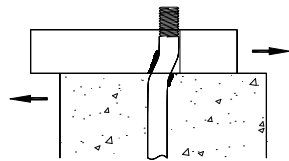
^c $V_{cb//}$ and $V_{cb//Avg}$ are the concrete break-out strength for shear parallel to the edge corresponding to the 5% fractile estimate and mean value estimate.



Mode I (wood bearing deformation in side member)



Mode III_s (wood bearing deformation in side member and dowel bending in concrete)



Mode IV (wood bearing deformation in side member and dowel bending in wood member and concrete)

Image 7 – NDS Yield Modes I, III_s, and IV for an anchor bolt.

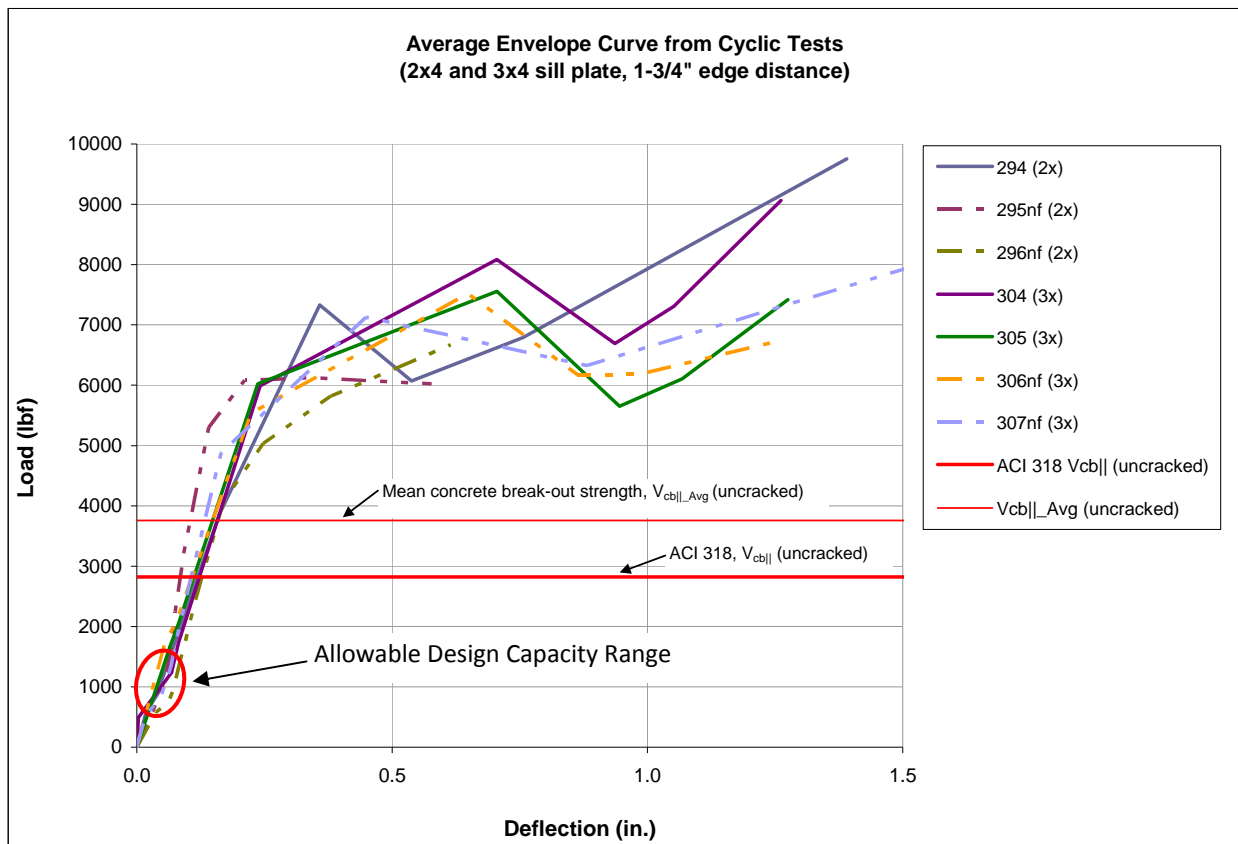


Chart 9 – Average envelope curves for 2x4 & 3x4 cyclic tests

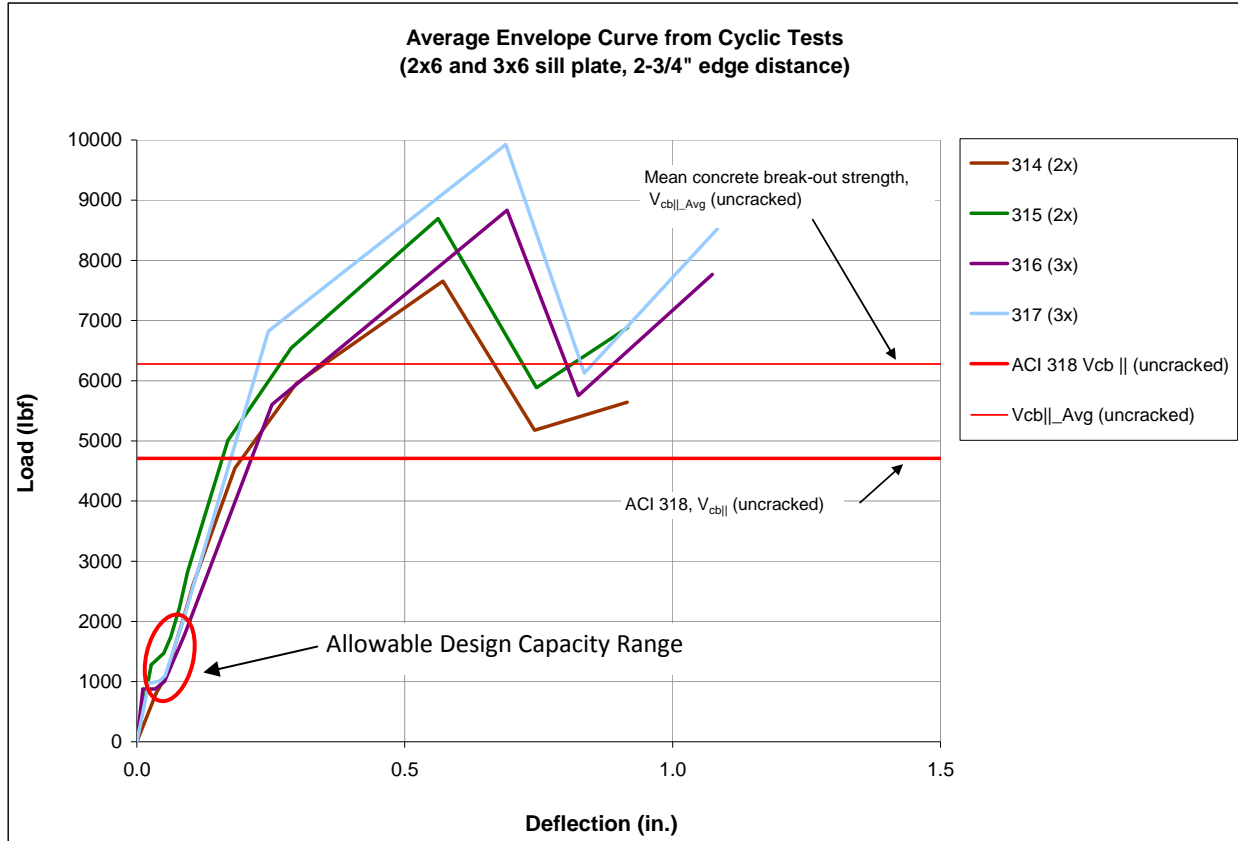


Chart 10 – Average envelope curves for 2x6 and 3x6 cyclic tests.

Comparison of calculated bolt capacities:

Design capacities from current previous versions of the design codes in **Table 7** are based on nominal dimensions and properties (e.g. $D=0.625"$, $f'_c=2500$ psi, $1-3/4"$ edge distance, "uncracked" concrete). The only adjustment factor included is C_D (as shown). Design capacities in recent editions of ACI 318-08, based on assumed non ductile behavior, are approximately 1/3 of those previously used for the design of the sill plate to foundation connection.

5/8" dia. bolt Code(s)	ASD Design Capacity (#) 2x4 DF-L (seasoned) (including C_D)	ASD Design Capacity (#) 3x4 DF-L (seasoned) (including C_D)	Comment
1991 UBC (NDS-86)	1306 # $C_D = 1.33$	1326 # $C_D = 1.33$	2510 (b), T-25-F
1994 UBC (NDS-91)	1173 # $C_D = 1.33$	1,492 # $C_D = 1.33$	2336.2.3, T-23-III-J NDS allows $C_D=1.6$.
1997 UBC (NDS-91)	1408 # $C_D = 1.60$	1790 # $C_D = 1.60$	2316, T-23-III-B-1
IBC-2003 (NDS-01)	1,424 # $C_D = 1.60$	1,824 # $C_D = 1.60$	NDS-01 Table 11E
IBC-2006 (NDS-05)	1,488 # $C_D = 1.60$	1,888 # $C_D = 1.60$	NDS-05 Table 11E
ACI-318-08 App. D, $f'_c = 2500$ psi.	Non ductile: 500 # Ductile: 1000 #	Non ductile: 500 # Ductile: 1000 #	0.5 factor used for non ductile behavior. Values assume "uncracked" concrete.
ACI-318-05 App. D, $f'_c = 2500$ psi.	Non ductile: 400 # Ductile: 1000 #	Non ductile: 400 # Ductile: 1000 #	0.4 factor used for non ductile behavior. Values assume "uncracked" concrete.
Peak values from average of Cyclic tests	Avg from test ID's: 294, 295-NF, 296-NF: 6710 # @ 0.44"	Avg from test ID's: 304, 305, 306-NF & 307-NF: 7572 # @ 0.63"	Peak cyclic test values are at least 4 times historic design values.

Table 7 - Comparison of allowable design capacities based on typical "nominal" inputs to average peak values from cyclic tests.

The cyclic tests show that for 2x4 and 3x4 plates, the average peak loads are at least 4 times historic allowable capacities (including C_D for 10 minutes).

Findings & Conclusions:

Recall, this project had the following primary goals:

1. Determine whether the wood controls the connection capacity when loaded parallel to the edge.

Yes, it does. The wood "yield" is the first material limit state. The "peak" however derived from the cyclic average envelope curves correlate strongly with the concrete delaminations (when detected in this testing program).

The connection went through the following phases which can be described qualitatively:

- initial take-up and displacement as the connection gets “seated”.
- elastic bolt bending combined with wood crushing (dowel bearing).
- plastic bolt bending combined with wood crushing. Some bolt stretching occurs. As the bolt deflects and goes into tension; a clamping force develops.
- plastic bolt bending combined with wood crushing + shallow concrete delamination. Clamping forces continue to develop as the bolt goes into tension.
- plastic bolt bending combined with wood crushing + shallow spalling of concrete at anchor.
- to the extent the sill plate splits; it occurs during the last 2 phases described above.

2. Determine whether the connection exhibits “ductile” behavior.

The connection behavior is clearly ductile. See **Chart 9** and **Chart10**. For additional discussion, see the SEAOC Seismology Committee’s Blue Book article on anchor bolts.

3. Propose design capacities for the connection based on this testing.

The final goal of this program was to propose design capacities for shear parallel to free edge in pounds (ASD) to be used to resist seismic loading in Seismic Design Categories C through F, (SDC C-F). It has been subsequently decided to defer this task to the full SEAOC Seismology Committee who has drafted a Blue Book article on this subject (available from <http://www.SEAOC.org/bluebook>). It should be noted that load values from these tests should be considered to be 10-minute values (including CD = 1.6).

In addition to the primary project goals, we also developed the following preliminary findings:

- Friction developed between the bottom of the wood plate is real and substantial. See **Chart 7**. This report has not focused on quantifying the effect. To be conservative, future testing should consider incorporating the friction-reducing membrane into testing protocols. In addition, the membrane may replicate as-built assemblies such as a sill plate with a sheet-metal termite shield.
- Delaminations and/or the formations of flaws within the concrete is often detectable during the testing if the impact-echo method is correctly applied. Since the visually-apparent spalls often formed some time after the initial flaw detection; impact-echo testing is recommended for future testing programs.
- A note for post-earthquake inspectors. Damage following cyclic loading was not readily observable when viewed from above, even with the nut and plate washer removed. The tested specimens experienced limited plate splitting and limited stretching of the bolt hole at the upper surface of the sill plate. The wood crushing and subsequent concrete delaminations are only visible when a section of sill plate is removed or when the outer concrete face is exposed/testable.

Acknowledgements:

The Simpson Strong-Tie Company (SSTC) generously donated the testing services to load and instrument the samples. SSTC also donated the procurement and construction of the specimens tested.

- Special thanks to the following SSTC engineers; Steve Pryor, Ricardo Arevelo and Tim Murphy.

The American Forest and Paper Association (AF&PA) provided analytical support throughout.

- Special thanks to the following to Phil Line, Shane Cochran and Brad Douglas.

The Structural Engineers Association of Northern California (SEAONC) provided a \$10,000 grant through their 2008 Special Projects Initiative.

- Special thanks to the 2008 SEAONC Board of Directors; Reinhard Ludke (President), Rafael Sabelli (Vice-President), Kate Stillwell (Treasurer), Bret Lizurdia (Past-President), Greg Deierlein, Mark Ketchum, Karin Kuffel, and John Osteraas.

Several Technical Committees from the Structural Engineers Association of California (SEAOC) provided technical oversight throughout the testing protocol development and the report preparation.

- Thanks to the 2008-2009 Seismology and Structural Standards Committee; Kevin Moore (Chair), Mehran Pourzanjani (Vice-Chair), John Diebold (past-Chair), Andy Fennell, Tom Hale, Ryan Kersting, Doug Magee, Nic Rodrigues, James Lai, Geoff Bomba, and Tom VanDorpe
- Thanks to the 2008-2009 Light-Frame Construction Subcommittee; Gary Mochizuki (Chair), Tom VanDorpe, Norm Scheel, Chris Kamp, and Andy Fennell.

The following individuals also provided valuable technical support; Mark Moore, Robert Kent, Achim Groess.

Selected References:

1. Canadian Journal of Civil Engineering, 2003 - Lateral resistance of bolted wood-to-concrete connections loaded parallel or perpendicular to grain. Mohammad, M.; Karacabeyli, E.; and Quenneville, J.H.P.
2. CUREE Publication W-02 – Development of a testing protocol for wood-frame structures.
https://secure.curee.org/catalog/index.php?main_page=product_info&products_id=4
3. VPI Research Report No. TE-1994-003 - Determination of short-term duration-of-load performance of nailed and bolted connections using sequential phased displacement tests. Virginia Polytechnic Institute and State University, Blacksburg, VA. Dolan, J.D.; Gutshall S.T.; and McLain T.E. 1996b.
<http://swst.metapress.com/content/xl785t72261h52jr/>
4. 1991 International Timber Engineering Conference, London - United States adaptation of European Yield Model to large diameter dowel fasteners specifications. Soltis, L.A.; Wilkinson, T.L.
<http://www.fpl.fs.fed.us/documnts/pdf1991/solti91a.pdf>
5. ASTM Standard D 5764 - 97a, 2007 - Standard Test Method for evaluating dowel-bearing strength of wood and wood-based products. ASTM International, West Conshohocken, PA.
<http://www.astm.org/Standards/D5764.htm>

6. ASTM Standard E 2126, 2008 - Standard Test Methods for cyclic (reversed) load test for shear resistance of vertical elements of the lateral force resisting systems for buildings. ASTM International, West Conshohocken, PA. <http://www.astm.org/Standards/E2126.htm>
7. FEMA 461 - Interim Protocols for determining seismic performance characteristics of structural and nonstructural components through laboratory testing. May 2007. <http://www.atcouncil.org/pdfs/FEMA461.pdf>
8. ASTM Standard C 1383, 2004 - Standard Test Methods for measuring the P-Wave speed and the thickness of concrete plates using the impact-echo method. ASTM International, West Conshohocken, PA. <http://www.astm.org/Standards/C1383.htm>
9. Fuchs, W., Eligehausen, R., and Breen, J. E., "Concrete Capacity Design (CCD) Approach for Fastening to Concrete," *ACI Structural Journal*, V. 92, No. 1, January-February 1995, pp. 73-94.

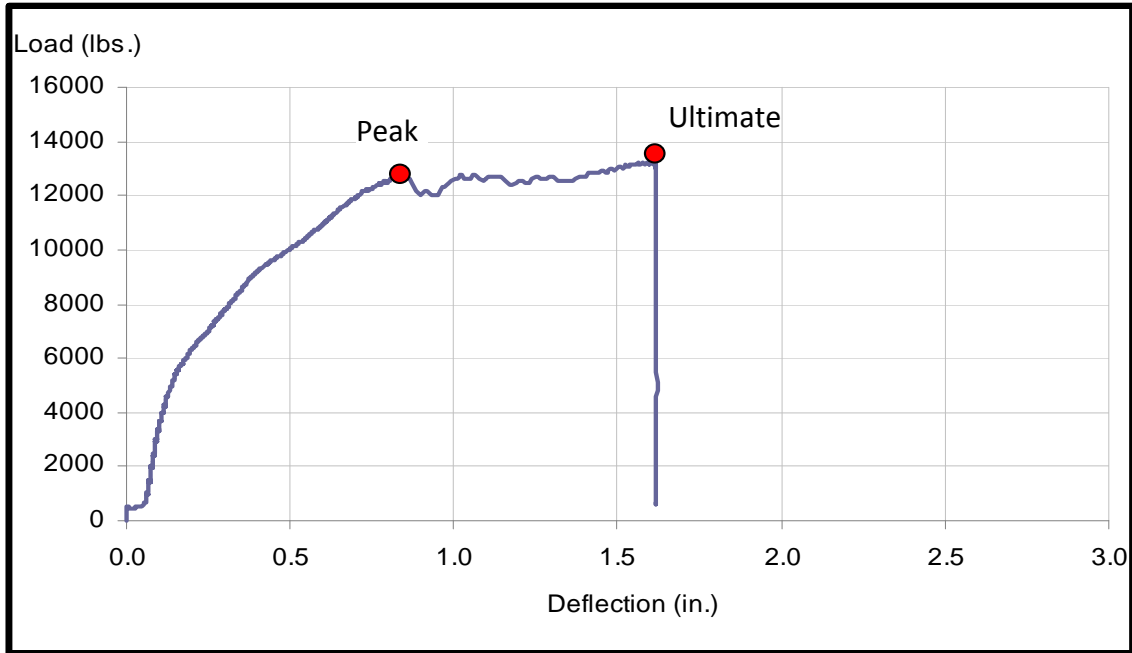
Attachments:

Appendix Table A – Summary of test data and observations, (2 pages).

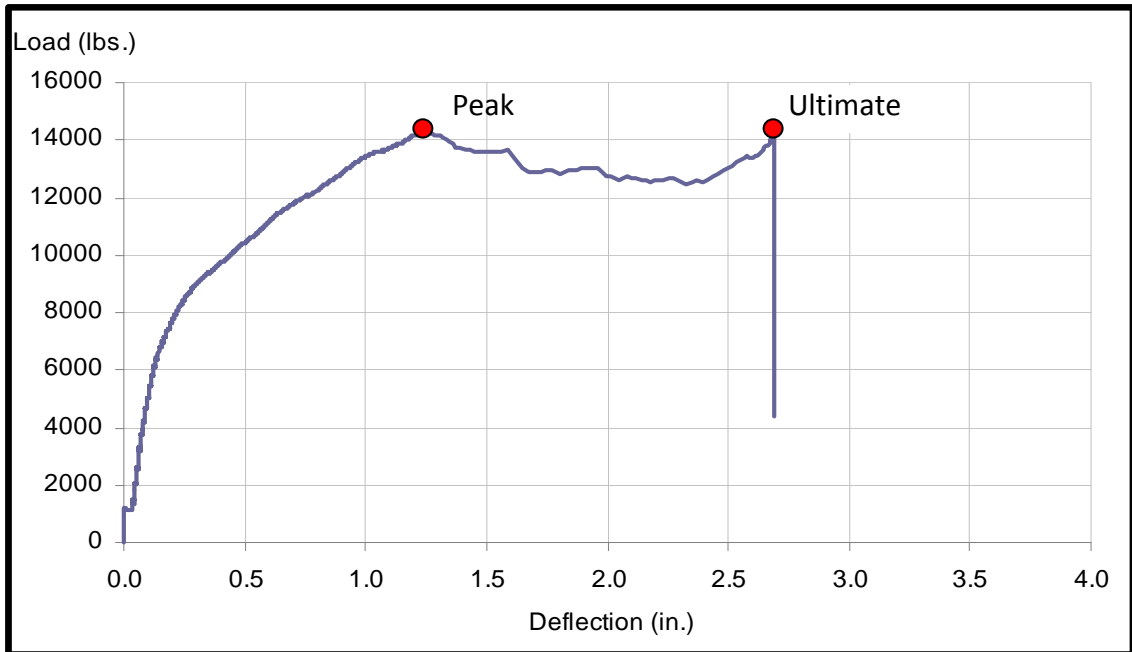
Appendix B – Graphs of peak and ultimate data as tabulated in **Table 4**, (17 pages)

Test ID's Test Date	Test Summary	Anchor Bolt Edge Distance: Nominal, Actual.	Testing Protocol	Moisture Content, Lumber grade PT Treatment.	Observations / Testing Notes			Additional Notes
					Concrete (Main Member)	Wood Sill (Side Member)	Anchorage	
								f'c 11/12/08, 2500 psi. f'c 11/19/08, 2710 psi
1-A-1-f Simpson 289 11/12/08	2x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.9"	Monotonic 250#/sec.	9.1 % to 9.7 %, DF-Standard & Better, Borate.	Shallow delam + spall ≅ N/A . See note. No bolt pull-out.	No split.	After test; plate bent & nut tight.	Hydraulic/loading protocol issues prevented delamination detection.
1-A-2-f Simpson 290 11/12/08	2x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.8"	Monotonic 250#/sec.	8.4 %, DF-Standard & Better, Borate.	Shallow delam + spall ≅ N/A . See note. No bolt pull-out.	Split at grain at adjacent knot. No split at bolt.	After test; plate bent & nut tight.	Hydraulic/loading protocol issues prevented delamination detection.
2-A-1-f Simpson 293 11/12/08	2x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.9"	Cyclic SEAOC @ 50#/sec.	7.9 % to 8.5 %, DF-Standard & Better, Borate.	Shallow delam + spall ≅ N/A . See note. No bolt pull-out.	Split at bolt.	After test; plate bent & nut tight.	Stopped at cycle CUREE-2 due to hydraulic issue.
2-A-2-f Simpson 294 11/12/08	2x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.7"	Cyclic (0.2 Hz) SEAOC-Modified. See 2-A-2-f note.	7.5 % to 8.1 %, DF-Standard & Better, Borate.	Shallow delam + spall ≅ N/A . See note. No bolt pull-out.		After test; plate bent & nut tight.	Due to hydraulic limits; changed to displacement-controlled loading protocol.
1-C-1-nf Simpson 291 11/12/08	2x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.9"	Monotonic 250#/sec.	6.5 % to 8.1 %, DF-Standard & Better, Borate.	Shallow delam + spall ≅ N/A . See note. No bolt pull-out.	Split at bolt.	After test; plate bent & nut tight.	No Impact-Echo available (no access).
1-C-2-nf Simpson 292 11/12/08	2x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.9"	Monotonic 250#/sec.	7.0 % to 8.3 %, DF-Standard & Better, Borate.	Shallow delam + spall ≅ 7800# (≅ 0.69"). No bolt pull-out.	No split.	After test; plate bent & nut tight.	
2-C-1-nf Simpson 295	2x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.8"	Cyclic (0.2 Hz) SEAOC-Modified. See 2-A-2-f note.	9.1 % to 10.2 %, DF-Standard & Better, Borate.	Shallow delam + spall ≅ 6100# (0.34"); -6200# (-0.34"). No bolt pull-out.	Split at bolt.	After test; plate bent & nut tight.	
2-C-2-nf Simpson 296 11/13/08	2x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.9"	Cyclic (0.2 Hz) SEAOC-Modified. See 2-A-2-f note.	7.5 % to 8.1 %, DF-Standard & Better, Borate.	No spall/delam. Confirmed by Impact- Echo. Test stopped early at: ≅ 6600# (0.63") & -6700# (-0.60").	No split.	After test; plate bent & nut tight.	See also SPD-1-f for second test of this anchor (with new wood).
1-B-1-f Simpson 298 11/13/08	3x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.9"	Monotonic 0.75"/min.	12.1 % to 13.0 %, DF-Standard & Better, Borate.	Shallow delam ≅ 8500# (≅ 0.56"). Concrete spall. No bolt pull-out.	Checking at/near bolt.	After test; plate bent & nut tight.	
1-B-2-f Simpson 299 11/13/08	3x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.8"	Monotonic 0.75"/min.	12.5 %, DF-Standard & Better, Borate.	Shallow delam ≅ 7500# (≅ 0.33"). Concrete spall. No bolt pull-out.	No split.	After test; plate bent & nut tight.	Test stopped. P _{MAX} 13k at 1.3".
2-B-1-f Simpson 304 11/14/08	3x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 2.0"	Cyclic (0.2 Hz) SEAOC-Modified. See 2-A-2-f note.	10.6 % to 12.3 %, DF-Standard & Better, Borate.	Shallow delam + spall ≅ 9600# (0.69"); -6500# (-0.72"). No bolt pull-out.	No split.	Nut loose. Plate bent.	Delam noted on first 0.7Q ₀ cycle. Compare to Test 306 for "nf".
2-B-2-f Simpson 305 11/14/08	3x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.7"	Cyclic (0.2 Hz) SEAOC-Modified. See 2-A-2-f note.	10.7 % to 11.8 %, DF-Standard & Better, Borate.	Shallow delam + spall ≅ 8900# (0.70"); -6200# (-0.71"). No bolt pull-out.	Split starting in line with bolt.	Nut loose. Plate bent.	Delam noted on first 0.7Q ₀ cycle. Compare to Test 306 for "nf".
1-D-1-nf Simpson 300 11/13/08	3x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.9"	Monotonic 0.75"/min.	10.1 % to 12.4 %, DF-Standard & Better, Borate.	Shallow delam ≅ 7000# (≅ 0.66"). Concrete spall. No bolt pull-out.	No split. Plate dented by plate washer.	After test; plate bent & nut tight.	Test stopped. P _{MAX} 9.5k at 3".
1-D-2-nf Simpson 301 11/13/08	3x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.8"	Monotonic 0.75"/min.	11.2 %, DF-Standard & Better, Borate.	Shallow delam ≅ 7000# (≅ 0.44"). Concrete spall. No bolt pull-out.	Split at bolt. Plate dented by plate washer.	After test; plate bent & nut tight.	Test stopped. P _{MAX} 12.5k at 3".
2-D-1-nf Simpson 306 11/14/08	3x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.8"	Cyclic (0.2 Hz) SEAOC-Modified. See 2-A-2-f note.	10.9 % to 11.1 %, DF-Standard & Better, Borate.	Shallow delam. ≅ 6500 # (0.71"); -5500# (-0.71"). No bolt pull-out. No concrete spall.	No split.	Nut loose. Plate bent.	Input error. Used same protocol as for 304 & 305 (3x4-f tests).
2-D-2-nf Simpson 307 11/14/08	3x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.9"	Cyclic (0.2 Hz) SEAOC-Modified. See 2-A-2-f note.	9.0 % to 9.1 %, DF-Standard & Better, Borate.	Shallow delam + spall ≅ 6100# (0.48"); -6000# (-0.49"). No bolt pull-out.	(E) checking prior to test. Split widened during test.	Nut loose. Plate bent.	Delam noted on second 0.7Q ₀ cycle.

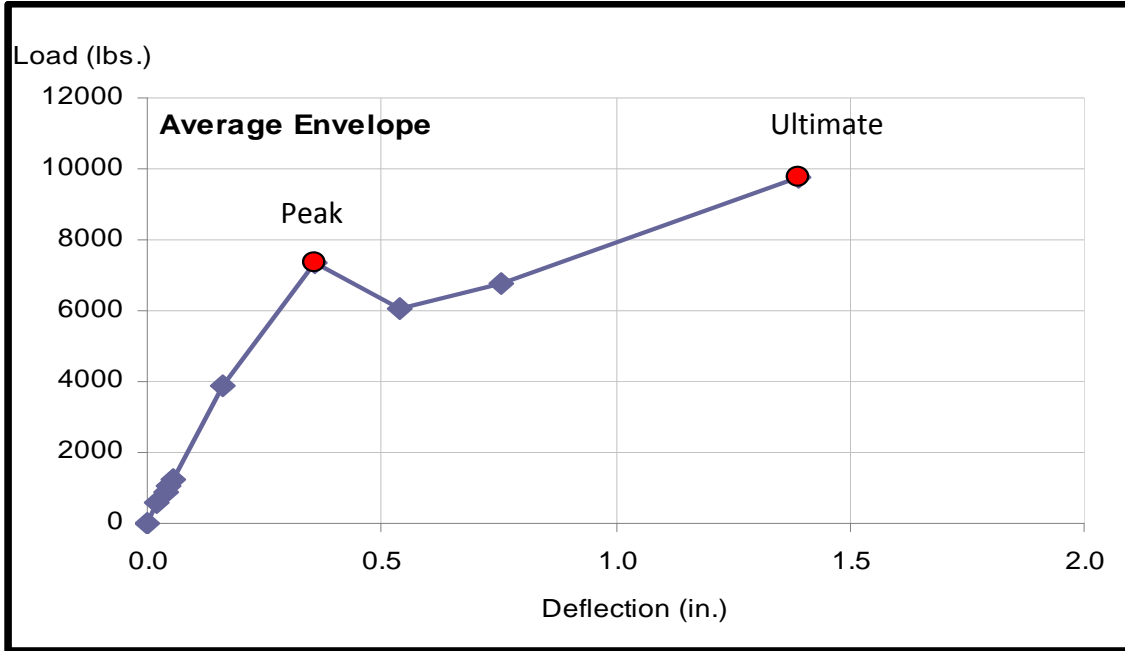
Test ID's Test Date	Test Summary	Anchor Bolt Edge Distance: Nominal, Actual.	Testing Protocol	Moisture Content, Lumber grade PT Treatment.	Observations / Testing Notes			Additional Notes
					Concrete (Main Member)	Wood Sill (Side Member)	Anchorage	
4-A-1-f Simpson 310 11/14/08	2x6 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	2.75" 2.6"	Monotonic 0.75"/min.	14.0 % to 17.4 %, DF-#2, Borate.	Shallow delam between 15k ($\approx 1.25"$) & 16.3k ($\approx 1.53"$). No concrete spall. No bolt pull-out.	Plate split late in test.	After test; plate bent & nut tight. Some plate crushing below washer.	f'c 11/12/08, 2500 psi. f'c 11/19/08, 2710 psi Impact-Echo stopped 117 seconds into the test. No concrete delam at that time.
4-A-2-f Simpson 311 11/14/08	2x6 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	2.75" 2.7"	Monotonic 0.75"/min.	17.6 % to 18.2 %, DF-#1 or Better, Borate.	No delam at $\approx 14000\#$ ($\approx 1.33"$). No spall of course. No bolt pull-out.	Plate split late in test.	After test; plate bent & nut tight.	Impact-Echo stopped 152 seconds into the test. No concrete delam at that time.
4-C-1-f Simpson 314 11/19/08	2x6 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	2.75" 2.6"	Cyclic (0.2 Hz) SEAOC-Modified. See 2-A-2-f note.	14.0 % to 17.4 %, DF-#2, Borate.	No delamination. No concrete spall.	Plate split.	After test; Nut loose & Plate bent.	
4-C-2-f Simpson 315 11/19/08	2x6 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	2.75" 2.4"	Cyclic (0.2 Hz) SEAOC-Modified. See 2-A-2-f note.	17 %, DF-#1 or Better, Borate.	No delamination. No concrete spall.	Plate splitting just starting at end of test.	After test; Nut loose & Plate bent.	Video from rear incorrectly notes test ID.
4-B-1-f Simpson 312 11/14/08	3x6 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	2.75" 2.7"	Monotonic 0.75"/min.	14 %, DF-#1 or Better, Borate.	Shallow delam + spall $\approx 16400\#$ ($\approx 1.53"$). No bolt pull-out.	Plate split late in test.	After test; plate bent & nut tight.	Video from rear incorrectly notes test as 4-C-1-f.
4-B-2-f Simpson 313 11/14/08	3x6 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	2.75" 2.9"	Monotonic 0.75"/min.	9.2 %, DF, Grade N/A, ACQ.	Test stopped before delam. could form. See note.	Plate split late in test.	After test; plate bent & nut tight.	Test stopped; failing at "grips" (fastener tear-out). P _{MAX} 15.6k @ 1.5" Video from rear incorrectly notes test as 4-C-2-f.
4-D-1-f Simpson 316 11/19/08	3x6 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	2.75" 2.6"	Cyclic (0.2 Hz) SEAOC-Modified. See 2-A-2-f note.	10.1 %, DF-#1 or Better, Borate.	No delamination. No concrete spall.	No split.	After test; Nut loose & Plate bent.	
4-D-2-f Simpson 317 11/19/08	3x6 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	2.75" 2.7"	Cyclic (0.2 Hz) SEAOC-Modified. See 2-A-2-f note.	11.6 %, DF, Grade N/A, ACQ.	No delamination. No concrete spall.	Plate splitting just starting at end of test.	Bolt broke approx. 1" below the top of concrete late in test.	
Spare Tests								
Spare 1-f Simpson 308 11/14/08	3x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer Loose nut O/S hole = 0.75" f	1.75" 1.9"	Cyclic (0.2 Hz) SEAOC-Modified. See 2-A-2-f note.	10.1 % to 11.8 %, DF-Standard & Better, Borate.	Shallow delam + spall $\approx 8400\#$ (0.78"); -6600# (-0.77"). No bolt pull-out.	(E) checking prior to test. Did not widen during test.	Additional nut looseness.	
Spare Test Spare 2-f Simpson 309 11/14/08	3x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.7"	Cyclic (0.2 Hz) SPD (D control). Input error.	8.8 %, DF-Standard & Better, Borate.	Shallow delam detected during "mega- wave" (between 0.60" & 1.00"). See note. At 0.60", load was 9k. Concrete spall. No bolt pull-out.	No split.	Nut loose. Plate bent.	1 cycle incorrectly specified produced a "mega-wave". Say 15/16" instead of 5/16". See photo WAF_111408_120.
Spare Test SPD-1-f Simpson 302 11/13/08	(N) 2x4 sill plate. Used same anchor tested in 2-C-2-nf. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.9" Same as 2-C-2-nf	Cyclic (0.2 Hz) SPD (D control).	9.5 %, DF-Standard & Better, Borate.	Shallow delam + spall $\approx 5200(0.53")$; -6400# (-0.52"). No bolt pull-out.	No split.	After test; plate bent & nut tight.	See also 2-C-2-nf.
Spare Test SPD-2-f Simpson 303 11/13/08	2x4 sill plate. 5/8" A/B (0.559") 3"x3"x0.229" washer	1.75" 1.8"	Cyclic (0.2 Hz) SPD (D control).	8.8 %, DF-Standard & Better, Borate.	Shallow delam + spall $\approx 5600(0.36")$; -4900# (-0.34"). No bolt pull-out.	No split.	After test; plate bent & nut tight.	Spare test. Other spare anchor on this block not used due to (E) damage.



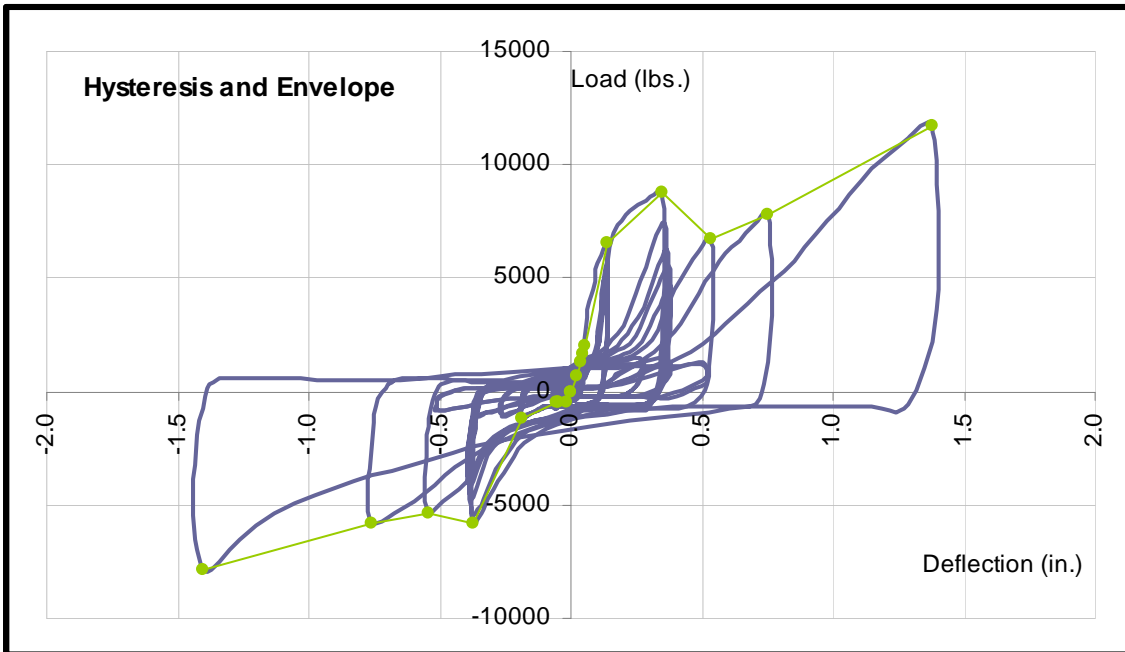
Test ID: 1-A-1-f-289, 2x4 Monotonic



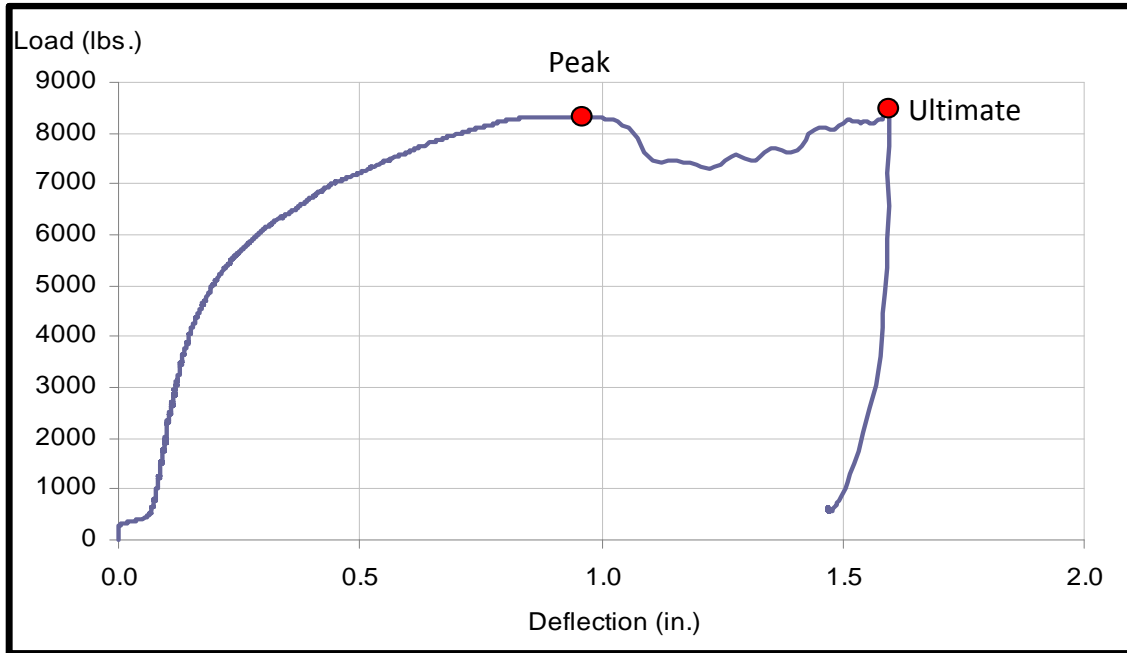
Test ID: 1-A-2-f-290, 2x4 Monotonic



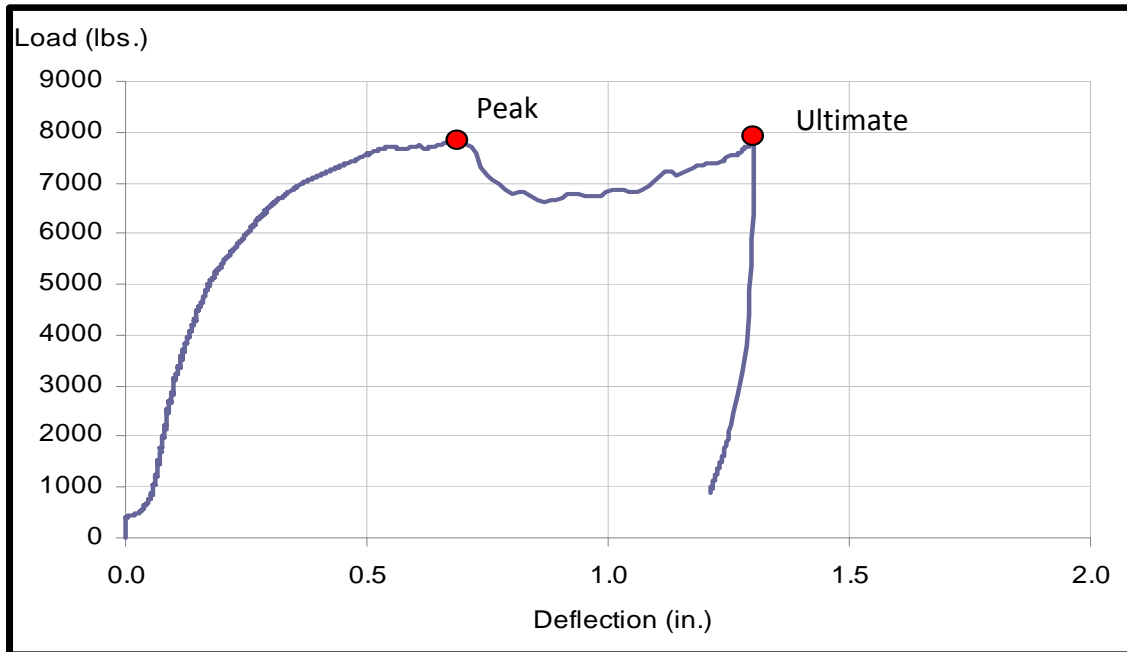
Test ID: 2-A-2-f-294, 2x4 Cyclic



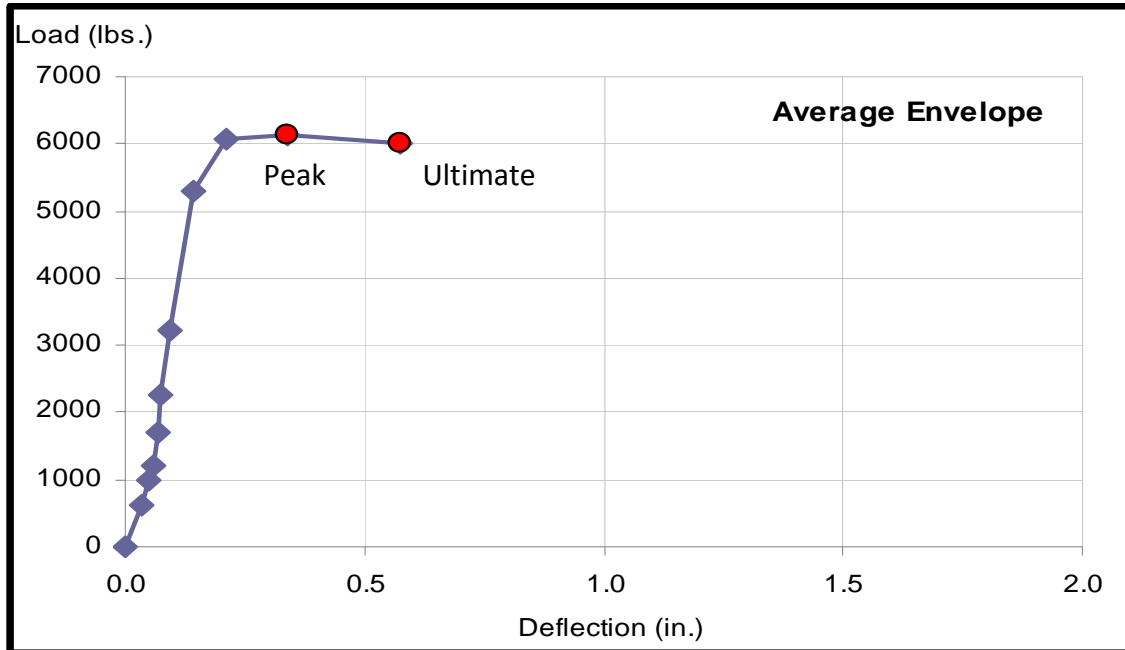
Test ID: 2-A-2-f-294, 2x4 Cyclic



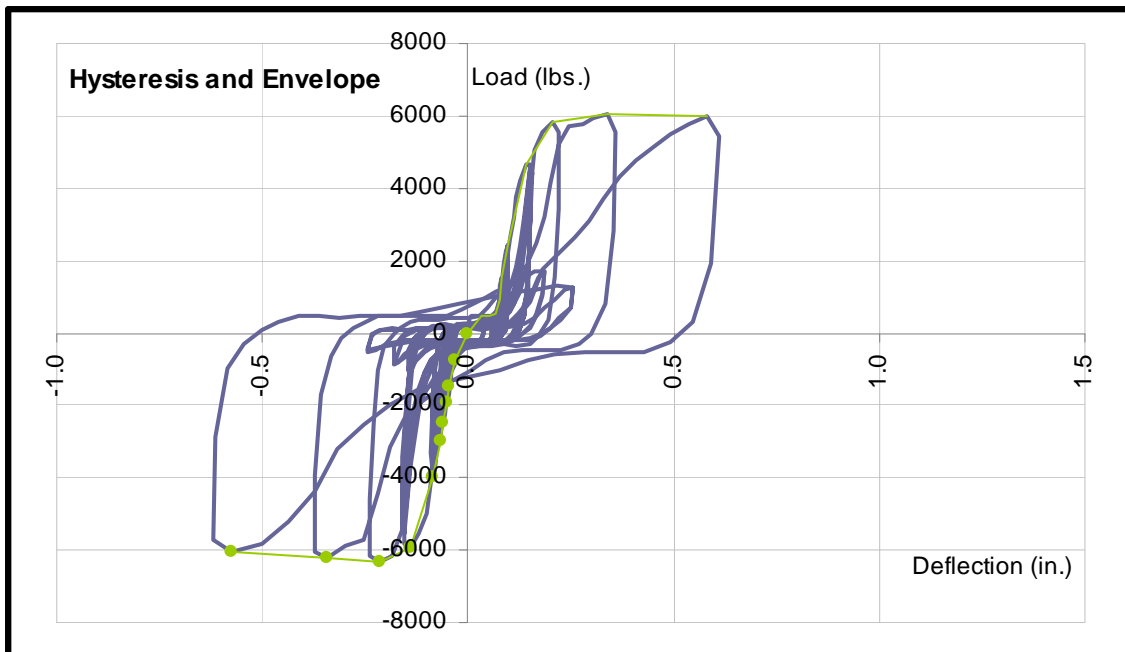
Test ID: 1-C-1-nf 291, 2x4 Monotonic



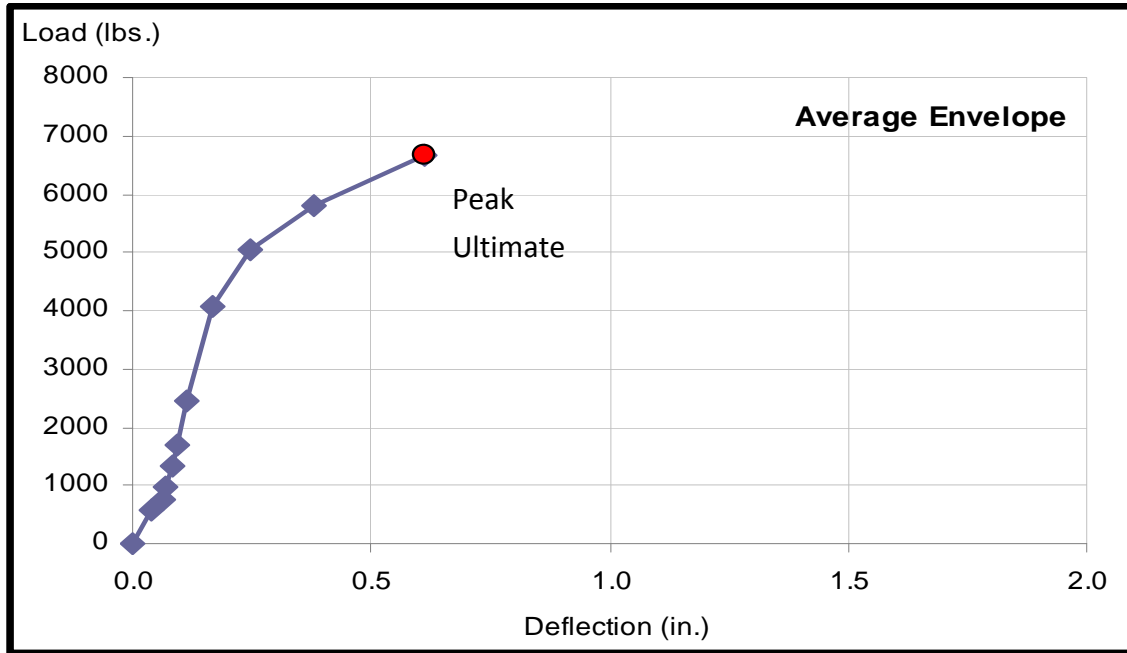
Test ID: 1-C-2-nf 292, 2x4 Monotonic



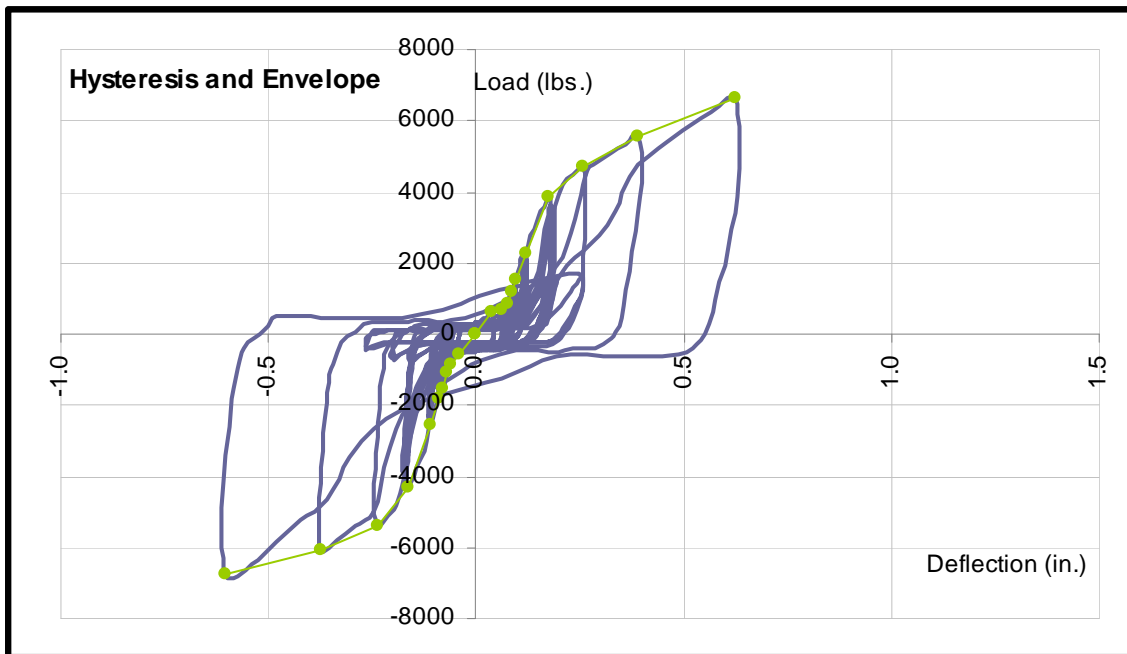
Test ID: 2-C-1-nf 295, 2x4 Cyclic



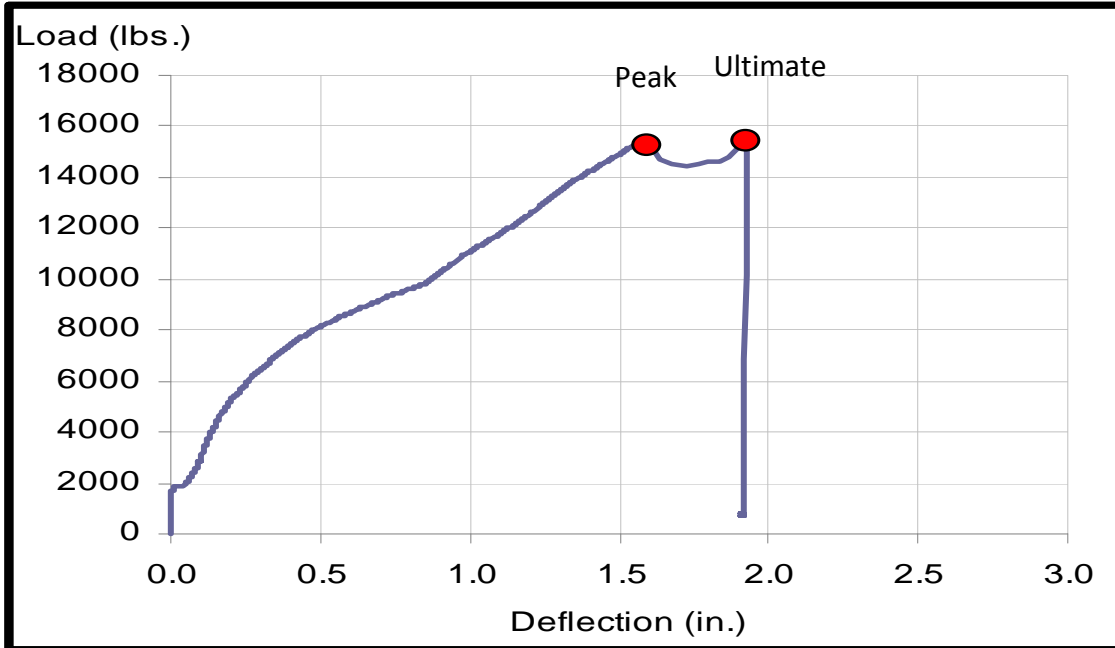
Test ID: 2-C-1-nf 295, 2x4 Cyclic



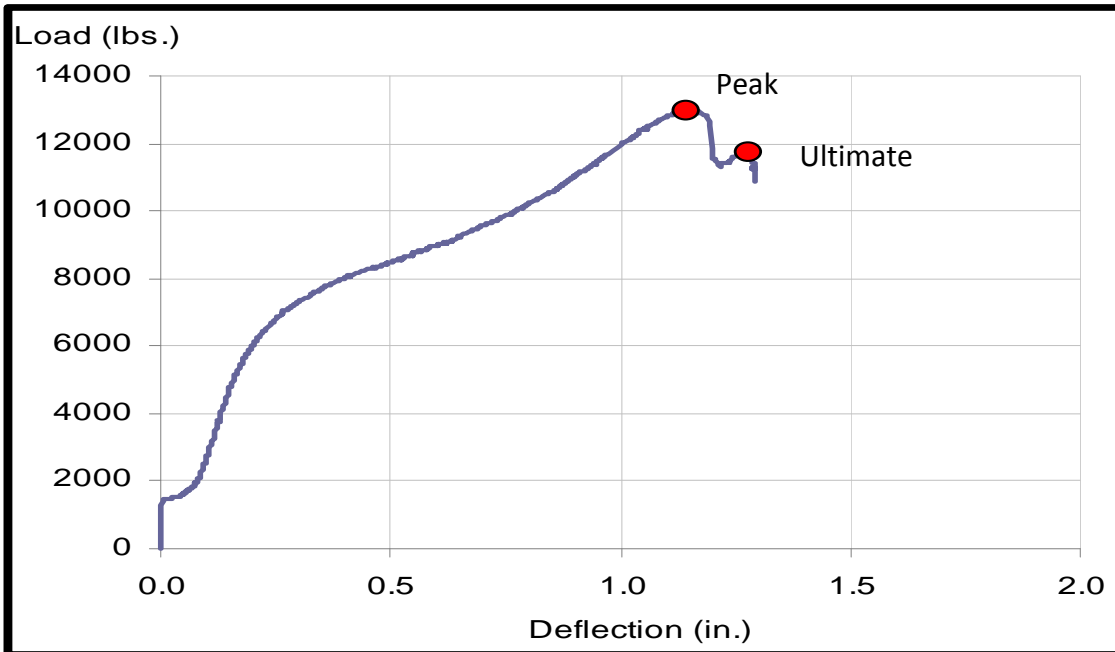
Test ID: 2-C-2-nf 296, 2x4 Cyclic



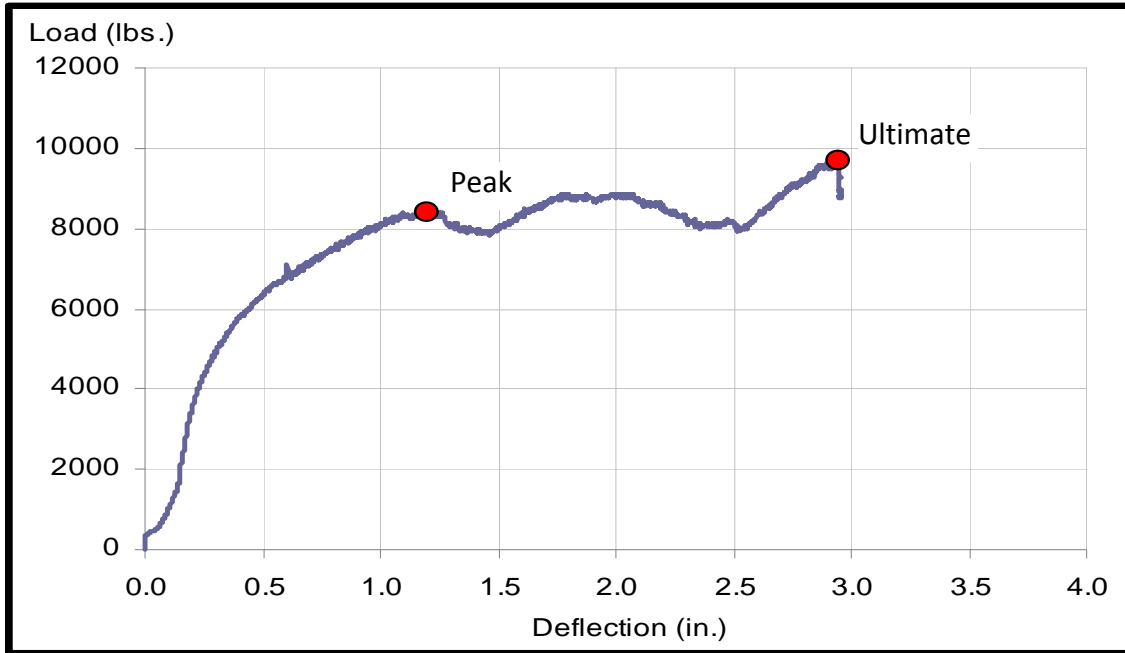
Test ID: 2-C-2-nf 296, 2x4 Cyclic



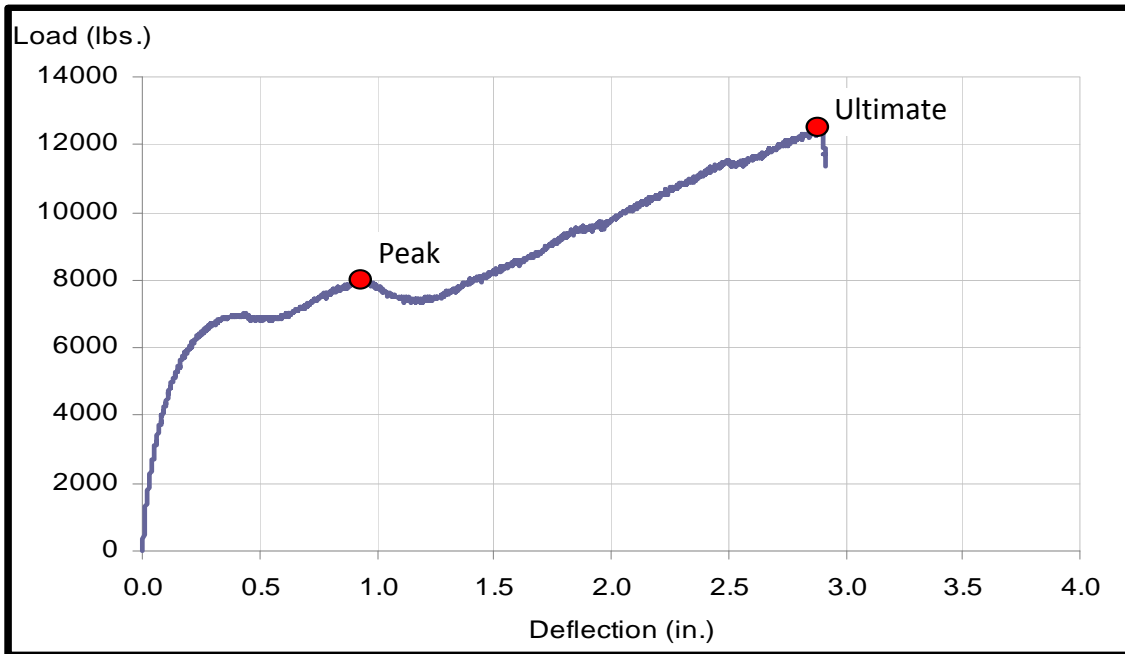
Test ID: 1-B-1-f 298, 3x4 Monotonic



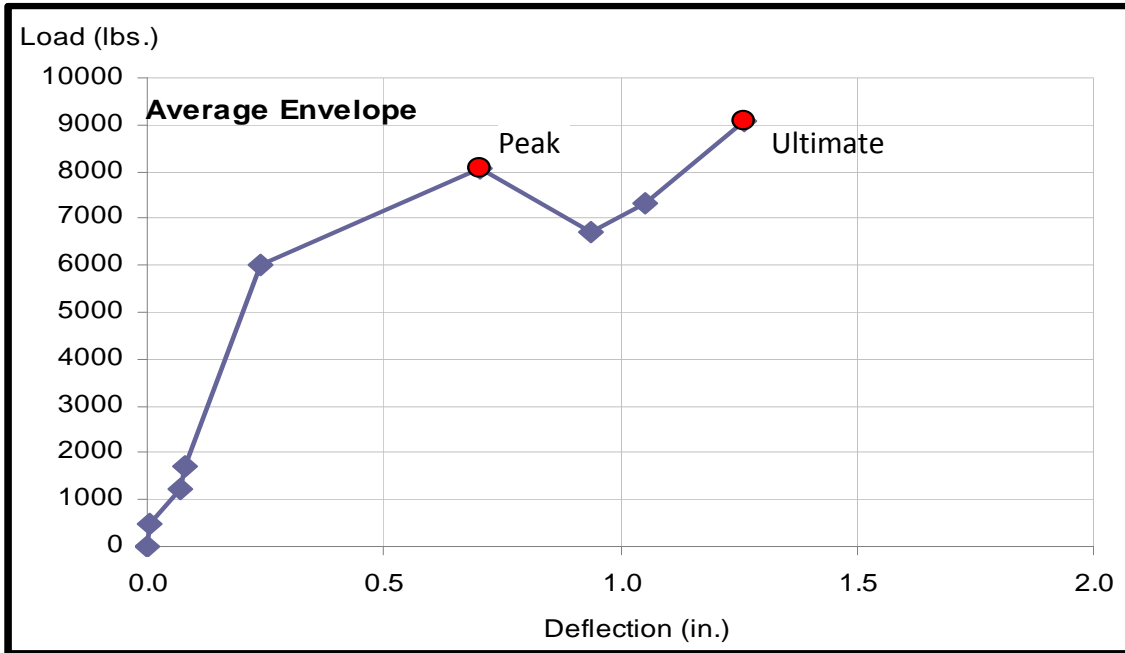
Test ID: 1-B-2-f 299, 3x4 Monotonic



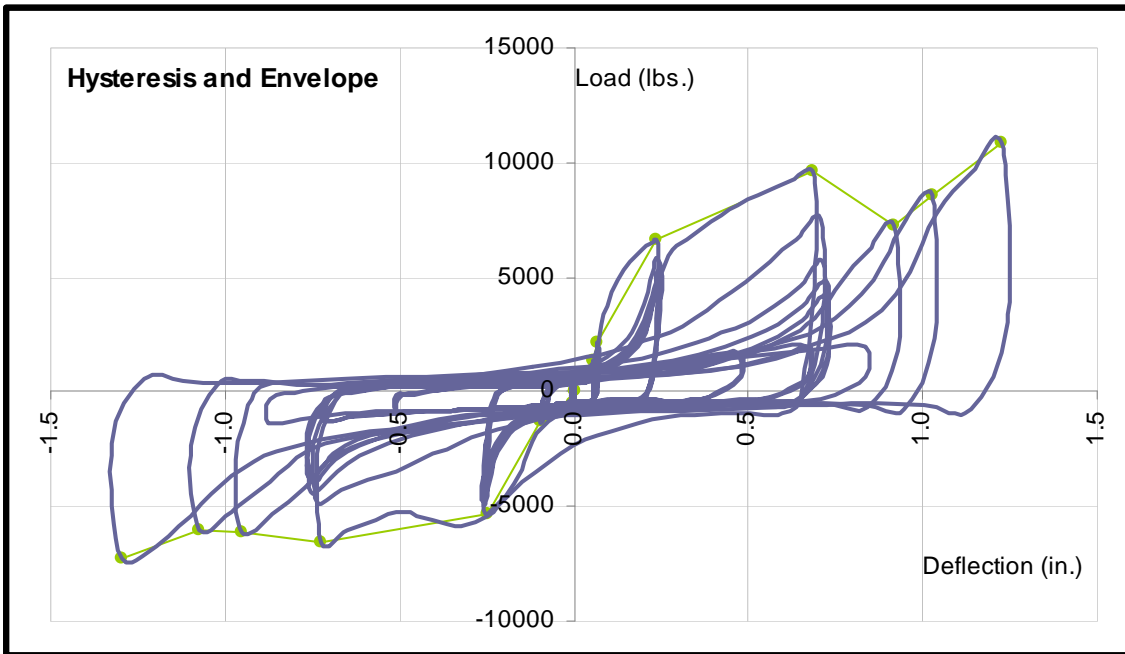
Test ID: 1-D-1-nf 300, 3x4 Monotonic



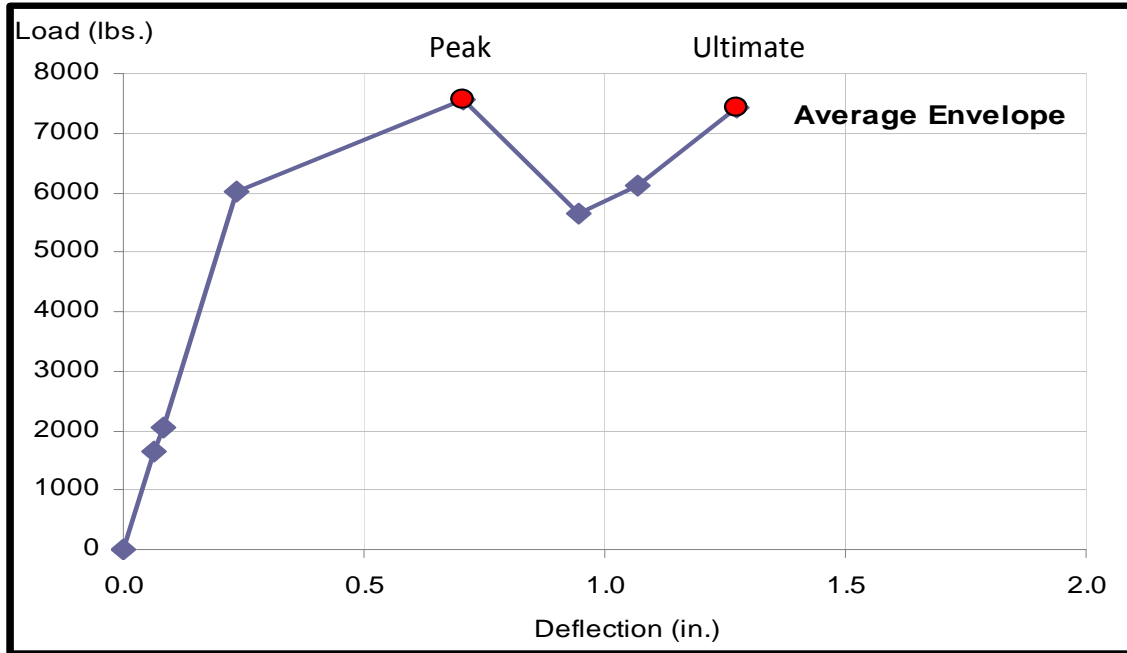
Test ID: 1-D-2-nf 301, 3x4 Monotonic



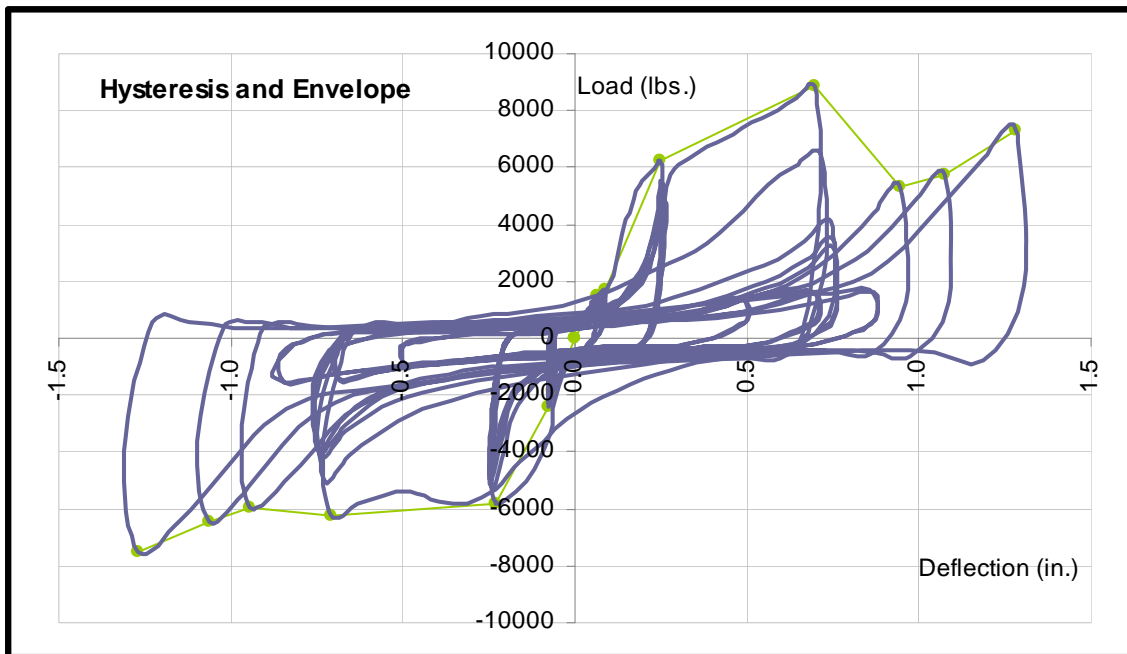
Test ID: 2-B-2-f 304, 3x4 Cyclic



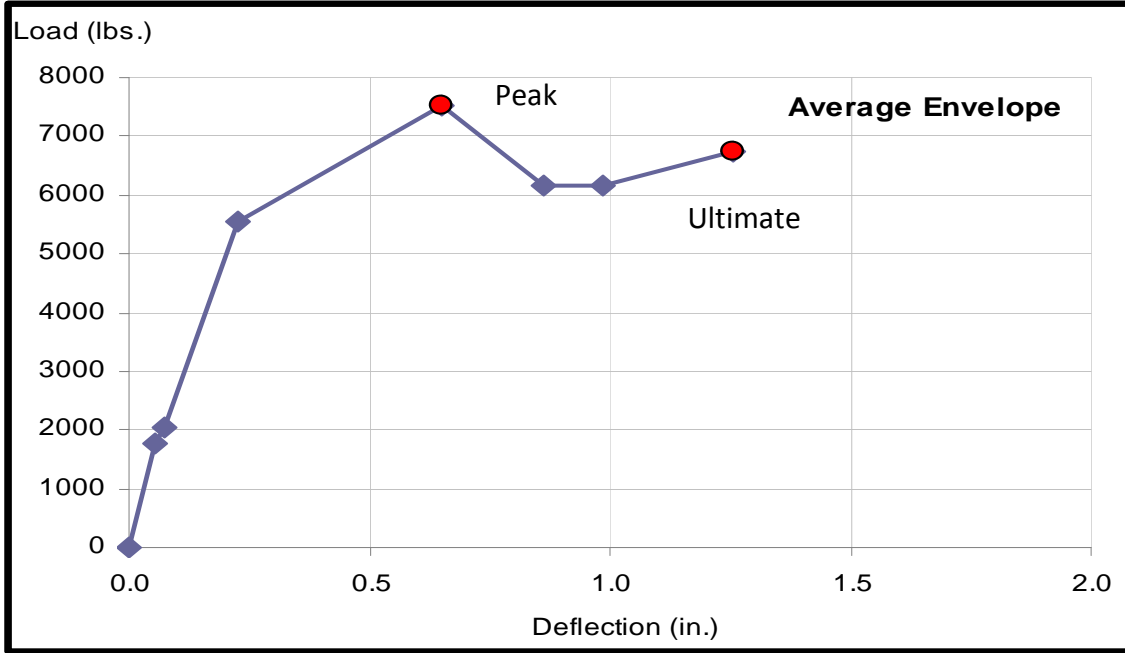
Test ID: 2-B-2-f 304, 3x4 Cyclic



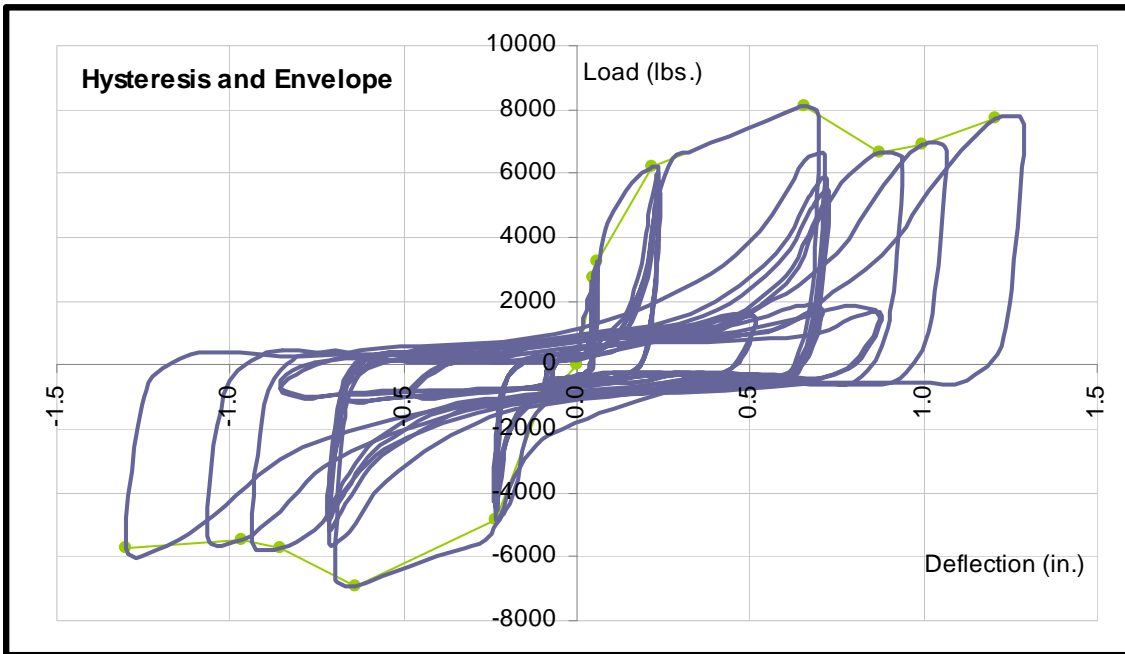
Test ID: 2-B-2-f 305, 3x4 Cyclic



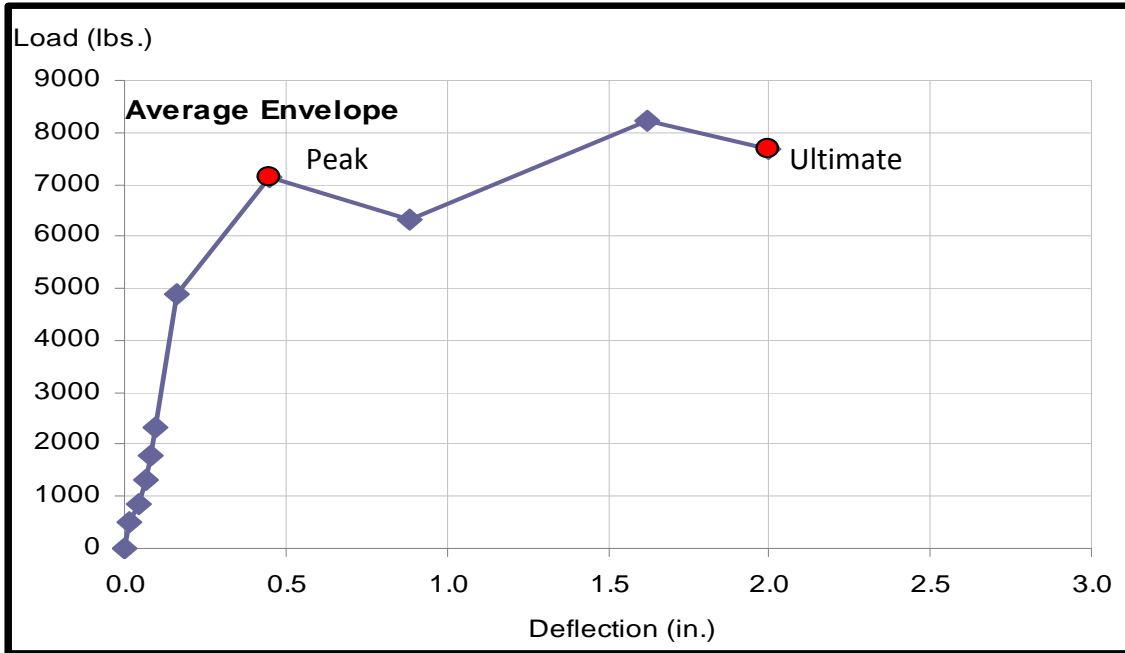
Test ID: 2-B-2-f 305, 3x4 Cyclic



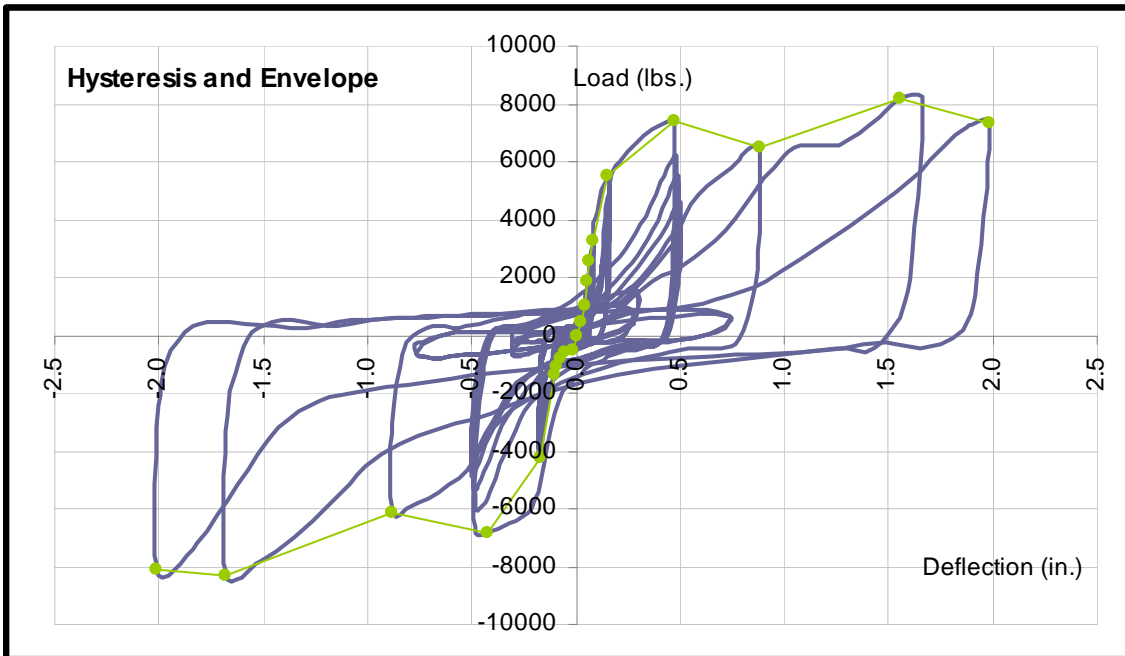
Test ID: 2-D-1-nf 306, 3x4 Cyclic



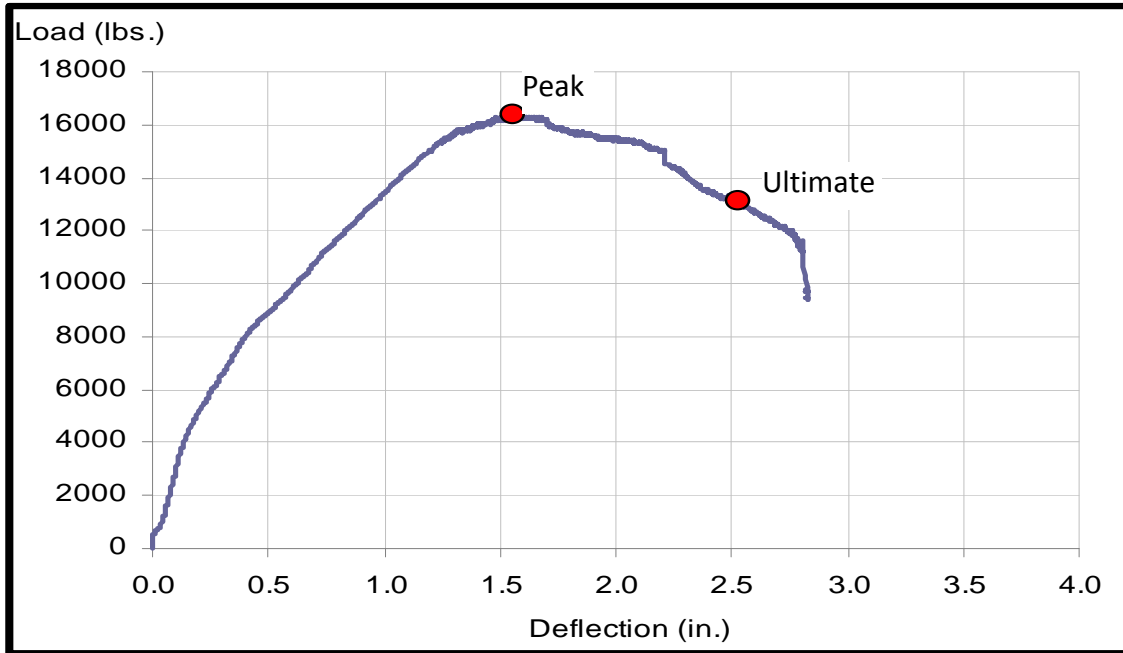
Test ID: 2-D-1-nf 306, 3x4 Cyclic



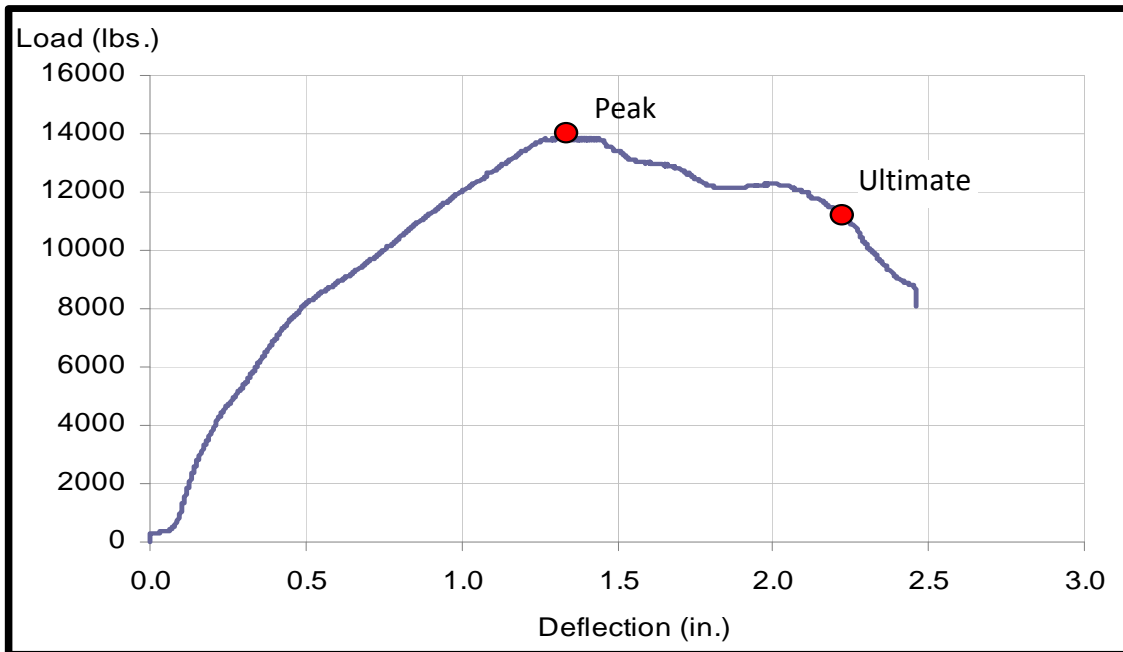
Test ID: 2-D-2-nf 307, 3x4 Cyclic



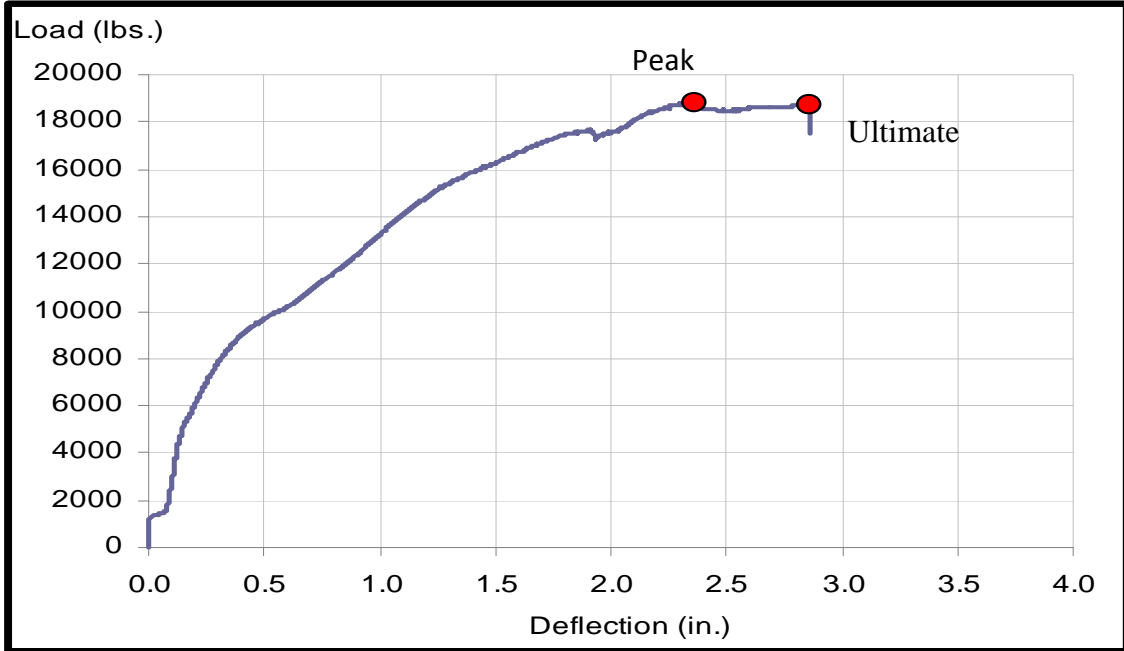
Test ID: 2-D-2-nf 307, 3x4 Cyclic



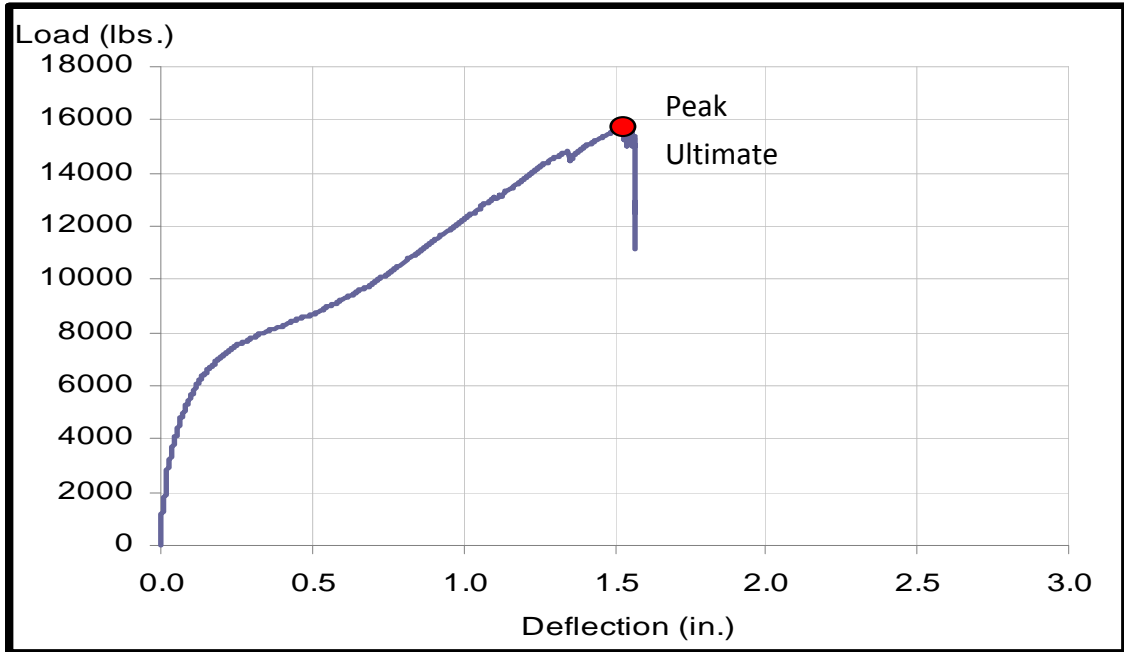
Test ID: 4-A-1-f 310, 2x6 Monotonic



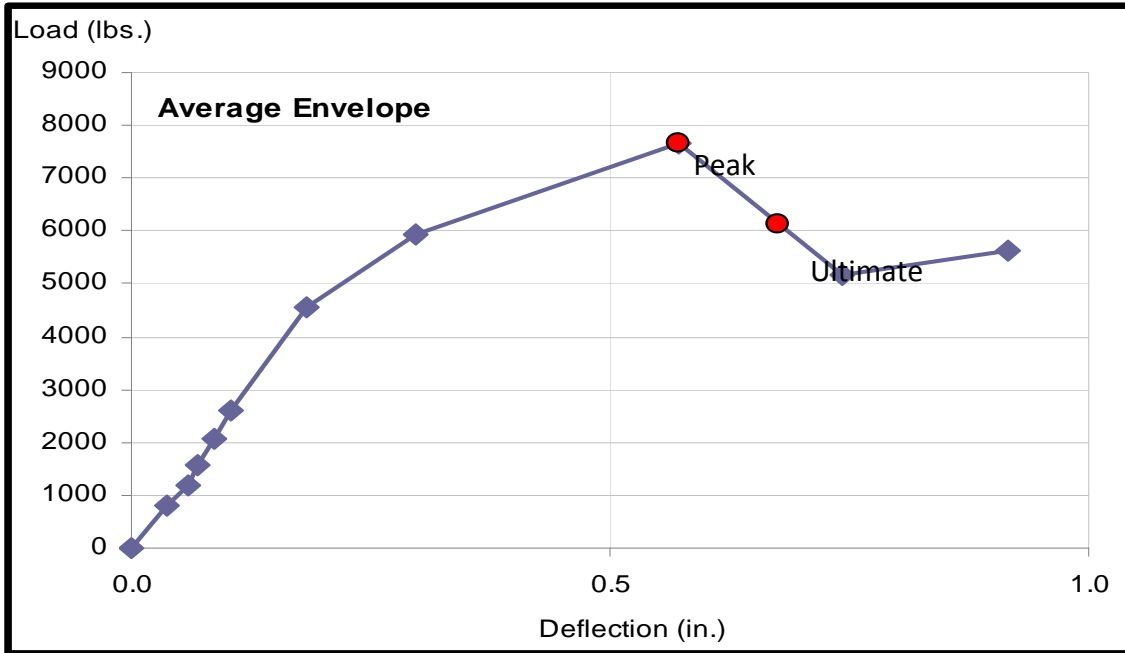
Test ID: 4-A-2-f 311, 2x6 Monotonic



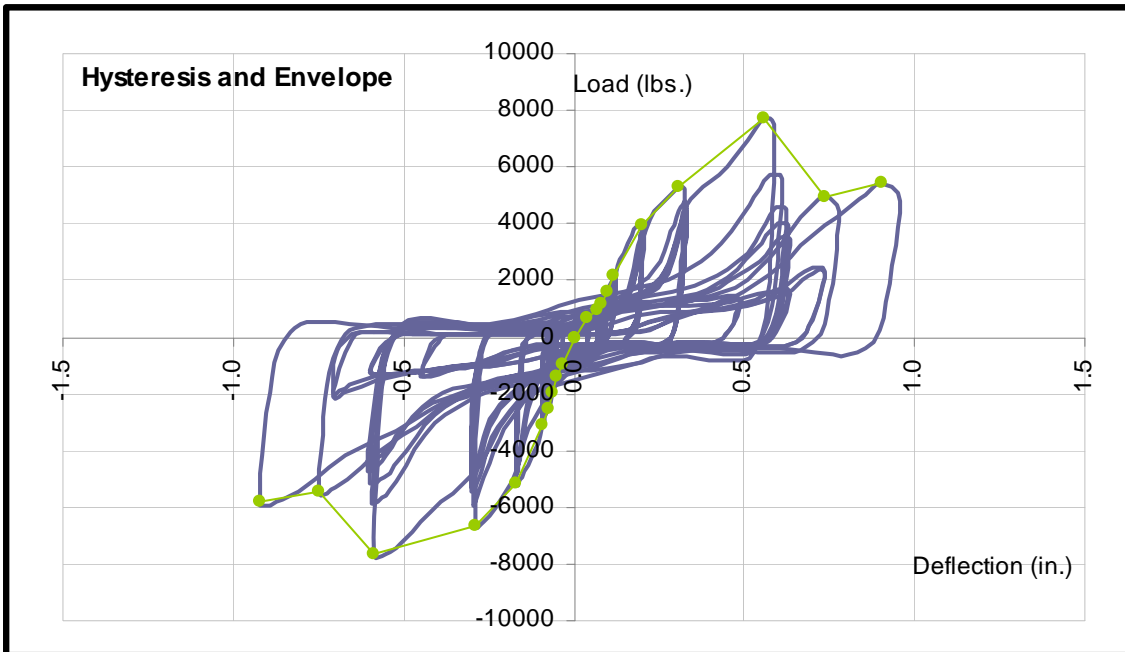
Test ID: 4-B-1-f 312, 3x6 Monotonic



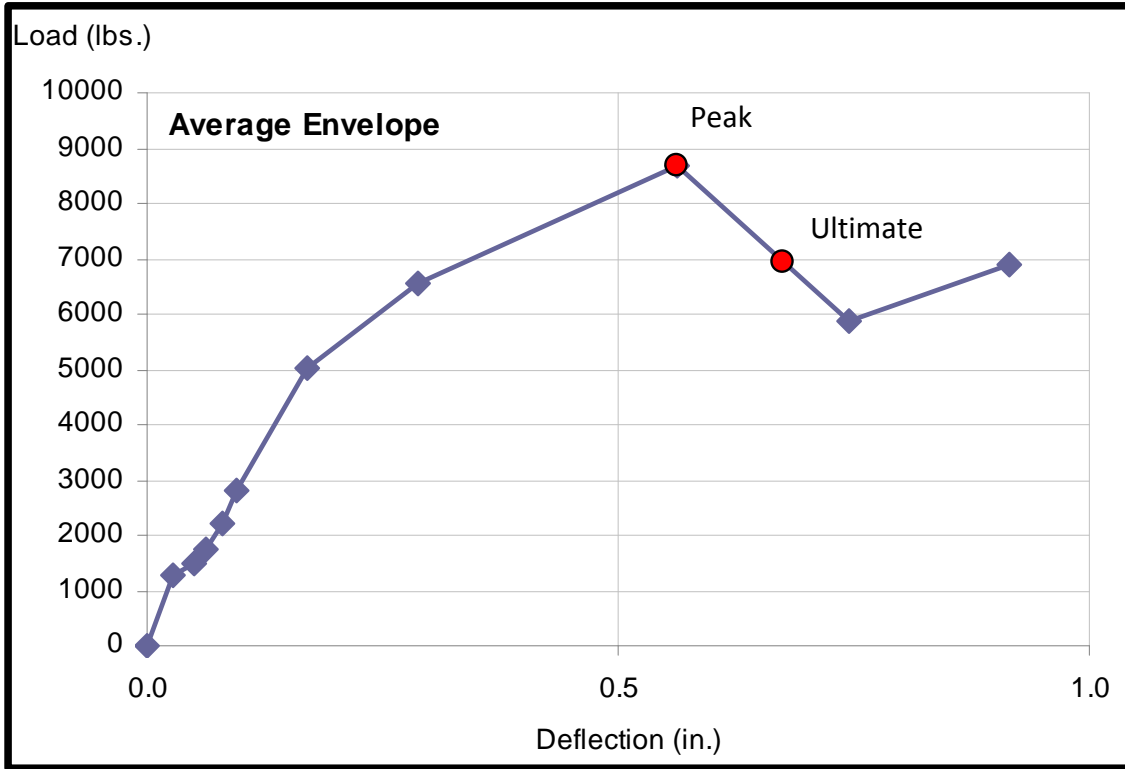
Test ID: 4-B-2-f 313, 3x6 Monotonic



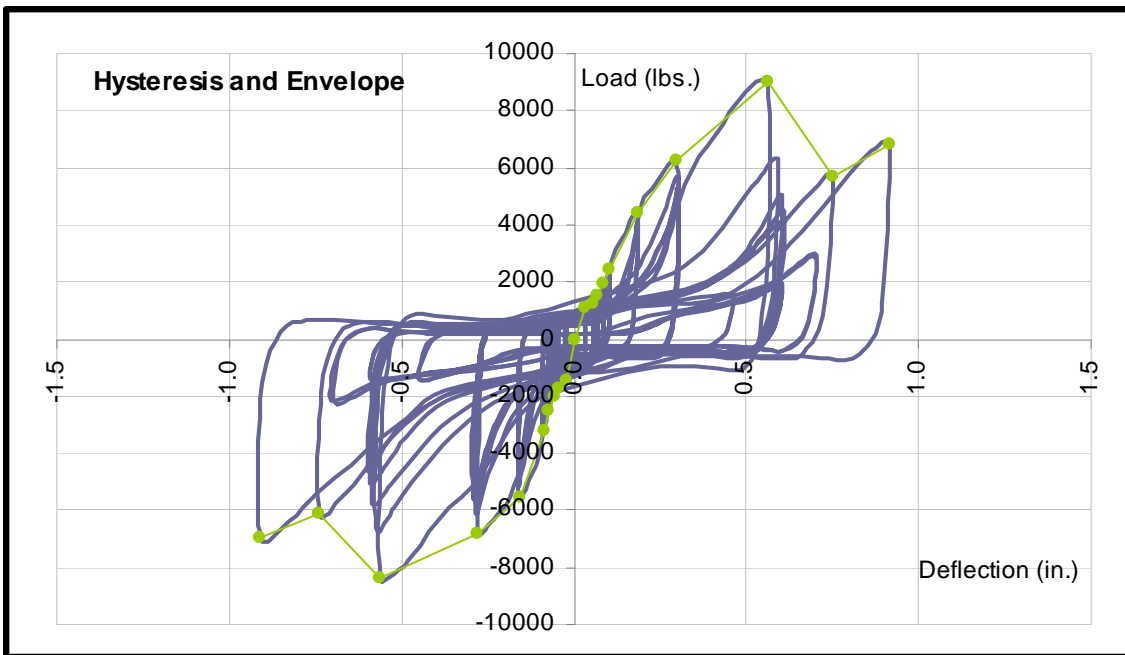
Test ID: 4-C-1-f 314, 2x6 Cyclic



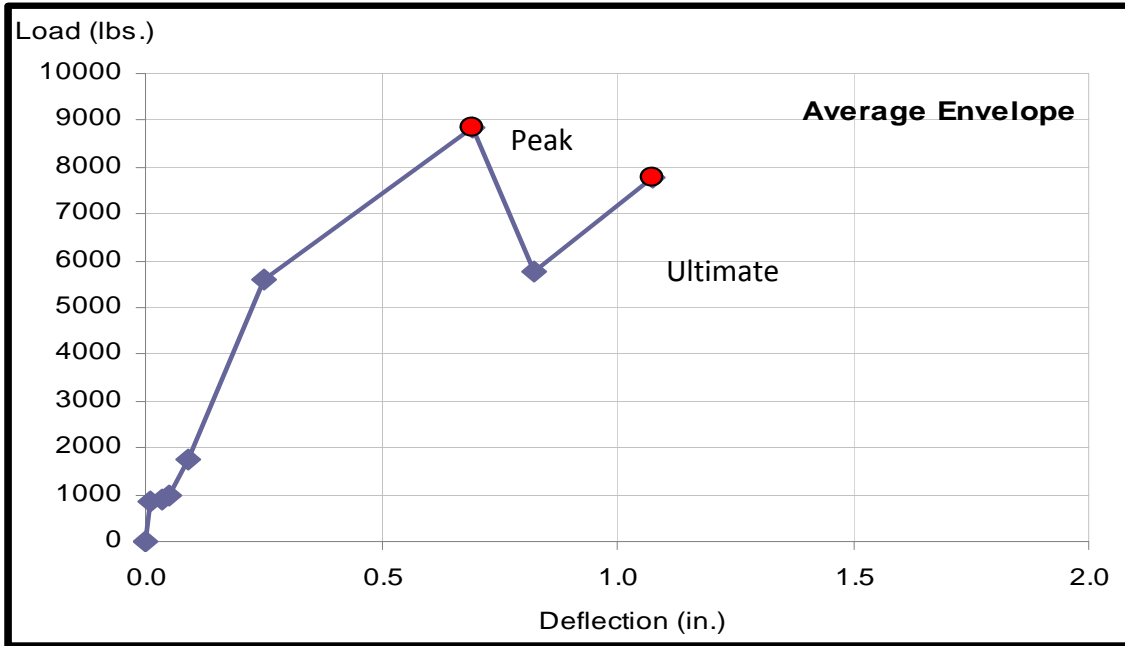
Test ID: 4-C-1-f 314, 2x6 Cyclic



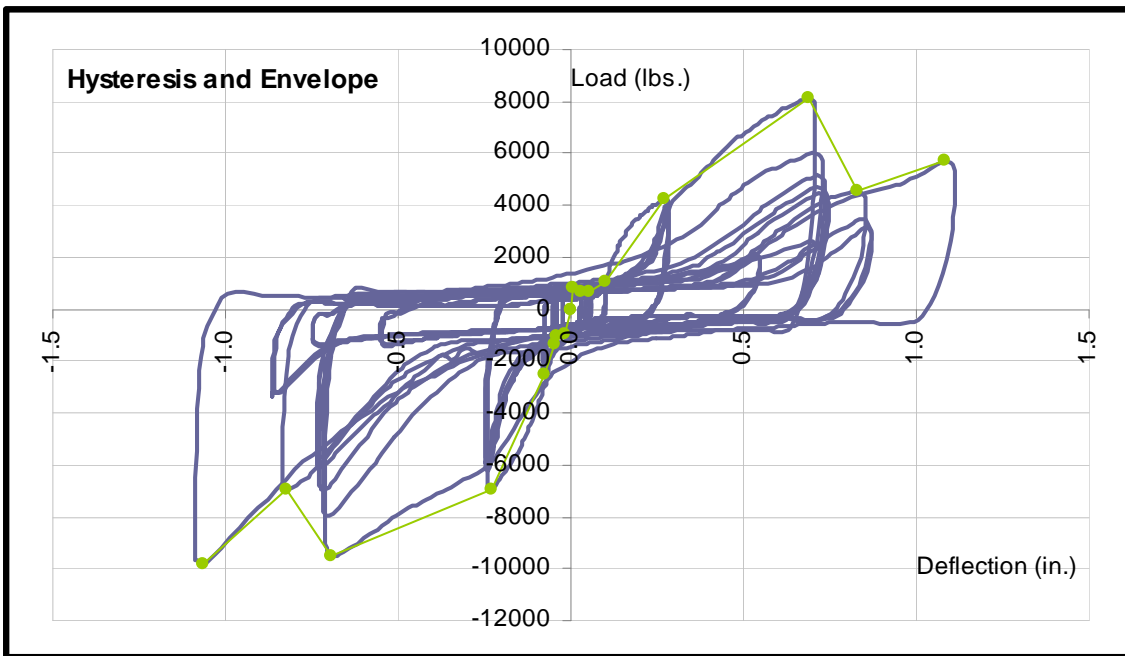
Test ID: 4-C-1-f 315, 2x6 Cyclic



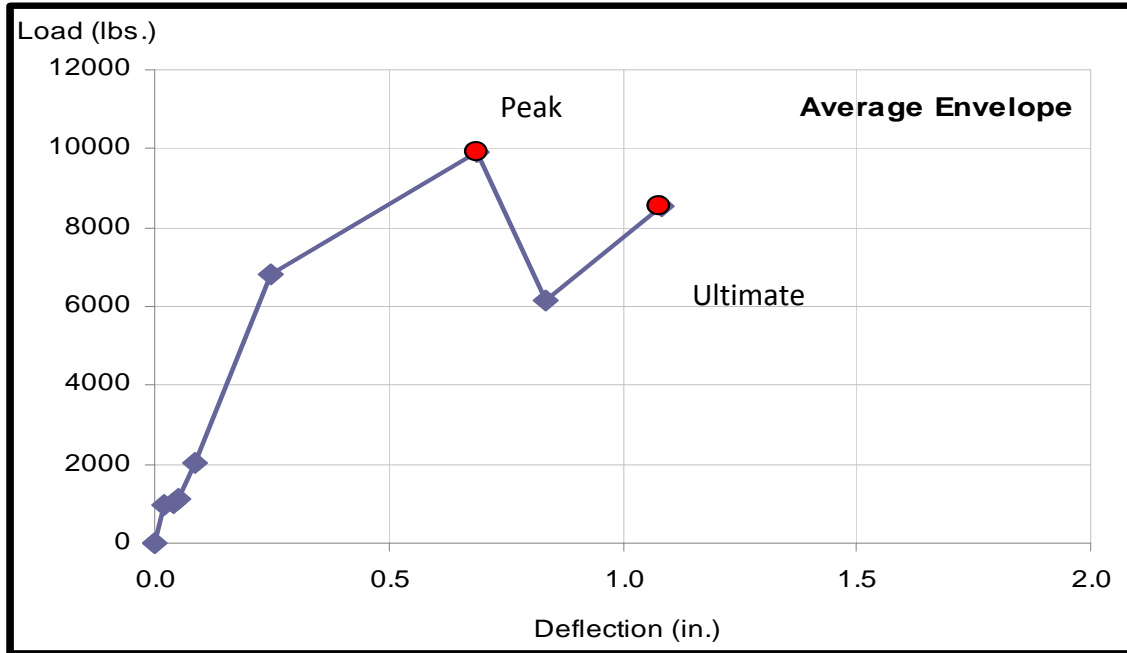
Test ID: 4-C-1-f 315, 2x6 Cyclic



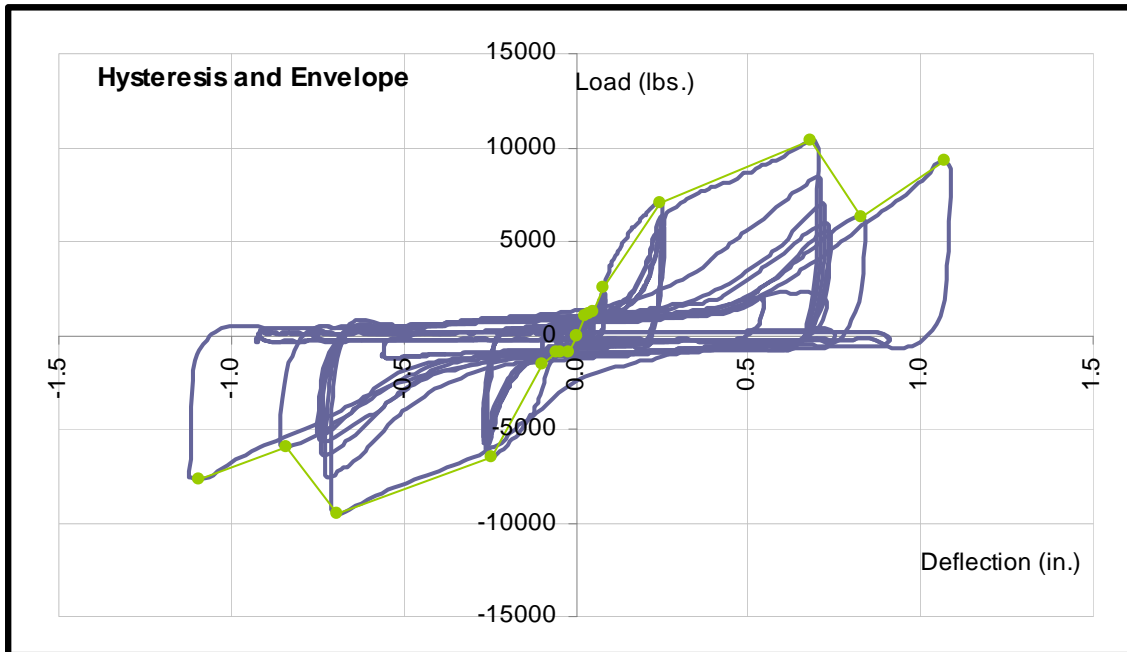
Test ID: 4-D-1-f 316, 3x6 Cyclic



Test ID: 4-D-1-f 316, 3x6 Cyclic



Test ID: 4-D-2-f 317, 3x6 Cyclic



Test ID: 4-D-2-f 317, 3x6 Cyclic

SEAOC Blue Book – Seismic Design Recommendations Anchor Bolts in Light-Frame Construction at Small Edge Distances

ASCE 7-05 reference section(s)	2007 CBC reference section(s)	Other standard reference section(s)
Ch 12 Seismic Design Requirements	1613 Earthquake Loads 1911 Anchorage to Codes- Allowable Stress Design 1912 Anchorage to Concrete- Strength Design	ACI 318-05 Appendix D ACI 318-08 Appendix D ACI 355.2 NDS [®] -05 Table 11E

Introduction

The subject of this article is the anchorage of wood-frame sill plates on structural walls in light-frame construction. Wood-frame shear walls have traditionally been connected to concrete foundations with cast-in anchor bolts or post-installed anchors in accordance with varied local practices. At shear wall locations, substantial loads are assumed to load the anchor bolts in pure shear parallel to the concrete edge. For the purposes of this article the following are assumed:

- Typical cast-in place “L-bolt,” minimum 7-inch embedment
- Bolt diameter of nominal ½ inch through ¾ inch
- Standard or 3-inch square plate washer with standard nut
- Bolts assumed to act in pure shear, loaded parallel to free edge of concrete
- Bolt corner distance minimum 8 inches
- Preservative-treated wood sill plate (2x4, 2x6, 3x4, 3x6, etc.)
- Typical wood-frame construction with redundant anchor bolts
- Foundation minimum $f'c=2500$ psi, conventional or pre-stressed concrete.

The change of model codes in California in January 2007 from the 1997 UBC to the 2006 IBC required a number of fundamental changes to the accepted design practices of wood-frame sill plate anchorage in light-frame structures. A significant change to design practice was also necessary to apply the IBC provisions for the seismic design of anchor bolt connections occurring near a concrete edge. These changes have been a source of much discussion and frustration for code users in high seismic areas subject to the IBC and ACI codes.

Two sensitive assumptions that affect the ACI Appendix D calculation are the ductility parameter and the cracked concrete parameter. The ductility parameter of IBC 1908.1.16 [D3.3.5] alone requires a 60 percent reduction to the connection capacity in concrete if the attachment to concrete is not ductile at the concrete design strength. (ACI 318-08 has reduced the reduction to 50 percent in light-frame construction.) The resultant low concrete capacity values indicate that a failure of the connection is expected to occur in the concrete long before it occurs in the anchor bolt or the wood sill plate, which is counter-intuitive. The SEAOC Seismology Committee performed a literature search of anchor bolt testing for wood sill plates with small concrete edges distances and discovered very limited research was available. The SEAOC Seismology Committee then decided to embark on an anchor bolt testing program. Using the Tyrell Gilb facility in Stockton, California, members of the SEAOC Seismology Light-frame Subcommittee conducted the first test program of its kind where the behavior of light-frame wood sill plate anchorage at small edge distances was targeted. Additionally, the test program included non-destructive impact-echo readings to continuously monitor the progression of any delaminations in the concrete. The results of this testing program are published in the document “Report on Laboratory Testing of Anchor Bolts Connecting Wood Sill Plates to Concrete with Minimum Edge Distances,” dated March 29, 2009. This report is available for download from the SEAONC website: www.SEAONC.org/member.

The SEAOC test data show that the yield strength of the wood sill plate connection governs over the strength of the concrete in the subject connections. This component testing was necessary to determine the specifics of the connection behavior, particularly the large amount of yielding the bolts achieve above the concrete surface and the beneficial clamping effect due to the square plate washer.

In this article we present additional commentary to the test report findings and review the underlying assumptions that may be appropriately considered by the designer. The recommendation is presented that the subject anchors may be conservatively designed assuming a wood yield mode as predicted by the yield limit equations associated with Mode III_s and Mode IV behavior in the *ANSI/AF&PA NDS-2005 National Design Specification® (NDS) for Wood Construction*. These values are subject to the same limitations as NDS Table 11E and are included at the end of this article for reference. These values do not apply to anchorage in light-weight concrete, post-installed anchors, or anchorage of cold-formed steel track. Finally, recommendations for further testing are discussed.

Background

In California, the design procedure and code-prescribed capacity of the subject bolts had not changed since the values were first tabulated and introduced in the 1979 UBC. In the IBC jurisdictions outside California, new ACI strength-based provisions for the design of seismically loaded cast-in anchors have been a part of the IBC since the 2000 edition. Regarding the provisions of 2006 IBC, which are currently applicable in many states, anchor bolt design is covered in IBC sections 1911 (Allowable Stress Design) and 1912 (Strength Design). IBC 1911 requires that with any seismic loading, anchor bolt capacities must use a strength-based design procedure. Per IBC 1912, the subject L-bolt is specifically required to be designed to the requirements of ACI 318 Appendix D provided its application “is within the scope of the appendix.” The strength design of anchors that are not within the scope of Appendix D shall be designed by an “approved procedure.” Therefore the subject anchor bolts are required to use strength-based design for seismic loads, but for wind loads the anchor bolt capacities may be taken from IBC Table 1911.2, which still contain the historical values used prior to the IBC.

The scope and provisions of ACI 318 Appendix D resulted from many years of testing and substantial effort directed at providing designers more transparency into the limit states associated with various classes of concrete anchorage. Wood sill plate anchorage forms a small subset of possible anchorage conditions covered by Appendix D. This connection is of greater regional importance than international importance, and there was a gap in the literature addressing this condition prior to the SEAOC testing. As a result, the present code provisions did not fully anticipate this narrow but important condition, and the generalized provisions produced design results inconsistent with the needs of light-frame design.

The problems light-frame designers have faced with the ACI Appendix D provisions are rooted in the very low capacity values that seemed to be required relative to past practice. As described herein, proper application of the ductility and cracked concrete parameters provide a rational, usable set of bolt values. Such a rational anchor bolt value should embody the following characteristics:

- 1) The capacity is internally consistent with other material chapters (e.g. shear capacity due to embedment in concrete should be proportionately stronger than masonry or wood).
- 2) The seismic capacity versus wind capacity is internally consistent with that required for other code-approved components and assemblies.
- 3) The design capacity is not overly sensitive to any particular assumption. (For assumptions that are highly sensitive by nature, it is appropriate to use a continuous function or finely divided steps).

Light-frame designers have derived bolt values through Appendix D on the order of one-quarter to as little as one-fifth of the traditional value when assuming a non-ductile connection and cracked concrete. Such a result is very low and leads to a design solution that would be inappropriate for the wood sill attachment of many code-listed shear wall systems. For example, some designers have derived a capacity of approximately 300 pounds (ASD) for an anchor that traditionally carried approximately 1200 pounds (ASD). Accordingly, a fairly heavily loaded shear wall that would have traditionally required two anchors per stud bay would now require eight anchors per stud bay, which do not physically fit.

A final complication has been the inconsistency of design capacities determined by different designers. The traditional practice of using table values for anchor capacities was replaced by a design procedure with over a dozen variables. Amid the added complexity, practitioners have questioned the marginal benefit in implementing dramatic

SEAOC Blue Book – Seismic Design Recommendations Anchor Bolts in Light-Frame Construction at Small Edge Distances

changes to the anchor bolt design methodology. Since issues with the old values were not apparent, the need for substantial change was puzzling.

Testing

The primary goals of the SEAOC Anchor Bolt Test program were to:

- 1) Determine whether the wood connection yielding controls the connection capacity when loaded parallel to an edge and if the equations found in each material standard are good predictors of behavior.
- 2) Determine whether the connection exhibits ductile behavior.
- 3) Propose rational design capacities for the connection.

It was decided to test the 5/8-inch diameter bolts since they are representative of most medium and heavy duty shear wall applications. While much residential concrete construction is specified at $f'_c=2500$ psi, in-service concrete is expected to experience some strength gain over time. For this reason, a range of 2500 to 3000 psi was specified for the test concrete compressive test. In actuality, the highest compressive test cylinder result was 2710 psi. As also detailed in the SEAOC “Report on Laboratory Testing of Anchor Bolts Connecting Wood Sill Plates to Concrete with Minimum Edge Distances,” the tests included two unique features. First, the effect of friction was isolated on half of the tests by providing a lubricated polyethylene membrane at the wood-concrete interface. This allowed the contribution of friction to be better understood from the test data. Second, impact-echo testing was conducted during the test to continuously monitor the status of delamination that developed in the concrete that may not have been visibly apparent. Aside from these unique features, every effort was made to test materials representative of the most common shear wall connections.

The independent variables tested were:

Item	Configuration Tested
Sill plate size	2x4, 3x4, 2x6 and 3x6
Anchor bolt edge distance	1.75 inches or 2.75 inches, dependent upon sill plate
Testing protocol	monotonic versus pseudo-cyclic
Wood-concrete interface condition	friction versus “frictionless” membrane

To properly generate test data for the purpose of assessing behavior, a new displacement based loading protocol was developed. Using data from an initial set of monotonic pull tests, cyclic tests were calibrated so that damage produced by the test would best represent actual in-service failure modes. For the new protocol, the SEAOC Seismology Committee used a hybrid approach essentially taking the CUREE protocol with additional cycles added at low load levels. Independently, the SEAOSC sequential phased displacement (SPD) loading was used on several tests to compare results.

Findings

The first result to note was that the monotonic tests were an accurate predictor of the elastic performance characteristics exhibited in the cyclic tests. Once the anchors were loaded to approximately 5000 pounds, the anchors slowly started to exhibit some plastic behavior as further displacement occurred. The frictionless membrane applied under the length of sill plate had a minor effect at small displacements within the elastic range. For loads in the range of design values, which were well within the elastic range, there was little difference between the pseudo-cyclic, monotonic, and sequential phased displacement test results.

Second, the test showed that fastener fatigue was not a limit state influenced by any of the various loading protocols. This is an important observation since it limits the area of concern to the strength of wood and concrete elements tested.

Third, the class of anchorage tested was ductile, and concrete side-breakout was not detected until the resistance force was significantly beyond the elastic range, specifically not until the peak value was achieved. In addition to the observation of significant bolt bending, peak strengths from cyclic tests of the 1¾-inch edge distance case (e.g. 2x4 and 3x4 sill plate) ranged from 2.3 to 2.9 times the NDS calculated yield values for the wood sill plate connection, which indicates substantial loading beyond the yield limit state of the connection. The peak value was generally accompanied by a complete, but shallow concrete delamination. Use of the impact-echo measurements often signaled internal concrete delamination prior to any visual evidence, although no evidence of any sort was noted in the elastic range or below 6000 pounds in any test. After the initial shallow delamination occurred, the anchors were in tension, and a secondary peak was recorded--often with a higher ultimate value than the initial peak (see Figure 1). Significant ductile mechanisms were observed in the form of large deflections of the sill plate and bending of the anchor bolt. The failure mechanics of concrete predict that delaminations form initially from a series of micro-cracks. These internal micro-cracks propagate and interconnect along an eventual failure surface that corresponds roughly to the path of least energy. Although the study of fracture mechanics has not progressed to the point to have accurately predicted the information obtained from the SEAOB tests, it does predict significant energy can be absorbed by the concrete after the onset of inelastic behavior. The tests showed that in the post-elastic range, strength gain is slowed as micro-cracks grow to the point where the peak strength value occurs. The peak strength was noted to coincide with the point where initial delamination occurred.

Fourth, the ACI Appendix D concrete break-out strength taken from the estimated mean appears overly conservative for the 1¾-inch edge distance case (e.g. 2x4 and 3x4 wood sill plates). From cyclic test results, the tested peak strengths ranged from 1.7 to 2.2 times the ACI Appendix D calculated values adjusted to represent mean-based concrete break-out strengths. Taken on the whole (i.e. with and without the friction-reducing membrane) the 2x4 and 3x4 cyclic tests averaged 1.9 times the ACI concrete break out calculated value adjusted for the mean strength. Similarly the 2x6 and 3x6 cyclic tests achieved 1.4 times the ACI equation. If the equations were to accurately reflect the test results, the comparison would be expected to be on the order of 1:1.

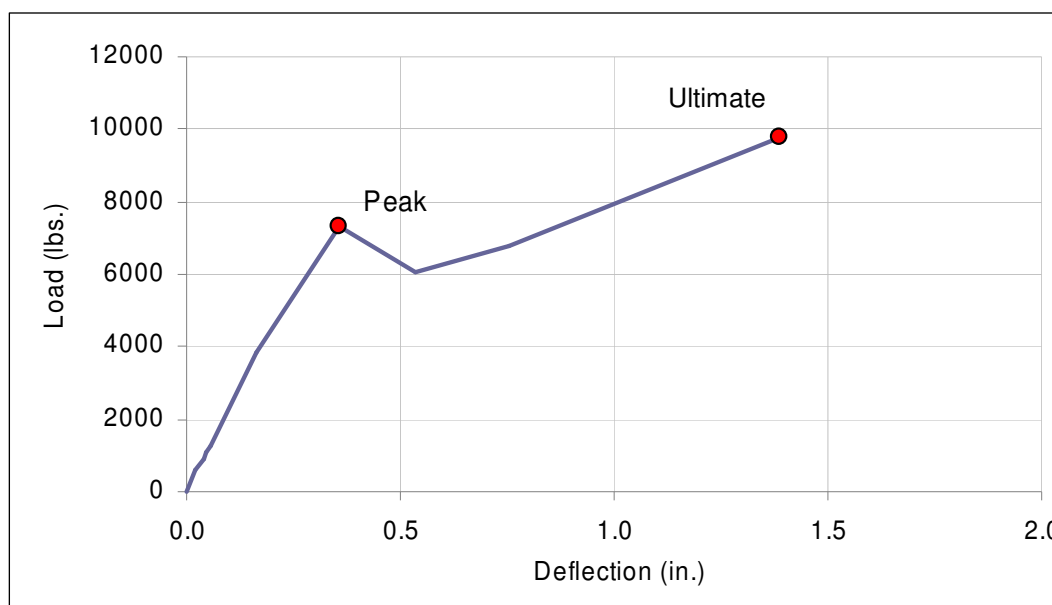


Figure 1: Typical inelastic behavior showing secondary peak.

Since the ACI 318 Appendix D break-out equation approximates the 5 percent fractile strength, the SEAOB test report adjusts the Appendix D break-out strength to the mean-based estimate in order to provide an appropriate

comparison to the mean of the test data. There is some degree of assumption regarding the variance of the data, and details are given for the specifics of each concrete test specimen in the SEAOC Anchor Bolt Test Report. However, whatever adjustment is made, the aggregate of testing has shown the connection to exhibit good capacity and ductility that was previously unaccounted for.

Finally, since the ultimate values corresponded to large drifts, the data reduction used in the test report was conservatively modified from the ASTM E2126 standard. In particular, the first peak was used rather than the ultimate load specified by the standard. This peak value was defined by the SEAOC Seismology Committee as the highest load prior to any drop of 5 percent in capacity.

Assumptions Applicable to Anchor Bolt Design

Scope. As indicated above, ACI appendix D is utilized for cast-in L-bolts “provided they are within the scope of Appendix D.” The ACI scoping provisions of D2.1 and D2.2 indicate the Appendix applies to “cast-in anchors” and “connected structural elements” of which the subject anchor bolts are clearly included. However, the ACI-05 commentary states that the scope envisions anchorages where a single anchor failure could result in a loss of stability of the structure. Generally speaking, sill plate anchorage is not a low redundancy application. There are typically at least four connections present in the sill plate (two hold downs and two anchor bolts), there are often other interior walls present, and there is also the likelihood of substantial friction at the sill plate connections. Thus multiple load paths exist. Therefore, some engineers have suggested that the subject anchor bolts may not fall within the scope of Appendix D based upon the commentary. While this point may have certain merits, the IBC provides that if anchors are not to be regulated by Appendix D, another “approved method” is necessary. Such an approved method should incorporate a similar level of sophistication as Appendix D. The IBC Table 1911.2 does not incorporate the various failure mechanisms that are addressed by Appendix D.

Supplementary Reinforcement. ACI 318-05 section D.4.4 provides for the use a strength reduction factor of $\Phi=0.75$ (rather than $\Phi=0.70$), if “reinforcement is proportioned to tie a potential concrete failure prism to the structural member.” ACI 318-08 section D.4.4 and related commentary further clarifies that supplementary reinforcement need only be present, and explicit design is not required in order to utilize the higher factor. Most light-frame foundations have a continuous #4 or #5 reinforcement bar (or a post-tension tendon) near the top and along the edge of the slab or curb, and it has been suggested that this bar may allow for an assumption of the higher factor. The Committee cautions designers who may be tempted to categorize this bar as supplementary reinforcement since in our experience the bar location is not sufficiently controlled in the field in a manner that would allow for relatively shallow embedments.

Cracked Concrete Assumption. The first UBC code reference regarding cracked concrete appeared in 1997 UBC section 1923.2, which referred to anchorage embedment in “tension zones.” At the time, overhead anchorage of structural members and equipment were a primary concern, and these regulations applied to anchorage occurring below the neutral axis on bending members such as beams or elevated concrete decks. IBC has also incorporated a cracked-concrete anchor reduction since the 2000 IBC [1319.5.2.7]. In the current code, ACI 318-05 section D6.2.7 stipulates “where analysis indicates cracking at service load levels,” $\Psi_{c,v}$ shall be taken as 1.0 for anchors “with no supplementary reinforcement or edge reinforcement smaller than a No. 4 bar.” (For testing, a crack width of up to 0.12 inches is produced.) Thus, in strength design, when the uncracked concrete is justified, cast-in anchors are allowed a 40 percent capacity increase, since $\Psi_{c,v}$ can be taken as 1.4.

The uncracked assumption is generally justified in light-frame construction as can be seen from the review of original testing in cracks. A good review of available test information was recently published by Eligehausen, Mollé, and Silva in the publication *Anchorage in Concrete Construction* (2006). In this publication the authors explain that cracked concrete is a concern with anchors in tension since diminished values have been obtained with testing and over time the fastening can loosen. However for shear loading they report that where “a shear load acts perpendicular to the crack, then the load-displacement behavior does not differ significantly from the behavior in non-cracked concrete. . . . [E]ven anchors that exhibit inferior performance when loaded in tension in cracks are usually adequate to resist shear loads in cracked concrete” (p. 157). It should be expected that the subject sill plate

anchors will not be compromised by any significant degree, since it would require cracks intersecting the anchor and running parallel to the concrete edge, which are highly unlikely in typical light-frame applications. Any cracks occurring in the concrete substrate would be expected to be more or less perpendicular to the concrete edge and thus perpendicular to the applied load and not affecting groups of anchors.

The code requires the determination of cracked versus uncracked to be made at service level loads and that the crack reduction applies to a full-depth crack along the axis of the anchor. In the practical sense, it is possible that in combination with the effects of restraint, expansive soils, or frost heave, limited areas of a conventional foundation, deck, or post-tension slab-on-grade could experience curvatures in excess of the cracking modulus as redistribution occurs. However, given the inherent redundancy of anchors in light-frame construction coupled with the low probability of coincidence between qualifying cracks and typical anchor placement, it is not reasonable to assume a cracked substrate unless specific conditions clearly indicate otherwise.

Conclusion and Recommendations

Based upon the SEAOC Report on Laboratory Testing of Anchor Bolts Connecting Wood Sill Plates to Concrete with Minimum Edge Distances, the connection will yield at the wood sill plate prior to the formation of a concrete limit state when loaded parallel to a concrete edge. In other words, the concrete exceeds the strength of the wood. In the non-linear range of performance, an initial and secondary peak load was recorded that indicated the connection showed excellent ductility.

The test data and examination of assumptions detailed above indicate that it is rational to use the values obtained from ACI Appendix D assuming uncracked concrete and a ductile attachment. Also based upon the test results that indicate concrete will not govern for the anchorage of the subject 2x and 3x sill plates, it is conservative to use the NDS design values for bolts up to 3/4 inch in diameter that meet the requirements shown at the beginning of this article. While 3/4-inch diameter bolts were not specifically tested, they may be used with 6-inch nominal width sill plates due to increased cover. Additionally, the NDS predicts the same type Mode III_s failure for the 3/4-inch anchors. Table 1 shows representative anchor bolt shear values based upon the NDS-05.

Table 1. Anchor Bolt Shear Values Based on the NDS 05 ($C_D=1.6$)

Sill Plate	Bolt Diameter ^{1,2}		
	1/2"	5/8"	3/4"
2x	1040	1488	2032
3x	1232	1888	2426

¹ 3/4" anchor bolt limited to 6-inch nominal width sill plates

² Values are shown in lbs. (ASD basis)

Another benefit of the testing was isolating the effect of friction under the sill plate. The testing data indicates that a portion of the shear load can be transferred through friction between the bottom of the sill plate and the concrete. The amount of load that is transferred by friction is significant for monotonic testing and less so for cyclical testing. This supports the notion that friction is significantly increased due to bending of the anchors and the clamping action of the plate washers. In a wall assembly, the studs and boundary elements in compression may play a more significant role than previously assumed and present the opportunity for further study.

Finally, the reader is cautioned that any damage occurring to this connection may not be readily apparent. Therefore, post-event observers should review the photos contained in the test result and be aware that severe damage can be masked by the top of the sill plate.

Through much effort coordinated by the Light-Frame System’s subcommittee, new testing specifications and loading protocols were developed to ensure that the data would be properly generated and assessed. In addition to the efforts of the 2008-2009 SEAOC Seismology Committee, a number of firms donated time, materials and/or

SEAOC Blue Book – Seismic Design Recommendations Anchor Bolts in Light-Frame Construction at Small Edge Distances

effort, including Scientific Construction Laboratories, Inc., Structural Solutions, Inc., Certus Consulting, Inc., and VanDorpe Chou Associates, Inc. In addition, Phil Line of the American Forest & Paper Association provided valuable effort and input. The Committee was also very fortunate to be able to conduct the tests at the Tyrell Gilb Research Laboratory owned by Simpson Manufacturing Company, in Stockton, California. This facility is accredited to comply with ANS/ISO/IEC Standard 17025:2005.

References

- ACI 318-05 Appendix D. *Building Code Requirements for Structural Concrete and Commentary*. American Concrete Institute, 2005, pp. 379-402, Farmington Hills, MI.
- AF&PA (2001). *National Design Specification for Wood Construction*. American Forest & Paper Association, Washington, DC.
- ASTM Standard D 5764 - 97a, 2007: *Standard Test Method for Evaluating Dowel-Bearing Strength of Wood and Wood-Based products*. ASTM International, West Conshohocken, PA.
- ASTM Standard E 2126, 2008 - Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings. ASTM International, West Conshohocken, PA.
- BSSC (2004). *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1: Provisions* (FEMA 450-part 1/2003 Edition), Building Seismic Safety Council, Washington, DC.
- BSSC (2004). *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2: Commentary* (FEMA 450-part 2/2003 Edition), Building Seismic Safety Council, Washington, DC.
- CBC (2007). *California Building Code*. California Building Standards Code, 2007, Country Club Hills, IL.
- Cook, Ronald A. *Strength Design of Anchorage to Concrete*. Portland Cement Association, 1999, Skokie, IL.
- Cook, Ronald A. PhD, P.E., Wollmershauser, P.E. and Gerber, Brian S.E. *Post-Installed Anchor Approvals for ACI318 Appendix D*. Structure Magazine, January 2006, pg 58.
- CUREE Publication W-02: *Development of a Testing Protocol for Wood-Frame Structures*, Richmond, CA.
- Dolan, J.D.; Gutshall S.T.; and McLain T.E (1996).. VPI Research Report No. TE-1994-003 - Determination of Short-Term Duration-of-Load Performance of Nailed and Bolted Connections Using Sequential Phased Displacement Tests, Virginia Polytechnic Institute and State University, Blacksburg, VA.
- Eligehausen, Rolf, Mallee, Ranier and Silva, John F. *Anchorage in Concrete Construction*. Ernst & Sohn, 2006, Berlin, Germany.
- Fogstad, Christian, P.E. and Gerber, Brian, P.E., S.E. “Code Changes Affecting Post-Installed Concrete Anchor Design.” *Structure Magazine*, December 2007, Pages 45-48.
- FEMA 461: *Interim Protocols for Determining Deismic Performance Characteristics of Structural and Nonstructural Components Through Laboratory Testing*. May 2007, Washington, D.C.
- Ghosh, S.K., Dowty, Susan, Dasgupta, P. *Analysis of Revisions to the 2006 IBC Structural Provisions, pgs 97-106, 111-112*. Structures and Codes Institute, 2006, Palatine, IL.
- Hess, Richard. For What Planet is This Code Written? *Structure Magazine*, November 2008, page 73.
- IBC (2003). *2003 International Building Code*. International Code Council, 2003, Country Club Hills, IL.
- IBC (2006). *2006 International Building Code*. International Code Council, 2006, Country Club Hills, IL.
- ICBO (1997). *1997 Uniform Building Code*. International Conference of Building Officials, Whittier, CA.
- ICC, IBC Final Draft Public Hearing “S-229/1912.” Proponent: James R. Cagley, Cagley & Associates and Joseph J. Messersmith, Portland Cement Association; representing the American Concrete Institute Committee 318 and Portland Cement Association respectively, March 1999, pp. S246-249, Falls Church, VA.
- ICC-ES Proposed Acceptance Criteria for Cast-in-Place Proprietary Bolts in Concrete for Light-frame Construction, Subject AC399 0508-R1, Proposed April 2008.
- Mendes, Stanley B. “Lessons Learned From Four Earthquake Damaged Multi-Story Type V Structures,” *Proc. 64th Annual SEAOC Convention*, Indian Wells, CA, October 19-21 1995, Sacramento, CA.
- Mohammad, M.; Karacabeyli, E. and Quenneville, J.H.P., “Lateral Resistance of Bolted Wood-to-Concrete Connections Loaded Parallel or Perpendicular to Grain,” *Canadian Journal of Civil Engineering* (2003), Vancouver, B.C.
- SEAOC (2002). *Seismic Design Manual, Volume II: Building Design Examples - Light Frame, Masonry and Tilt-up (1997 UBC)*, Structural Engineers Association of California, Sacramento, CA.

SEAOC Blue Book – Seismic Design Recommendations Anchor Bolts in Light-Frame Construction at Small Edge Distances

SEAOC Seismology Committee (1999). *Recommended Lateral Force Requirements and Commentary*, Seventh Edition, Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee: “Report on Laboratory Testing of Anchor Bolts Connecting Wood Sill Plates to Concrete with Minimum Edge Distances,” February 2009, Sacramento, CA.

SEAOC, Seismology and Structural Standards Committee, “Wood-Framed Design in Seismic Areas: Re-tooling for the 2006 IBC in Light of Current Design Practice,” October 2007.

Soltis, L.A. and Wilkinson, T.L. (1991). “United States Adaptation of European Yield Model to Large Diameter Dowel Fasteners Specifications,” International Timber Engineering Conference, London.

Keywords

light-frame

wood

anchor bolt

edge distance

How To Cite This Publication

In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2009)

In the writer’s reference list, the reference should be listed as:

SEAOC Seismology Committee (2009). “Anchor Bolts in Light-Frame Construction at Small Edge Distances,” June, 2009, *The SEAOC Blue Book: Seismic design recommendations*, Structural Engineers Association of California, Sacramento, CA. Accessible via the world wide web at: <http://www.seaoc.org/bluebook/index.html>