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, 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
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:

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

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 SEAOC 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.

![Graph](https://via.placeholder.com/150)

**Figure 1:** Typical inelastic behavior showing secondary peak.

Since the ACI 318 Appendix D break-out equation approximates the 5 percent fractile strength, the SEAOC 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, Mallé, 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 ¼ inch in diameter that meet the requirements shown at the beginning of this article. While ¾-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, failure for the ¾-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)**

<table>
<thead>
<tr>
<th>Sill Plate</th>
<th>Bolt Diameter¹,²</th>
<th>½”</th>
<th>5/8”</th>
<th>¾”</th>
</tr>
</thead>
<tbody>
<tr>
<td>2x</td>
<td></td>
<td>1040</td>
<td>1488</td>
<td>2032</td>
</tr>
<tr>
<td>3x</td>
<td></td>
<td>1232</td>
<td>1888</td>
<td>2426</td>
</tr>
</tbody>
</table>

¹ ¾” 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
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.

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