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.

Philip Line, PE
American Wood Council / AF&PA

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

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

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

Thomas A. Voss, CE
Scientific Construction Laboratories, Inc.
What we did
Anchor Bolt Testing in < 25 minutes
2x Stud Wall
Concrete Slab Per Plan
2x P.T. Sill W/A.B. Per Plan
E.N.
Finish Grade
Pad Grade
Enlarged FTG As Occurs
Cont. No. 4 Bar Top and Bottom
Sub-Base Per Soils Report
15" 2-Story
18" 3-Story
See Plan
Reinf Per Plan
24" 5-Story
3" CLR.
For What Planet Is This Code Written?

By Richard L. Hess, A.E., S.E., SECB, F. ASCE, CSI, CCCA

Members of the SEAOC Code Committee (as well as others) are busy calculating. They are trying to come up with an allowable shear value for a wood sill plate anchor bolt that will facilitate construction of a single-story wood-frame house with anchor bolts greater than one foot apart or multi-story wood-frame buildings under any conditions. They are trying to figure out what approach will produce the best result from ACI 318 Appendix D, which is required by the 2006 International Building Code. Others, sensing the profit potential in this new complication, are developing software that solves the problem for you.

And then there is the question of ductility: How to make the steel ing 5/8-inch bolts with 4-inch embedment being good for 1,000 pounds in shear with no edge distance restrictions.

The 6d (d = bolt diameter) minimum edge distance was not added until 1970. In 1979, after the 1971 San Fernando earthquake, the values for 5/8-inch anchor bolts were increased from 1,750 to 3,000 pounds. Subsequently, those values required a 12d edge distance with a 50% reduction allowed, which was maintained in the 1997 UBC three years after the Northridge earthquake. Section 10.5.6 of the code required 5/8-inch anchor bolts to be used in seismic zone IV and this was due, in part, because of the recommendations of light frame to be the failure of the wood plate or floor, and not the floor joist. Los Angeles separated the non-reinforced masonry as if all the walls and columns were not to be taken up by the recommendations of light frame to be the failure of the wood plate or floor, and not the floor joist. Los Angeles separated the non-reinforced masonry as if all the walls and columns were not to be taken up by the recommendations of light frame to be the failure of the wood plate or floor, and not the floor joist.
References

- Full 50-page testing report available at www.SEAONC.org
- SEAOC Blue Book article
Preliminary Phase of Anchor Bolt Testing

Testing Specifications and Loading Protocols

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Project Team:

Scientific Construction Laboratories, Inc. (SCL)
Andy Fennell, CE, SECB, GC
3397 Mt. Diablo Blvd., Suite E
Lafayette, California 94549
(T) 925.284.3363 (F) 925.284.3360
andy.sclabs@earthlink.net

CERTUS Consulting, Inc. (CCI)
Kevin Moore, CE, SE, SECB
405 Fourteenth Street, Suite 160
Oakland, California 94612
(T) 510.835.0705 (F) 510.835.0775
moore@certuscorp.net

Scientific Construction Laboratories, Inc. (SCL)
Tom Voss, CE
3397 Mt. Diablo Blvd., Suite E
Lafayette, California 94549
(T) 925.284.3363 (F) 925.284.3360
voss.sclabs@earthlink.net

Structural Solutions, Inc. (SSI)
Gary Mochizuki, CE, SE
150 N. Wiget, Suite 102
Walnut Creek, CA 94598
(T) 925.938.3303 (F) 925.938.3522
gary@structsol.com
3"x4" P.T. SILL PLATE

NUT AND PLATE WASHER

2"x4" P.T. SILL PLATE

ISOLATION MEMBRANE (WHEN SPECIFIED)

7"

1\(\frac{3}{4}\)" CENTER LINE OF BOLT TO FACE OF CONCRETE
1. Single anchor tested (5/8”ϕ) in 7’ long sill plate
2. Direction of load (monotonic or pseudo-cyclic)
3. 2500-3000 psi concrete
4. Displacement gauge (string-pot)
5. Loading “grip” from hydraulic ram to wood sill
6. Previously tested anchor

Image 2 – Typical set-up for anchor tests at Tyrell Gilb Research Laboratory in Stockton, CA.
What we found
Chart 7 – Comparative plot of monotonic tests with (291, 292) & without (289, 290) membrane.

2 monotonic tests:
No membrane

2 monotonic tests:
Membrane present to minimize friction.

Range of ASD design capacities.
Mode I
(wood bearing deformation in side member)

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

Mode IV
Mode IV (wood bearing deformation in side member and dowel bending in wood member and concrete)
Test 296 stopped at approx ± 0.6”. No concrete side-break at this time.

A new pieces of 2x4 sill plate was installed and the same anchor was retested as Test 302.
Test 296 stopped at ± 0.6". No delamination.
<table>
<thead>
<tr>
<th>Code(s)</th>
<th>5/8” dia. bolt</th>
<th>ASD Design Capacity (#) 2x4 DF-L (seasoned) (including $C_D$)</th>
<th>ASD Design Capacity (#) 3x4 DF-L (seasoned) (including $C_D$)</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1991 UBC (NDS-86)</td>
<td></td>
<td>1306 #</td>
<td>1326 #</td>
<td>2510 (b), T-25-F</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$C_D = 1.33$</td>
<td>$C_D = 1.33$</td>
<td></td>
</tr>
<tr>
<td>1994 UBC (NDS-91)</td>
<td></td>
<td>1173 #</td>
<td>1,492 #</td>
<td>2336.2.3, T-23-III-J</td>
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<tr>
<td></td>
<td></td>
<td>$C_D = 1.33$</td>
<td>$C_D = 1.33$</td>
<td>NDS allows $C_D=1.6$.</td>
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<tr>
<td>1997 UBC (NDS-91)</td>
<td></td>
<td>1,408 #</td>
<td>1,790 #</td>
<td>2316, T-23-III-B-1</td>
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<tr>
<td></td>
<td></td>
<td>$C_D = 1.60$</td>
<td>$C_D = 1.60$</td>
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<tr>
<td>IBC-2003 (NDS-01)</td>
<td></td>
<td>1,424 #</td>
<td>1,824 #</td>
<td>NDS-01 Table 11E</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$C_D = 1.60$</td>
<td>$C_D = 1.60$</td>
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<tr>
<td>IBC-2006 (NDS-05)</td>
<td></td>
<td>1,488 #</td>
<td>1,888 #</td>
<td>NDS-05 Table 11E</td>
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<td></td>
<td></td>
<td>$C_D = 1.60$</td>
<td>$C_D = 1.60$</td>
<td></td>
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<tr>
<td>ACI-318-08 App. D, $f'_c= 2500$ psi.</td>
<td>Non ductile: 500 #</td>
<td>Non ductile: 500 #</td>
<td>0.5 factor used for non ductile behavior. Values assume “uncracked” concrete.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ductile: 1000 #</td>
<td>Ductile: 1000 #</td>
<td></td>
</tr>
<tr>
<td>ACI-318-05 App. D, $f'_c= 2500$ psi.</td>
<td>Non ductile: 400#</td>
<td>Non ductile: 400#</td>
<td>0.4 factor used for non ductile behavior. Values assume “uncracked” concrete.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ductile: 1000 #</td>
<td>Ductile: 1000 #</td>
<td></td>
</tr>
<tr>
<td>Peak values from average of Cyclic tests</td>
<td>Avg from test ID’s: 294, 295-NF, 296-NF: 6710 # @ 0.44”</td>
<td>Avg from test ID’s; 304, 305, 306-NF &amp; 307-NF: 7572 # @ 0.63”</td>
<td>Peak cyclic test values are at least 4 times historic NDS wood design values.</td>
<td></td>
</tr>
</tbody>
</table>
Test Findings

• When loaded parallel to the concrete edge; the **wood governs the connection**. The wood sill plate “yields” well before any concrete limit state.

• The connection showed **excellent ductility**. Beyond the initial “peak” used for calculating capacities, tests often continued on to a second (and higher) peak.
Test Findings, Cont’d

• For 2x4 and 3x4 plates, the average peak loads:
  – were more than 6 times higher than ductile connection design strengths obtained from the equations from ACI 318-05 (&d 08), App. D, and
  – were more than 4 times higher than the allowable capacities obtained from IBC 2006 (NDS-05).
Conclusions

• Anchor bolts connecting wood sill plates to concrete with minimum edge distances have capacities much, much higher than those that would be obtained using the ACI Appendix D.

• Therefore code changes should be passed to allow for higher design values for these anchor bolts.
The next step
ICC Code Change Proposals

• At least 3 separate proposals,
  – one authored by SEAOC Seismology Committee (submitted via NCSEA),
  – one by James Russell, City of Palo Alto, as an extension of SEAOC’s proposal, and
  – one by SEAOC building code committee.

• ICC will hear the proposals in October 2009 for inclusion in the 2012 IBC (if accepted).
THANK YOU