Structural Commentary

for the

National Simplified Residential Roof Photovoltaic Array Permit Guidelines

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to the

National Simplified Permit Guidelines for Residential Photovoltaic Arrays

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0.1 INTRODUCTION

This commentary provides the technical analysis that supports the structural provisions of the *National Simplified Residential Roof Photovoltaic Array Permit Guidelines* (the *Guidelines*)*,* also called "Step 1: Structural PV Array Mounting Requirements Checklist" (the "Checklist"). It describes the structural engineering principles and assumptions behind the *Guidelines* Checklist, and delineates how the document conforms to the *International Residential Code* (IRC) and *International Building Code* (IBC), the model codes upon which all USA state Building Codes are based.

The goal of the Checklist is to provide assurance that a solar array does not overload (1) an existing residential roof, or (2) the attachments to the roof. *These rules do not address the structural sufficiency of the components of the array above the roof.* It remains the installer's responsibility to ensure the components above the roof are structurally sufficient, typically achieved by adhering to the manufacturers' recommendations for the solar panel and support components.

While many of the provisions can also apply to multi-family residences and to metal-framed structures, for simplicity the Checklist is written explicitly for wood-framed, detached, single- and two-family structures, with the resilience and robustness associated with wood framing. In principle, the analysis could be extended to metal-framed roofs, but key factors such as the Concentrated Load Sharing Factor (C_{LSF}) would need to be adjusted for metal framing. See Section D.6 for further discussion of C_{LSF}.

The analysis also assumes that the wood-framed roof was designed to comply with the Building Code in effect at the time it was built. Building codes as far back as the early 1900s have required that roofs be designed to carry temporary construction loads termed "Roof Live Loads". Flush-mounted solar arrays are assumed to displace roof live loads, since piling bundles of shingles or other building materials on solar panels could scratch or damage the panels, and perhaps also slide off. Because the roof was designed for roof live load, where such loads cannot be placed, the roof has reserve load-carrying capacity to support solar panels.

The structural provisions of the *Guidelines* are based on several assumptions that encompass the great majority of detached single- and two-family homes. Key assumptions include:

- The building is wood-framed
- The building's structure was "code compliant" at the time it was built
- No significant deterioration or weakening has occurred since original construction
- The array is mounted parallel-to-roof, sometimes termed "flush-mounted"

0.2 CODE HISTORY

The assumption that the roof was "code compliant" at the time it was built, combined with verification that no significant deterioration or weakening has occurred since then, allows us to calculate the roof framing capacity based on the design rules used at that time. This in turn requires some knowledge of the history of Building Codes in the United States. Gregory J. McFann, a California building official, provides a good overview:

> *Since the early 1900s, the system of building regulations in the United States was based on model building codes developed by three regional model code groups. The codes developed by the Building Officials Code Administrators International (BOCA) were used on the East Coast and throughout the Midwest of the United States, while the codes from the Southern Building Code Congress International (SBCCI) were used in the Southeast and the codes published by the International Conference of Building Officials (ICBO) covered the West Coast. . . The nation's three model code groups decided to combine their efforts and in 1994 formed the International Code Council (ICC) to develop codes that would have no regional limitations.*

After the first IBC edition in 1997, a new edition has been released every three years.

International Residential Code (IRC) versus International Building Code (CBC): For many states, one- and two-family dwellings use the IRC instead of the IBC. Regarding structural requirements for wood-framed roofs, the requirements of the two codes are virtually identical. For instance, the roof rafter span tables governing conventional wood-framed construction in the two codes are identical. For other provisions, the IRC may lag behind the IBC in adopting reference standards such as ASCE 7. Wind loads are a good example, where the 201_ IRC still refers to ASCE 7-05, while the 201_ IBC uses ASCE 7-10, and is about to be updated to refer to ASCE 7-16.

For residential wood-framed structures, the structural provisions of the current IBC and legacy codes are largely consistent over the past century, with minor variations over time. The most notable of these minor code changes are:

0.2.1 Roof Live Load as a Function of Roof Slope

Roof live load has always been a function of roof slope, with reduced live loads at greater slopes. Before ASCE 7-05 (typically adopted by state codes around 2008) the decreases occurred at specific slopes. Under the older codes, roof live load drops from 20psf to 16psf at a 4:12 slope, and drops from 16 psf to 12 psf at 12:12 slope. Since the adoption of ASCE 7-05, roof live load continuously decreases as a smooth function as roof slope increases, with 20 psf at a flat slope, 16 psf at an 8:12 slope, and 12psf at a 12:12 slope.

Roof live load controls the design of roofs in regions of zero to low snow load. In these regions, solar arrays can be considered to offset roof live loads, justifying an orthogonal layout of mounts spaced relatively far apart. In low snow regions at 4:12 to 6:12 slopes, this creates minor differences in the maximum mount spacing and the snow load under which one must switch from orthogonal to staggered mount spacing.

0.2.2 Lumber Allowable Bending Stresses

Based on extensive testing of more than 70,000 specimens dating back to 1977, new allowable stress design values for sawn lumber were documented in the 1991 National Design Specification for Wood Construction. This was subsequently adopted in the 1994 UBC (and in similar years for BOCA and SSBCI), and subsequently adopted by the states. In California, for instance, the 1991 NDS was adopted in the 1995 Building Code, which started to be enforced in 1996 building designs, showing up in buildings constructed in 1997.

Despite being a major re-write of the code, the effect on design values was relatively minor. As stated in the Commentary to the 1991 NDS (article 4.2.3.2, p. 57) *"Strength design values based on in-grade test results are generally higher than previous assignments except for Fb values for the lower grades and larger widths.*"

Table 0.2.2 summarizes a comparison between three common lumber groups under pre-1994 and current code values. The three species are Douglas Fir-Larch, the most common framing lumber west of the Rocky Mountains, Spruce-Pine-Fir, the most common framing species in the Midwest, Northeast and mid-Atlantic states, and Southern Pine, the most common framing species in the south. For many wood grading species groups, such as Spruce-Pine-Fir (SPF) and Southern Pine (So. Pine), the new allowable stress values were essentially the same or even larger. Shallower members (2x4 and 2x6) saw the greatest increase in allowable stresses, while deeper members (2x10 and 2x12) had smaller increases or even small. Douglas Fir-Larch (DF-L), the most common wood species group used in the western states, had the largest drop in allowable stress values. Even for this species group, the changes do not become substantial until lumber depths reach 2x10 or deeper. Because the wind load duration factor increased from 1.33 to 1.60, wind load combinations had a greater increase than loads where duration factor remained unchanged, such as roof live load and snow load.

The following sections are organized to follow the sequence of items as they appear in the Checklist, and provide the technical justification for each item.

Table 0.2.2: Comparison of Allow. Bend. Stress for Three Common Lumber Groups under Pre-1991 & Post-1991 NDS														
New stress ratings adopted in 1991 NDS, 1994 UBC, 1995 CBC eff. 1996														
"Strength design values based on in-grade test results are generally higher than previous assignments except for the lower grades and larger widths." 1991 NDS Commentary														
	1944 - 1986 NDS, 1991 UBC					1991 NDS, 1994 UCB, 2015 IBC, 2016 CBC								
Doug Fir No. 1	$F_{b,r}$	$C_{D,snow}$	$C_{D,wind}$	$F_b'_{snow}$	$F_b'_{wind}$	F_b	C_F	C_r	$C_{D,snow}$	$C_{D,\ wind}$	$F_b'_{snow}$	$F_b'_{wind}$	new/old, snow	new/old, wind
2x4	2050	1.15	1.33	2358	2727	1000	1.5	1.15	1.15	1.60	1984	2760	0.84	1.01
2x6	1750	1.15	1.33	2013	2328	1000	1.3	1.15	1.15	1.60	1719	2392	0.85	1.03
2x8	1750	1.15	1.33	2013	2328	1000	1.2	1.15	1.15	1.60	1587	2208	0.79	0.95
2x10	1750	1.15	1.33	2013	2328	1000	1.1	1.15	1.15	1.60	1455	2024	0.72	0.87
2x12	1750	1.15	1.33	2013	2328	1000	1.0	1.15	1.15	1.60	1323	1840	0.66	0.79
Doug Fir No. 2	$F_{b,r}$	$C_{D,snow}$	$C_{D,wind}$	$F_b'_{snow}$	$F_b'_{wind}$	F_b	C_F	C_r	$C_{D,snow}$	$C_{D,\ wind}$	$F_b'_{snow}$	$F_b'_{wind}$	new/old, snow	new/old, wind
2x4	1650	1.15	1.33	1898	2195	900	1.5	1.15	1.15	1.60	1785	2484	0.94	1.13
2x6	1450	1.15	1.33	1668	1929	900	1.3	1.15	1.15	1.60	1547	2153	0.93	1.12
2x8	1450	1.15	1.33	1668	1929	900	1.2	1.15	1.15	1.60	1428	1987	0.86	1.03
2x10	1450	1.15	1.33	1668	1929	900	1.1	1.15	1.15	1.60	1309	1822	0.79	0.94
2x12	1450	1.15	1.33	1668	1929	900	1.0	1.15	1.15	1.60	1190	1656	0.71	0.86
SPF No. 1	$F_{b,r}$	$C_{D,snow}$	$C_{D,wind}$	$F_b'_{snow}$	$F_b'_{wind}$	F_b	C_F	C_r	$C_{D,snow}$	$C_{D,\ wind}$	$F_b'_{snow}$	$F_b'_{wind}$	new/old, snow	new/old, wind
2x4	1400	1.15	1.33	1610	1862	875	1.5	1.15	1.15	1.60	1736	2415	1.08	1.30
2x6	1200	1.15	1.33	1380	1596	875	1.3	1.15	1.15	1.60	1504	2093	1.09	1.31
2x8	1200	1.15	1.33	1380	1596	875	1.2	1.15	1.15	1.60	1389	1932	1.01	1.21
2x10	1200	1.15	1.33	1380	1596	875	1.1	1.15	1.15	1.60	1273	1771	0.92	1.11
2x12	1200	1.15	1.33	1380	1596	875	1.0	1.15	1.15	1.60	1157	1610	0.84	1.01
SPF No. 2	$F_{b,r}$	$C_{D,snow}$	$C_{D,wind}$	$F_b'_{snow}$	$F_b'_{wind}$	F_b	C_F	C_r	$C_{D,snow}$	$C_{D,\ wind}$	$F_b'_{snow}$	$F_b'_{wind}$	new/old, snow	new/old, wind
2x4	1150	1.15	1.33	1323	1530	775	1.5	1.15	1.15	1.60	1537	2139	1.16	1.40
2x6	1000	1.15	1.33	1150	1330	775	1.3	1.15	1.15	1.60	1332	1854	1.16	1.39
2x8	1000	1.15	1.33	1150	1330	775	1.2	1.15	1.15	1.60	1230	1711	1.07	1.29
2x10	1000	1.15	1.33	1150	1330	775	1.1	1.15	1.15	1.60	1127	1569	0.98	1.18
2x12	1000	1.15	1.33	1150	1330	775	1.0	1.15	1.15	1.60	1025	1426	0.89	1.07
So. Pine No. 1	$F_{b,r}$	$C_{D,snow}$	$C_{D,wind}$	$F_b'_{snow}$	$F_b'_{wind}$	F_b	C_F	C_r	$C_{D,snow}$	$C_{D,\ wind}$	$F_b'_{snow}$	$F_b'_{wind}$	new/old, snow	new/old, wind
2x4	1950	1.15	1.33	2243	2594	1850	1.0	1.15	1.15	1.60	2447	3404	1.09	1.31
2x6	1700	1.15	1.33	1955	2261	1650	1.0	1.15	1.15	1.60	2182	3036	1.12	1.34
2x8	1700	1.15	1.33	1955	2261	1500	1.0	1.15	1.15	1.60	1984	2760	1.01	1.22
2x10	1700	1.15	1.33	1955	2261	1300	1.0	1.15	1.15	1.60	1719	2392	0.88	1.06
2x12	1700	1.15	1.33	1955	2261	1250	1.0	1.15	1.15	1.60	1653	2300	0.85	1.02
So. Pine No. 2	$F_{b,r}$	$C_{D,snow}$	$C_{D,wind}$	$F_b'_{snow}$	$F_b'_{wind}$	F_b	C_F	C_r	$C_{D,snow}$	$C_{D,\ wind}$	$F_b'_{snow}$	$F_b'_{wind}$	new/old, snow	new/old, wind
2x4	1650	1.15	1.33	1898	2195	1500	1.0	1.15	1.15	1.60	1984	2760	1.05	1.26
2x6	1400	1.15	1.33	1610	1862	1250	1.0	1.15	1.15	1.60	1653	2300	1.03	1.24
2x8	1400	1.15	1.33	1610	1862	1200	1.0	1.15	1.15	1.60	1587	2208	0.99	1.19
2x10	1400	1.15	1.33	1610	1862	1050	1.0	1.15	1.15	1.60	1389	1932	0.86	1.04
2x12	1400	1.15	1.33	1610	1862	975	1.0	1.15	1.15	1.60	1289	1794	0.80	0.96

Table 0.2.2. Comparison of Allowable Bending Stress for Three Common Lumber Groups under Pre-1991 & Post-1991 NDS

A. GENERAL SITE AND ARRAY REQUIREMENTS

A.1. Wind Exposure and Design Wind Speed

A.1.a. Member-Attached System: Exposure B or C, and design wind speed does not exceed 150 mph.

Member-attached systems are those systems where the mounts/feet/stand-offs fasten through the roof sheathing into rafters or the top chords of manufactured trusses. With this system, design wind speeds are limited to 150 mph (per ASCE 7-10). This encompass almost all the land area of the continental United States, except for the southern half of Florida. This limits allowable stress design (ASD) uplift demand pressures to 25.7 psf (140 mph, Exp. C, 30 ft mean roof height, gable roof with slope less than 7 degrees).

The capacity against uplift is usually limited by the fastener(s), typically one or two lag screws or a self-drilling screws, between the mount to the wood member.

The uplift pressure described here, and in other sections, can be reduced significantly by applying the "Kopp factor", which recognizes that most solar arrays can be considered "air-permeable cladding" (Stenabaugh et al, 2014). Wind tunnel research shows that the Kopp factor ranges from 0.8 to as low as 0.4, and depends on the height of the modules off the roof (smaller is better) and the gaps between modules (bigger is better).

A.1.b. Sheathing-Attached System:

- ☐ i. *Exposure C (open terrain/fields), and design wind speed does not exceed 120 mph, or*
- ☐ ii. *Exposure B (urban, suburban and wooded areas more than 500 yards from open terrain), and design wind speed does not exceed 140 mph.*

Sheathing-attached systems anchor to plywood or oriented strand board that in turn is nailed to rafters or the top chord of trusses. The uplift capacity may be limited by either the new sheathing connection, or the existing nailing of the sheathing to the rafters or trusses. Mount fastening to the sheathing depends on the specific mounting product, and is assumed to be sufficient. Sheathing-to-rafter nailing strength has been studied extensively by one sheathing-attached manufacturer, SMASHsolar, which conducted scores of full-size tests of the capacity of sheathing to resist concentrated uplift loads from mounts.

The 120 mph Exposure C and 140 mph Exposure B both limit ASD uplift demand pressure for systems attached to bands of strength 16.5 psf (120 mph Exposure C, 30 ft mean roof height, gable roof with slope less than 7 degrees).

A.2. The Structure is not in Wind Exposure D (within 200 yards of a water body wider than a mile).

Exposure D uplift forces are 17 percent higher than Exposure C. Adding Exposure D was judged not worth the complexity of addressing this unusual case, which only occurs within 200 yards of the ocean, the Great Lakes or other large bodies of water wider than one mile. Note that in reality 130 mph Exposure D has about the same uplift wind pressure as 140 mph Exposure C, so Exposure D conditions in design wind speed areas less that the maximum speed are probably acceptable, require special calculation to justify.

A.3. The structure is not on a hill with a grade steeper than 5%, where topographic effects can significantly increase wind loads.

Where hills have grades steeper than 5%, wind accelerates as it flows over such hilltops, and these topographic effects can significantly increase wind loads. Projects on the top half of steep hills, especially in regions at the limit of wind exposure and wind Speed, require special calculations.

A.4. Ground snow loads do not exceed 60 psf.

Snow loads greater than 60 psf are unusual, and deserve closer examination. For the rails (or long edges in rail-less systems) to carry such loads, the spacing between anchors/feet/mounts/stand-offs may need to be very small. The panels themselves may not be designed to carry such loads (standard minimum rating for panels used to be 30 psf, and has recently been reduced to 15 psf). Finally, the loads to the roof need to be checked –if the cross-slope mount spacing skips over rafters, it is crucial to stagger the mount layout between rows to effectively load every rafter. In truth, all these considerations apply even to snow loads as small as 20 psf, but become critical at higher ground snow loads, especially at flatter slopes.

It is important to note that ground snow load does not translate directly to snow loads perpendicular to the face of panels. *Figure A.4.1* shows panel load as a function of roof slope for 20 psf, 40 psf and 60 psf ground snow load. Note that per the commentary in section C7.8 of ASCE 7-10, solar "collectors" (presumably both solar thermal and solar PV) can be designed as unobstructed slippery surfaces using Figure 7-2a in the ASCE standard, which is otherwise typically applied to "warm roofs". Note that C_S, the thermal snow factor, remains 1.2 to reflect outside open air conditions.

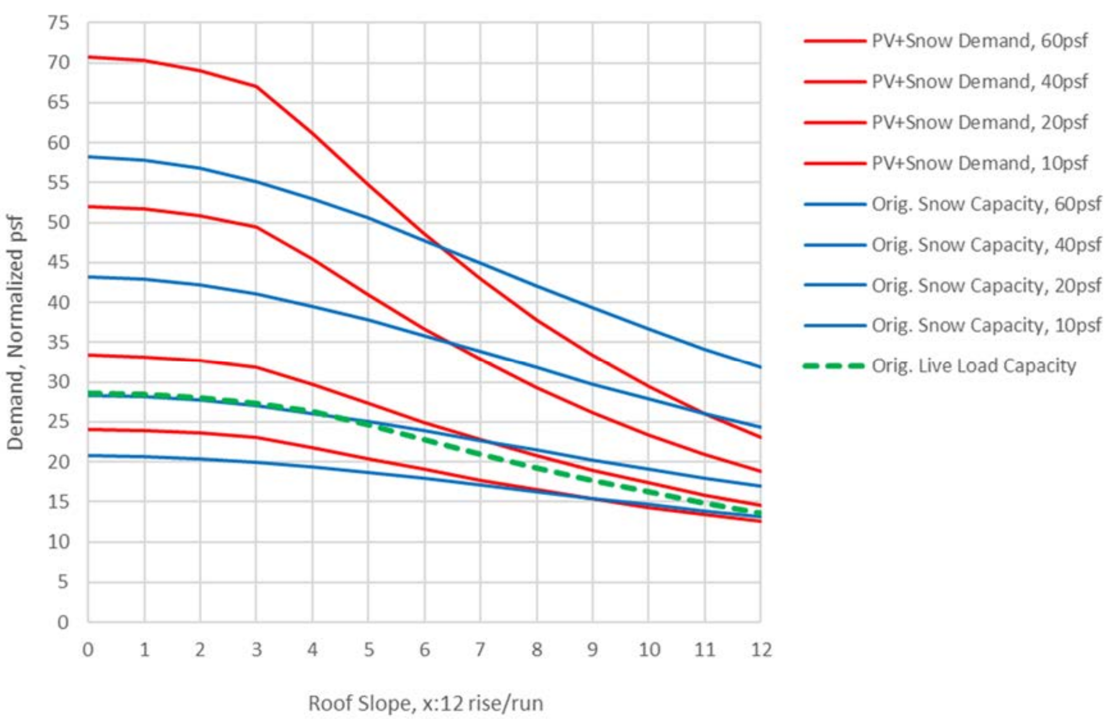


Figure A.4.1. Panel Snow Load as a Function of Roof Slope

A.5. Distributed weight of the PV array is less than 4 lbs/ft^2 (5 lbs/ft^2 for thermal systems).

Practical weight limits need to be set for solar systems. The 4 psf average self-weight limit of a PV array, including its support components, is easily met by virtually all PV systems. A 5 psf weight limit for thermal solar collectors is likewise usually met. These limits are similar to the weight of roof overlays, which were usually allowed automatically in 1990s and earlier Building Codes.

B. ROOF INFORMATION

B.1. The array is mounted on a permitted one- or two-family roof structure or similar structure.

If the roof is not permitted, the building official can either assume the building has stood the test of time and is essentially code compliant, or ask to show that the roof rafter spans comply with the International Residential Code (IRC) roof span tables.

If span tables are applied to per-1960 lumber, credit should be given for lumber sizes that are greater than current nominal lumber sizes. This correction factor typically ranges from 1.13 to 1.16, allowing 13% to 16% longer spans than current tables. Because pre-1960 lumber was often cut from larger trees, especially on the west coast, it is often reasonable to assume No.1 grade lumber.

If lumber grade stamps are not visible, in applying the IRC span tables in jurisdictions west of the Rocky Mountains, it may be reasonable to assume the lumber is No. 1 Douglas Fir-Larch. For southern states (Texas to Florida, and up to North Carolina) it may be reasonable to assume No. 1 Southern Pine. For mid-western and northeastern states, it may be reasonable to assume No. 1 Spruce-Pine-Fir.

The Structural Criteria are based on an important underlying assumption that the existing roof was code-compliant at the time of construction, and has not deteriorated since then. One significant question for those designing criteria for expedited residential solar permitting is whether rafter span checks should be made to verify that an existing roof is code compliant, or whether to instead assume the roof was originally designed to meet Building Code requirements at the time of construction. This decision requires considerable judgment, and reasonable engineers and code officials can and do have differing opinions on this question.

Choose by Advantage: One way of exploring the options for verifying that an existing roof is code compliant is through a "Choose by Advantage" (CBA) process, where key stakeholders such as code officials, structural engineers and solar industry representatives meet to list and quantify the advantages of various options. *Figure B.1.1* illustrates one possible outcome of such a process. In this example, the "Trust but Verify" option has the greatest advantages, but the "Accuracy Trumps Simplicity" option comes in a close second, where span tables for pre- and post-1960's vintage construction are used.

Solar Permitting Initiative Existing Roof Rafter Span Check Options

	Option	Condition Check	Span Check	Bldg Age: Pre-1960 / Span Table: Pre-1960	Bldg Age: Pre-1960 / Span Table: Post-1960	Bldg Age: Post-1960 / Span Table: Pre-1960	Bldg Age: Post-1960 / Span Table: Post-1960	Nickname	Simplicity (10)	Rigor (8)	Least Cost (8)	Choose By Advantage Score
1	No Span Check, No Condition Check							"Trust Everybody"	10	0	8	18
2	No Span Check, Do Condition Check	✓						"Trust but Verify"	9	3	7	19
3	Check Pre-1960 Only w/ Pre-1960 Table	✓	✓	✓				"Trust New Stuff"	5	5	6	16
4	Check All Bldgs w/ Pre-1960 Table (EBGC)	✓	✓	✓		✓		"Sorta Trust New Stuff"	6	5	6	17
5	Check All Bldgs w/ Applicable Tables	✓	✓	✓			✓	"Accuracy Trumps Simplicity"	4	8	6	18
6	Check All Bldgs w/ Modern Table	✓	✓		✓		✓	"Flunk Old Stuff"	6	7	2	15

Figure B.1.1. Hypothetical results of a "Choose by Advantage" process where stakeholders meet to list and quantify the relative advantage of various options. In this example, the "Trust but Verify" option has the greatest advantages, but the "Trust Everybody" and "Accuracy Trumps Simplicity" options tie for second place.

The simplest version of the Structural Criteria uses the "Trust but Verify" approach. While checking for significant structural deterioration is always appropriate, omitting horizontal rafter span checks is considered appropriate, based on the following reasoning:

- Most roof structures are designed properly and are code compliant.
- Visual survey is done to check against weakening factors such as decay, fire damage or removal of truss web members.
- Roof overlays (reroofs) of similar weight to solar arrays have been allowed for many years, with no history of failures for sloping shingled roofs.
- The effect of placing an array on a non-compliant roof structure may, in a few cases, result in sagging and distress to finishes, alerting the owner to a problem and providing time to address. The chance of roof collapse is negligible due to roof sheathing's catenary and composite action. For instance, the Structural Engineers of Washington reports on the aftermath of a heavy snow load event where 57 roofs were damaged, but only two partial collapses occurred. Snow loads, with ongoing downward pressures that can drive a roof to collapse, are very different from the dominant wind load case in most of California, where downward wind loads are ephemeral and much less likely to drive a roof structure to collapse.
- Concentrated load effects from solar arrays are minimized if these guidelines are followed. Overloads from solar arrays on a non-compliant roof will result in Demand-Capacity Ratios (DCRs) of similar magnitude as the original DCR of the non-compliant roof for the dead load plus roof live load combination.

- The installation process of panels and workers on the roof is itself essentially a roof load test. If problems of over-deflection and rafter breakage do not occur during the solar array installation process, similar problems are unlikely to occur during service life, especially in regions of modest wind loads and zero snow loads typical of most of California.

B.2. Roof is framed with wood rafters or trusses at no greater than 48" on center. Roof framing members run upslope/downslope (not horizontal purlins).

These are basic assumptions about the roof framing configuration that will apply to almost all residential structures.

B.3. Roof structure appears to be structurally sound, without signs of alterations or significant structural deterioration or sagging.

Figure B.3.1, taken from the *California Solar Permitting Guidebook*, illustrates more specific checks regarding weakening alterations and deformations severe enough to raise concerns.

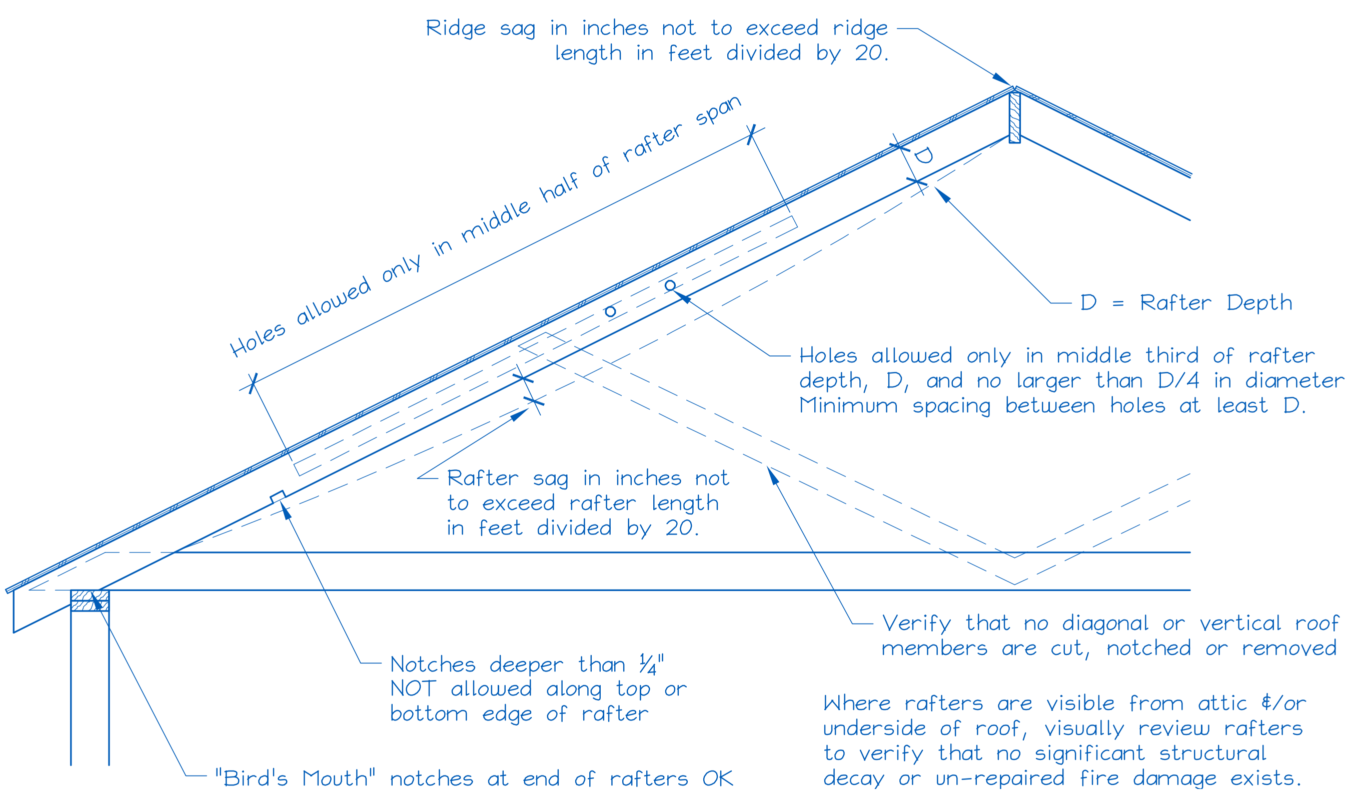


Figure B.3.1. Roof Visual Structural Review (Contractor's Site Audit) of Existing Conditions

The site auditor should verify the following:

1. No visually apparent disallowed rafter holes, notches and truss modifications as shown above.
2. No visually apparent structural decay or un-repaired fire damage.
3. Roof sag, measured in inches, is not more than the rafter or ridge beam length in feet divided by 20.

Rafters that fail the above criteria should not be used to support solar arrays unless they are first strengthened. Excessive roof sag can indicate an originally under-designed roof, or subsequent deterioration of a correctly designed roof. Roof sag, measured in inches, is not to exceed span, measured in feet, divided by 20. This corresponds to a dead load deflection of span L/240. Per IBC, dead plus live load deflections are not to exceed L/180, and if dead load is 10 psf and live load is in the range of 12 to 20 psf, the expected original dead load design deflection is of the order of one third to one half of L/180, that is, L/360 to L/540. Hence a larger dead load deflection of L/240 could indicate problems, warranting further investigation.

B.4. Sheathing is at least 7/16" or thicker plywood, or 7/16" or thicker oriented strand board (OSB).

The anchor spacing limitations described in section D are based, in part, on assumptions about how concentrated loads from the mounts loading one rafter can be shared by adjacent rafters. This factor is called the Concentrated Load Sharing Factor, C_{LSF}, a function of the ratio of sheathing stiffness to rafter stiffness. A lower bound value for this factor is based on plywood or OSB at least 7/16" thick. See Section D.6 for further discussion.

B.5. If composition-shingle, roof has a single roof overlay (no multiple-shingle layers). If not, show compliance with IRC span tables.

The existing roof shall not have a reroof overlay, for the following reasons:

To avoid "double-loading" the roof with both solar modules and a roof overlay.

To avoid adding so much mass to the roof from both solar arrays and reroof overlays that top story seismic loads increase by more than 10%, triggering seismic evaluation and potentially seismic strengthening per 2013 CBC Chapter 34.

To maintain the water tightness reliability of many types of anchors/stand-offs/feet/mounts/attachment points.

To avoid costly reroofing during the service life of the solar array. Because roof overlays often have a remaining expected service life shorter than a new solar array, placing modules over a roof overlay may be unwise because of the likelihood that the roof will need to be replaced before the twenty-year or longer service life of the solar array. Replacing a roof during the service life of a solar array can be a costly unnecessary expense.

To avoid reductions in lag screw capacity. A roof overlay creates a significantly thicker roofing assembly, forcing lag screw anchors to cantilever farther from the rafters. This can also reduce lag screw embedment. Both effects can reduce anchor shear and withdrawal capacities.

Recent and current Building Codes allow one asphalt composition reroof over an existing asphalt composition roof on a building of any vintage without requiring structural calculations. Previously, from 1979 through 1994, two reroofs over the original roof were explicitly allowed (UBC 1979 Appendix Chapter 32 "Reroofing" through UBC 1994 Appendix Chapter 15 "Reroofing"). One reroof over the original roof has been explicitly allowed for all vintage buildings since 1997 (UBC 1997 Appendix Chapter 15 "Reroofing" through CBC 2013 Chapter 15, Article 1510 "Reroofing"). The last two editions of the code have added the proviso that reroofing is allowed provided that the roof structure is sufficient to carry the reroof overlay. Many code officials allow reroof overlays without requiring calculations showing sufficient lateral strength, since structural overload problems from reroof overlays are very rare.

According to a year 2000 technical brief by Tom Bollnow, Director of Technical Services for the National Roofing Contractors Association, typical 30-year asphalt roofs (or added reroofs) weigh up to 3.25 psf, 40-year asphalt roofs up to 3.85 psf, and lifetime roofs up to 4.25 psf (ref: http://www.professionalroofing). net/archives/past/july00/qa.asp). The historical experience is that wood shingle and composition shingle reroof overlays seldom cause structural problems. This is codified in the *International Existing Building Code* (IEBC), which explicitly allows the "addition of a second layer of roof covering weighing 3 pounds per square foot or less" (IEBC Article 707.2, exception 3). This can be used to justify the added weight of an equivalent solar array, so long as the solar array uniformly loads the roof by being anchored to *every* rafter (or anchored to every other rafter in a staggered row-to-row pattern). Note that unlike sloping wood shingle and composition shingle reroofs, excessive built-up reroofing overlays on flat roofs is a relatively common problem that sometimes results in problematic structural overloading.

Reroof overlays can increase seismic loads significantly. The increase in inertial mass (and subsequent shears at the top story) might be 3 psf / 25 psf = 12%, which exceeds the 2012 IBC Chapter 34 (later adopted into the 2015 IEBC) limit of no more than 10% increase in seismic loads before seismic re-valuation and potential seismic strengthening is required. Note that the denominator includes the weight of the roof, ceiling and top half of the walls of a one-story building. For multistory buildings, the code static-equivalent triangular lateral force distribution will further "dilute" (reduce) the shear increase percentage. Even if the 10% rule of Chapter 34 is slightly exceeded, wood-framed residences are typically very resistant to seismic collapse once obvious weak spots like unsheathed cripple walls are addressed. However, adding a solar array to the south half of the roof could add an additional 3.5 psf x 40% / 20 psf = 7 %, so a solar array plus reroof overlay could easily amount to 12% + 7% = 19%, well over the 10% limit. Hence, in seismically active regions of California (i.e. most of the state), for seismic load reasons alone, placing solar arrays over reroof overlays is not recommended and likely to be a code violation.

B.6. Mean roof height is not greater than 40 feet (member-attached) or 30 feet (sheathing-attached)

Wind loads on a roof-mounted solar array increase with mean roof height. Mean roof height is shown in *Figure B.6.1*. The wind checks in the structural provisions of the *Guidelines* assume that the great majority of one- and two-family residences in a jurisdiction have a mean roof height less than or equal to 30 feet.

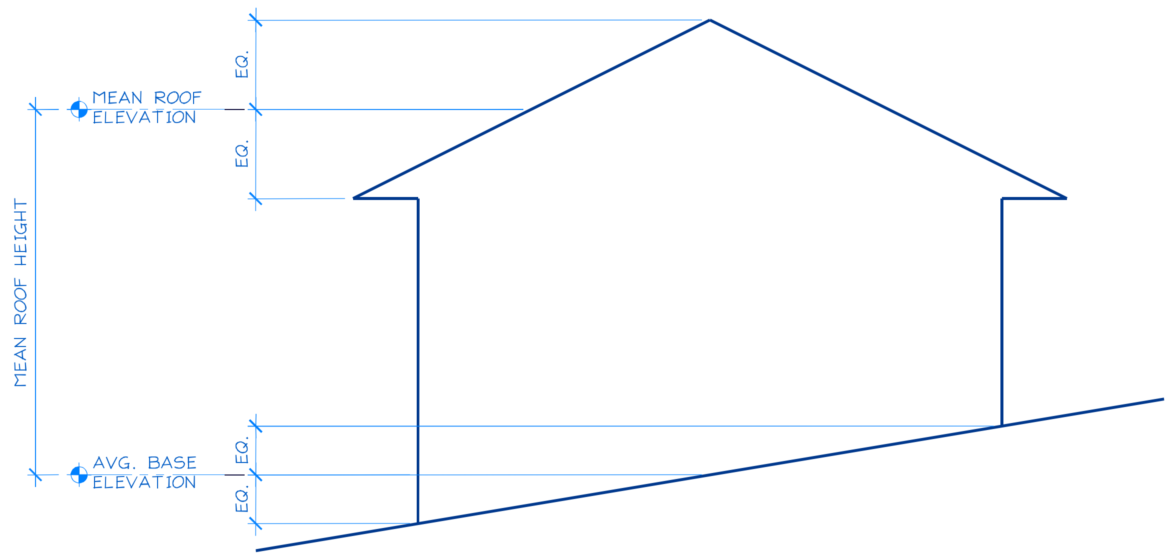


Figure B.6.1. Definition of mean roof height. The permitting guidelines assume a mean roof height of 30 feet or less.

B.7. In areas of significant seismic activity (Seismic Category C, D, E or F), PV array covers no more than half the total area of the roof (all roofs included).

To avoid significantly adding to the inertial mass of the roof and seismic lateral loads, limits are set on the maximum roof area covered by solar arrays. To understand why the limit is set at half the total roof area, it's instructive to look at a typical case: solar array weighs 3.5 psf, and covers 80% of the south facing gable roof. In a single story building, the global increase in lateral loads to the building would be: 3.5 psf x 40% / 20 psf = 7 % (less than the 10% trigger in 2012 IBC Chapter 34, article 3404.4). Plan torsion effects may make loads to individual elements slightly greater than 7%, but still likely to be less than 10%. If the building was more than one story tall, multistory effects would further dilute (reduce) the percentage increase in loads. To keep solar arrays from adding more than 10% to the seismic loads of the building, limiting the array coverage to no more than half the total roof area appears to be appropriate for most cases. Note that the "total roof area" is the sum of all roof planes, not just the roof plane where the array is located.

The re-roofing allowance that's been in the UBC since 1979 (and implicit before that) essentially allows a reroof overlay over the entire roof, and typically weighs between 2 to 4 psf (20 yr roof = approx 2 to 2.5 psf; 40 or 50 yr roof = approx 3.5 to 4 psf). Most code officials allow this without requiring calculations showing sufficient lateral strength, and there have been few problems from allowing these overlays. This appears to be the case even though the increase in inertial mass (and subsequent shears at the top story) might be 4 psf / 20 psf = 20%, although typical installations are closer to 3 psf / 25 psf = 12%. Note that the denominator includes the weight of the roof, ceiling, and top half of the walls of a one-story building. For multistory buildings, the code static-equivalent, triangular, lateral-force distribution will further "dilute" (reduce) the shear increase percentage. Even if the 10% rule of IBC Chapter 34 is slightly exceeded, wood-framed, one- and two-family dwellings are typically very resistant to seismic collapse once obvious weak spots like unsheathed cripple walls are addressed.

It's important to note, then, that a typical reroof overlay places greater seismic demands on a building's lateral system than a typical PV system.

C. ARRAY MOUNTING EQUIPMENT INFORMATION*:*

The following information needs to be filled in so that the mounting equipment can be identified.

1. Mounting Equipment Manufacturer
2. Product Name and Model Number
3. UL-2703 fire rating for the PV modules used in the project. Fire rating Class (A, B, or C).
4. Specify anchor-to-roof sealing (e.g. flashing, or sealant compatible with roofing)

The fire rating class is listed in case the state or local jurisdiction has requirements for fire-rating class A, B, or C. If so, the local jurisdiction may want to review this item further, or develop and enforce a list of required conditions for each fire-rating class.

The anchor-to-roof sealing item brings attention to an important aspect of solar mounting systems. Waterproofing failures are the most common cause of eventual attachment problems, and even failures, of residential rooftop solar support systems.

D. MEMBER-ATTACHED ARRAY REQUIREMENTS

D.1. Array is set back from all roof edges and ridge by at least twice the gap under the modules (or more, where fire access pathways are required).

This minimum set back rule is based on wind tunnel studies that show that as wind passes over a roof edge, it creates a high-velocity shear layer that bends toward the roof plane as it crosses over a building wall-roof edge interface (see *Figure D.1.1*). The angle of this shear layer in relation to the roof plane varies with time, and is affected by the angle between the wall-and-roof planes, or at hips and ridges, the angle between two roof planes. Solar module edges that align with the roof edge are within the shear layer, effectively turning the edge of the modules into roof overhangs. As the modules are pulled back away from the roof edge, their tendency to catch the shear layer updraft is reduced. According to Dr. David Banks (per. comm. e-mail to J. Wolfe, 3/18/2013), if "gap" is defined as the distance from the roof surface to the underside of the module, then the module should be set back about two gap lengths from any roof edge to ensure the module is outside the shear layer zone. ICC AC 428 addresses this effect by simply requiring that all modules be set back 10" from all roof edges. That rule may be conservative for gaps less than 5 inches, but is not conservative for gaps greater than 5”.

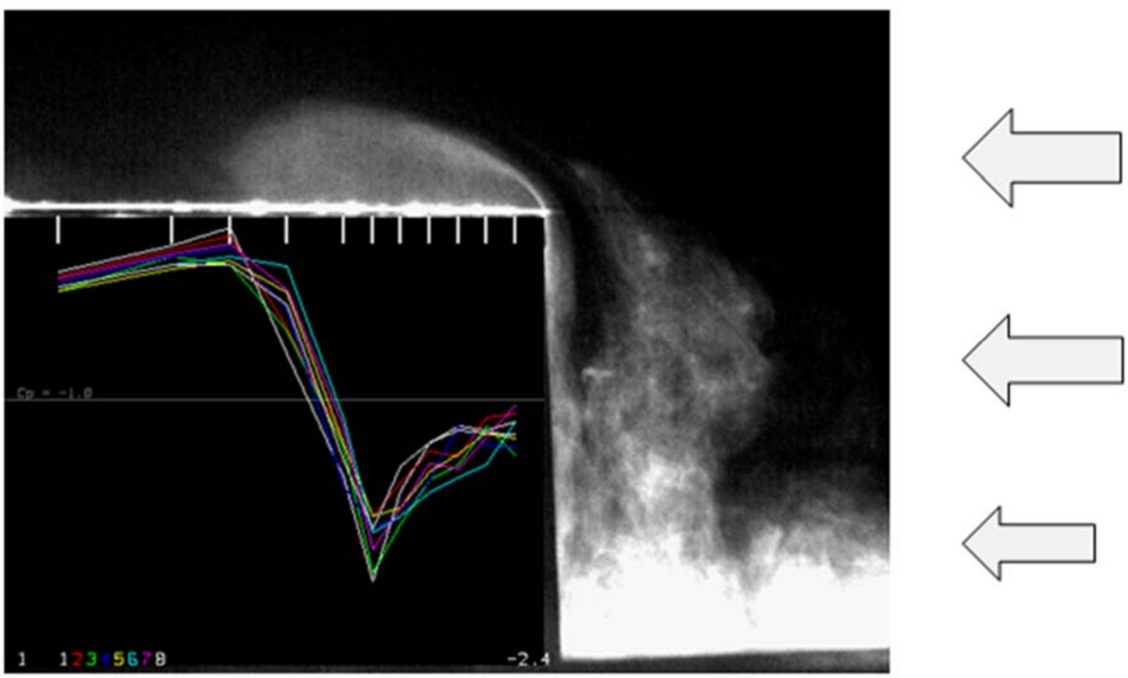


Figure D.1.1. Wind tunnel study showing high velocity shear layer near a roof edge, courtesy of Dr. David Banks. Dr. Banks explains: "This image is from my dissertation, and it is actually a still shot from an image sequence; the movie shows that the shear layer flaps up and down a fair bit. This is why we recommended V:2H. The position/shape of the shear layer will differ for roofs with eaves and high slopes, so I would be careful about drawing too many general conclusions from this sharp corner, low-rise, flat roof study."

The set back distance may be even more, where fire access pathways are required. For instance, California Cal Fire provisions usually require three feet between the ridge and the top of the array, to allow firefighters to scramble across the ridge punching holes to vent hot gases in a structure fire.

D.2. Array does not cantilever over the perimeter anchors more than 19”.

An array with large cantilevers can create loads on the end mounts that are significantly greater than other mounts. When the cantilever extends over the right or left end mounts, the rafter under these mounts can be overloaded under snow down or wind up loads. The simpler case is snow load, with no special roof edge effects – all rafters have been designed for the same uniform snow load. A cantilever analysis was made, studying cantilevers with different backspan conditions, looking at both the number of backspans and the boundary condition at the most interior backspan. Those boundary conditions were either (1) simply supported/free to rotate, or (2) fixed/restrained against rotating. The latter condition effectively models an array twice as large, symmetrically mirrored about this point of rotation fixity. The results in Figure D.2.1 suggest that for mounts at 48” on center, the largest cantilever that can be installed is about 19” before loads on the end mount exceed loads on interior mounts.

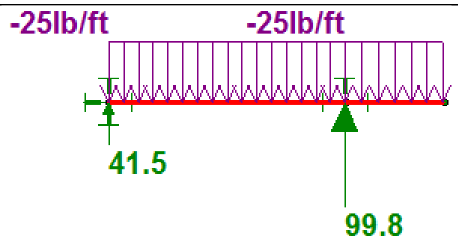
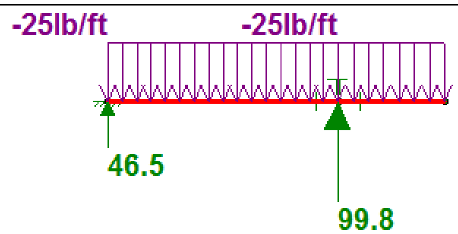
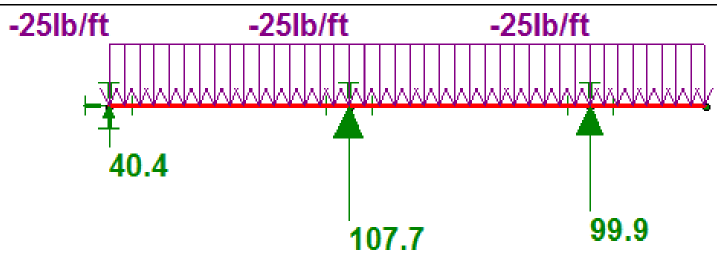
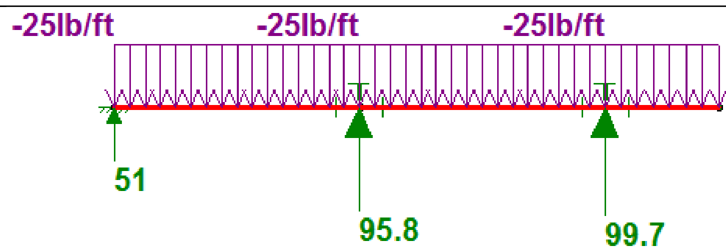
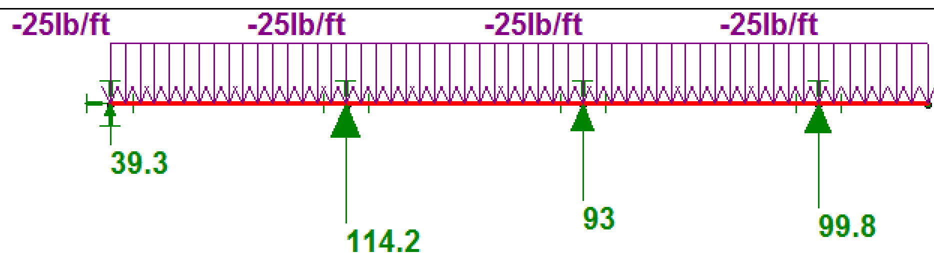
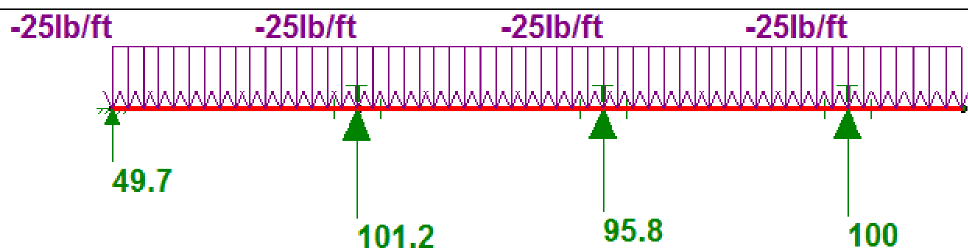
NATIONAL SOLAR ANCHORING RULES-MAX CANTILEVER AT THE END OF ARRAY		
		X (in)
1A		19.5
1B		22
2A		23
2B		22
3A		22
3B		22

Figure D.2.1. Cantilever analysis with varying number of backspans and varying boundary conditions at interior-most span.

D.3. Gap under modules (roof surface to underside of module) is no greater than 10".

For parallel-to-roof arrays, the distance between the roof surface and underside of module needs to be limited to 10 inches to control wind uplift pressures and take advantage of the "Kopp factor." Wind tunnel research (Stenabaugh et al, 2014) shows that this reduction factor is 0.80 or less for arrays up to 10 inches off the roof. See the discussion under E.3 for more information.

D.4. Gaps between modules

- ☐ *D.4.a. at least 0.25" on both short and long sides of modules, or*
- ☐ *D.4.b. 0" on short side, and at least 0.50" on long sides.*

The gaps between modules are key to reducing wind uplift and justifying the 0.8 reduction factor described in item D.3 above.

D.5. Mounting rail orientation or rail-less module long edges

- ☐ *D.5.a. run perpendicular to rafters or trusses, and are attached to them; or*
- ☐ *D.5.b. run parallel to rafters and are spaced no more than 4'-0" apart, ground snow load is no greater than 10 psf, and design wind speed does not exceed 120 mph.*

This section addresses the typical case, where rails run perpendicular to rafters (D.5.a), and the unusual case where the rails run upslope/downslope aligned with rafters (D.5.b). In the former case, section D.6 addresses the spacing and loading limits, while in the latter case, D.5.b addresses the spacing and loading limits.

D.6. The anchor/mount/stand-off spacing perpendicular to rafters or trusses

- ☐ *D.6.a. does not exceed 4'-0", and anchors in adjacent rows are staggered where rafters or trusses are at 24" or less on center (see Figure D.6.1); or*
- ☐ *D.6.b. does not exceed 4'-0", anchor layout is orthogonal, roof slope is 6:12 or less, ground snow load is no greater than 10 psf, and design wind speed does not exceed 120 mph; or*
- ☐ *D.6.c. does not exceed 6'-0", anchor layout is orthogonal, roof slope is 6:12 or less, ground snow load is zero, and design wind speed does not exceed 120 mph.*

The rules above are based on extensive calculations that examine the transition from a demand capacity ratio (DCR) less than one (acceptable) to greater than one (unacceptable) as a function of design wind speed, wind exposure (B, C or D), roof slope, and other factors. Some of the key assumptions behind this analysis are described below.

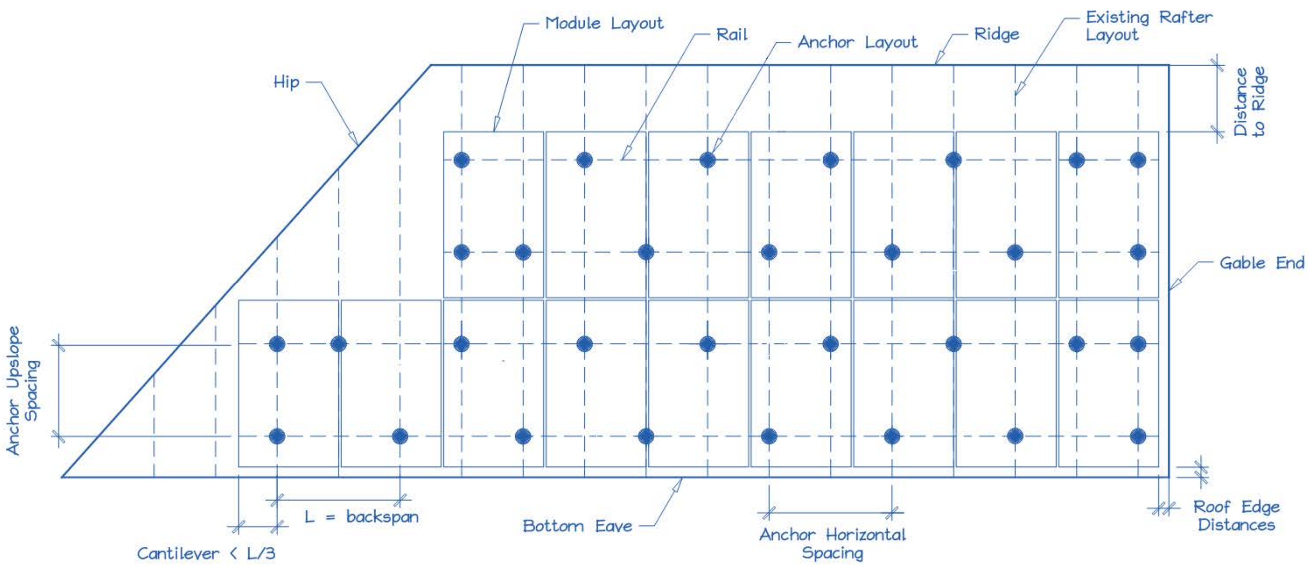


Figure D.6.1. Solar Panel Array and Staggered Anchor Layout Example (Roof Plan)

Concentrated Load Sharing Factor (C_{LSF})

Solar arrays anchored to every second, third, or fourth rafter concentrate solar array dead loads and wind downward loads onto a single rafter. For solar array dead and wind loads, the effective tributary width for that rafter becomes the anchor spacing rather than the rafter spacing. This concentration of loads is ameliorated by the tendency of adjacent rafters to redistribute concentrated loads by the spreading effect of the roof sheathing (typically plywood, oriented strand board or 1x sheathing). RISA-3D models were made to compare the ratio of moments on a rafter with no load sharing to that on a rafter with sheathing that can spread loads to adjacent rafters. Uniform loads and patterns of concentrated loads were assessed. See Figure D.6.2, Figure D.6.3, and Figure D.6.4.

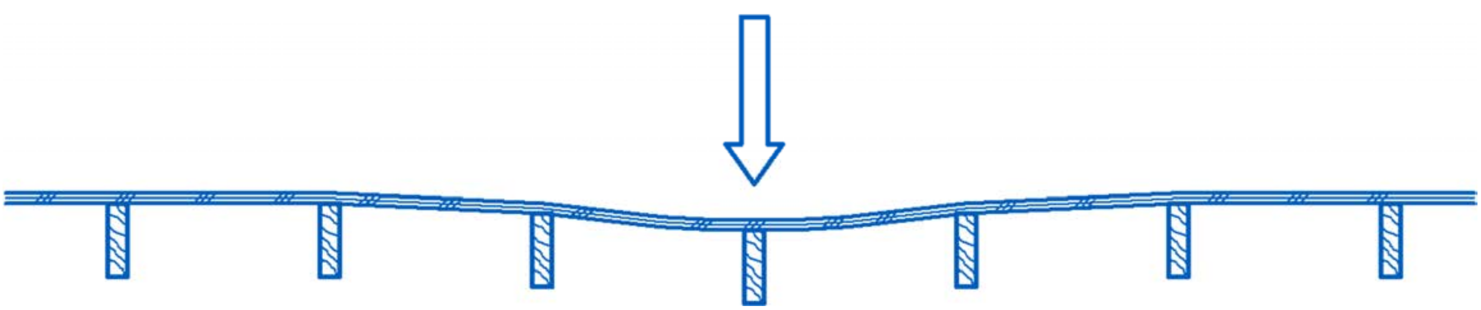


Figure D.6.2. Illustration of the concentrated load redistribution effect, where sheathing interconnects rafters so that a load concentrated on one rafter is shared by adjacent rafters. The Concentrated Load Sharing Factor, C_{LSF}, can be thought of as the effective number of rafters that resist a concentrated load imposed on a single rafter.

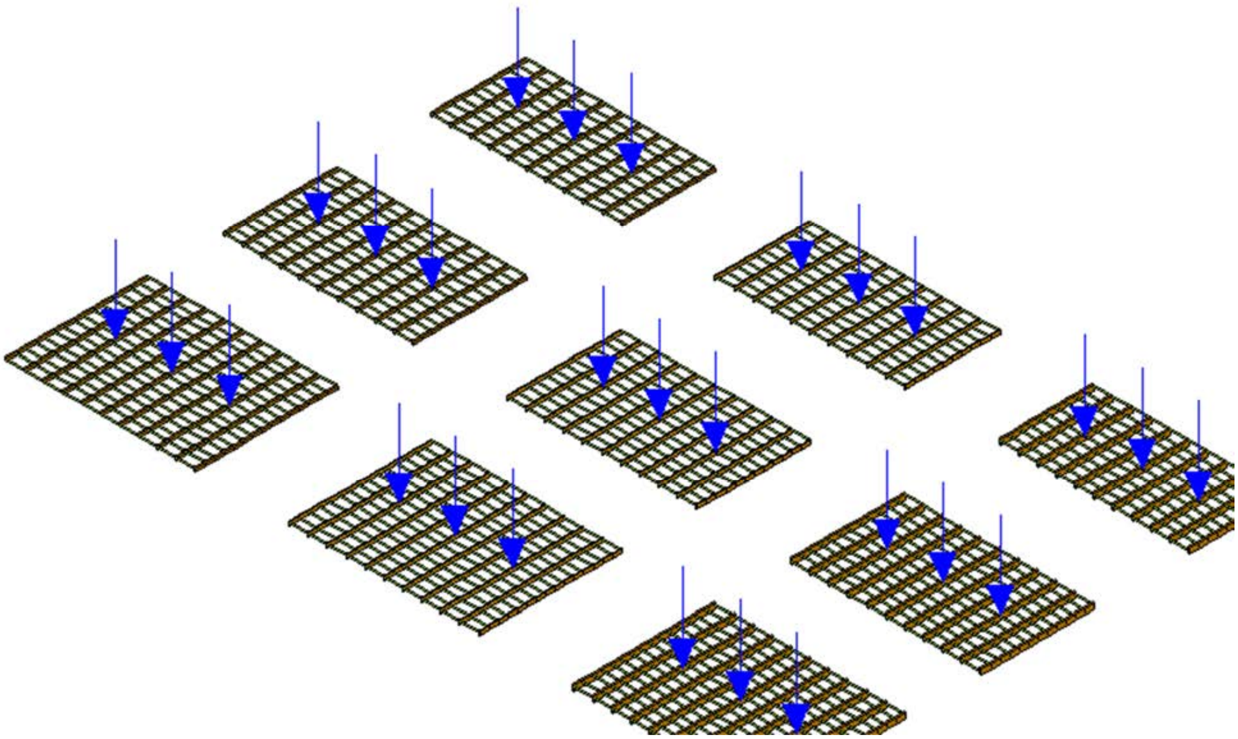


Figure D.6.3. Subset of RISA-3D models to determine Concentrated Load Sharing Factors. Midspan loads on every third rafter are shown; continuous loads and loading to every second rafter were also assessed.

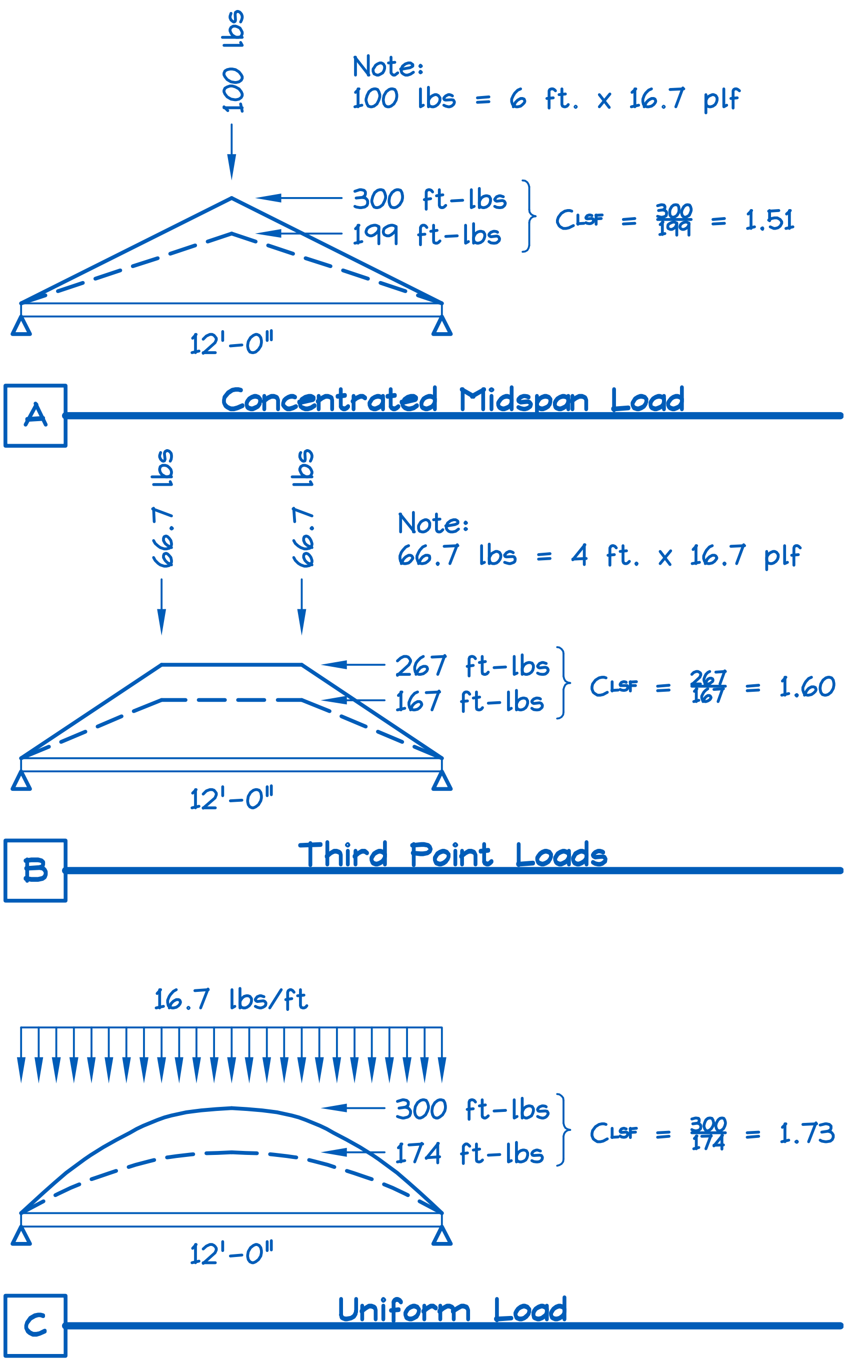


Figure D.6.4. Comparison of maximum moments with and without load-sharing effects from sheathing, for three loading patterns: midspan loading, third-point loading, and uniform loading. The Concentrated Load Sharing Factor, C_{LSF}, is the ratio of the maximum moment without load sharing to the maximum moment with load sharing. As the figure shows, the midspan loading generates the lowest C_{LSF} (1.51 in this case). To be conservative, C_{LSF} based on the midspan loading case was used in the subsequent analysis. Note that uniform loading has a C_{LSF} that is 15% greater than midspan loading.

The Concentrated Load Sharing Factors determined from the RISA-3D analysis vary slightly according to modeling idealizations for how the sheathing connects to rafters at panel butt joints, and to rafters between butt joints. Figure D.6.5 shows the idealized extreme assumptions at (1) panel butt joints (see subfigures 1A for the pinned idealization, and 1B for the fixed idealization), and at (2) plywood continuous over rafters (see subfigures 2A for pinned and 2B for fixed connection between sheathing and rafter). Panel butt joints are modeled in a staggered layout pattern ("case 1" illustrated in Building Code allowable diaphragm shear tables). Note that at both the butt joints and continuous sheathing over rafters, the question is whether the plywood can rotate independently of the rafter, forcing the nails to bend and withdraw, to allow the sheathing to rotate free of the rafter; or whether the nails effectively clamp the sheathing to the rafter. A real roof structure probably falls somewhere between these idealizations of pinned versus fixed. This analysis calculates load-sharing factors for the idealized cases, and takes the average.

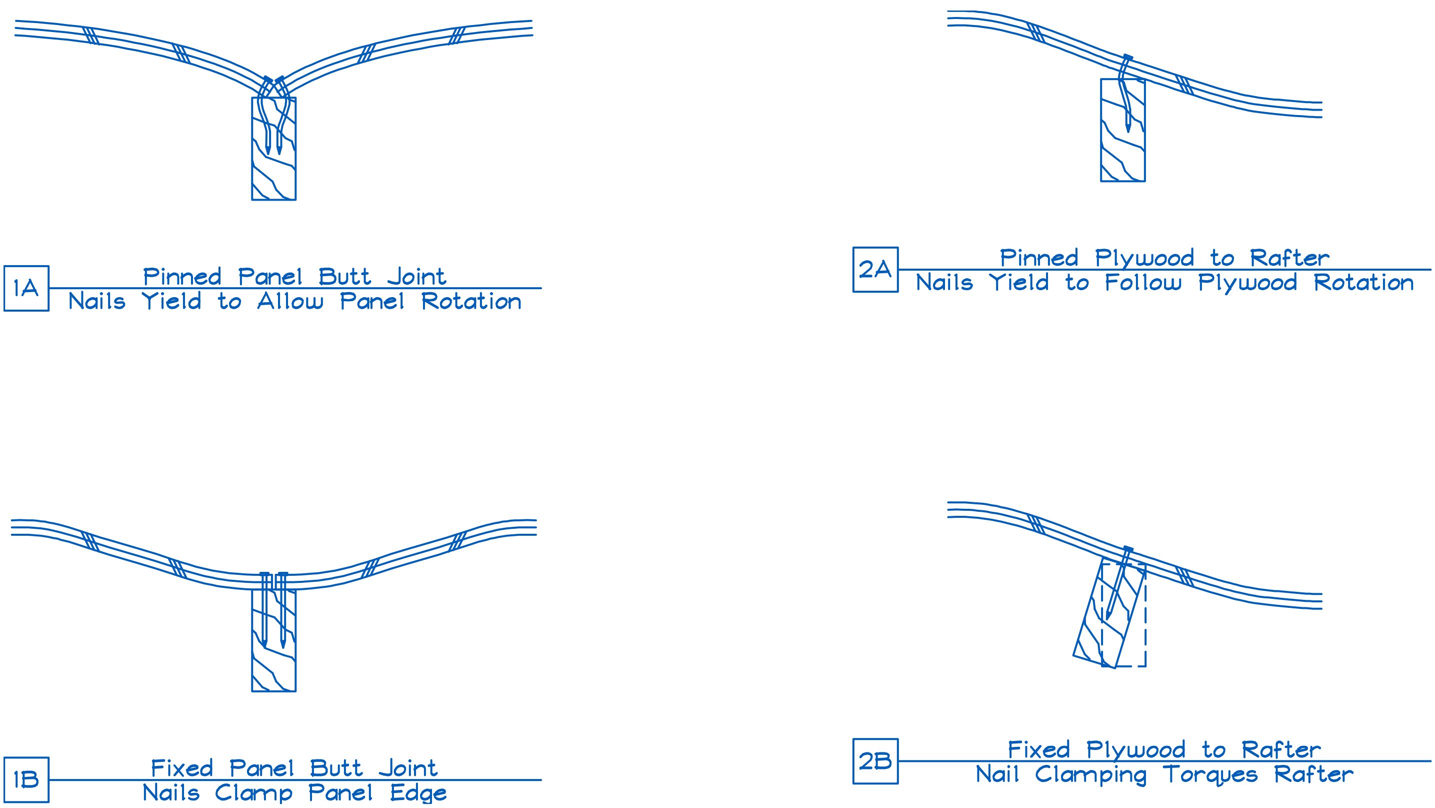


Figure D.6.5. Sheathing connection to rafter idealized as pinned or fixed at panel butt joints (1A versus 1B) and where sheathing runs continuously over a rafter (2A versus 2B). Real roof structural behavior lies somewhere between these idealized extremes.

The results of the analysis, based on examining a wide range of sheathing thicknesses, rafter sizes and spans, and sheathing-to-rafter fixity, are summarized in Table D.6.1, Table D.6.2, and Table D.6.3.

Table D.6.1. Concentrated Load Redistribution Factor from Sheathing

Anchor/Rafter Spacing, n	Rafter Spacing (in.)	Rafter Span[5] (ft-in)	Concentrated Load Sharing Factor, C_{LSF}								
			7/16" OSB [1]			1/2" nominal plywood [2]			5/8" nominal plywood [3,4]		
			2x4	2x6	2x8	2x4	2x6	2x8	2x4	2x6	2x8
2	16"	9'-10"	1.61	1.46	1.38	1.66	1.51	1.42	1.76	1.58	1.49
		14'-4"	1.75	1.58	1.49	1.82	1.64	1.54	1.92	1.73	1.62
		18'-2"	1.85	1.67	1.57	1.93	1.73	1.62	1.94	1.83	1.71
	24"	8'-0"	1.41	1.29	1.23	1.46	1.33	1.26	1.53	1.39	1.32
		11'-9"	1.53	1.39	1.32	1.58	1.44	1.36	1.67	1.51	1.42
		14'-10"	1.61	1.46	1.38	1.67	1.51	1.42	1.76	1.59	1.49
3	16"	9'-10"	1.91	1.54	1.34	2.05	1.66	1.45	2.27	1.86	1.63
		14'-4"	2.26	1.85	1.62	2.41	1.99	1.75	2.63	2.20	1.95
		18'-2"	2.48	2.06	1.81	2.64	2.2	1.95	2.66	2.42	2.16
	24"	8'-0"	1.42	1.16	1.06	1.54	1.24	1.11	1.72	1.38	1.21
		11'-9"	1.72	1.37	1.21	1.85	1.47	1.3	2.06	1.67	1.45
		14'-10"	1.92	1.54	1.35	2.06	1.67	1.45	2.27	1.86	1.63
4	16"	9'-10"	1.99	1.53	1.31	2.18	1.68	1.43	2.48	1.92	1.63
		14'-4"	2.46	1.91	1.62	2.67	2.09	1.78	2.99	2.38	2.04
		18'-2"	2.78	2.19	1.87	3.00	2.39	2.05	3.04	2.69	2.33
	24"	8'-0"	1.39	1.14	1.06	1.52	1.21	1.09	1.75	1.35	1.18
		11'-9"	1.74	1.34	1.18	1.92	1.47	1.27	2.19	1.68	1.43
		14'-10"	2.00	1.53	1.31	2.19	1.68	1.43	2.48	1.93	1.64

Legend:

Green shaded values (7/16" OSB) are the basis of the State Permitting Guidebook's Structural Toolkit Document's Table 1 (Anchor Maximum Horizontal Spacing).

Blue shaded values (15/32" plywood = 1/2" nominal) are the basis of the East Bay Green Corridor's Structural Check List's Table 2 (Maximum Horizontal Anchor Spacing).

Yellow shaded values indicate the effect of 19/32" (5/8" nominal) plywood or1x skip sheathing, with twice the sheathing stiffness as 1/2" nominal plywood.

Table Notes:

1. 7/16" thick OSB with 24/16 span rating and a minimum stiffness, EI = 78,000 lb-in^2/ft per 2012 AF&PA NDS Spec Table C9.2.3 and NDS Manual Table M9.2-1.

2. 15/32" and 1/2" thick plywood with 32/16 span rating and a minimum stiffness, EI = 125,000 lb-in^2/ft per 2012 AF&PA NDS Spec Table C9.2.3 and NDS Manual Table M9.2-1. Note: 15/32" OSB is slightly less stiff: EI = 115,000 lb-in^2/ft.

3. 19/32" and 5/8" thick plywood with 40/20 span rating and a minimum stiffness, EI = 250,000 lb-in^2/ft per 2012 AF&PA NDS Spec Table C9.2.3 and NDS Manual Table M9.2-1.

4. 1x sheathing typical stiffness, EI = (1,600,000)x(12x.75^3/12) = 675,000 lb-in^2/ft if solid sheathed, = approx 300,000 lb-in^2/ft if skip sheathed.

5. Rafter assumed to be Douglas Fir-Larch #2 with E = 1,600,000 psi per NDS Supplement Table 4A. Rafter span taken from 2013 CBC Table 1308.10.3(1) for Dead Load = 10 psf.

The Concentrated Load Sharing Factor (C_{LSF}) is a function of the relative stiffness of the sheathing and rafters, with thicker sheathing, tighter rafter spacing, shallower rafters, and longer rafters exhibiting a greater load-sharing effect. Deeper rafters have larger code-maximum spans, so relative stiffness and C_{LSF} values are quite similar for different size rafters at the same rafter spacing (16" or 24" o.c.). While C_{LSF} values for the DCR analysis are based on 2x6 rafters at the maximum Code allowed span, IBC Table A2.2 also shows that the analogous C_{LSF} values for 2x4 and 2x8 rafters are very similar. In IBC Table A2.2, these are the lightly tinted values diagonally adjacent the darker tinted values. Along the tinted diagonals, the C_{LSF} values reflect rafter spans with DCRs in the appropriate range of 0.90 to 1.00.

The values of the Concentrated Load Sharing Factor highlighted in the darker tints in IBC Table A2.2 correspond to roof structures with 7/16" oriented strand board (OSB), 15/32" (1/2" nominal) plywood, or 19/32" (5/8" nominal) plywood, with 2x6 DF #2 rafters at 16" on center spanning 14'-4", and rafters at 24" on center spanning 11'-9".

Using the NDS Manual Table M9.2-1, the plywood sheathing stiffness for 15/32” thick panels was assumed to be 125,000 lb-in^2/ft. This corresponds to a span rating of 32/16, the "Predominant" span rating for 15/32" sheathing in NDS Table C9.2.3. The stiffness listed in the NDS Manual Table M9.2-1 is described as a "minimum" value, with average values being higher. The sheathing stiffness also disregards the added stiffness from roofing, blocking, and underside gypsum board ceilings. For these reasons, a stiffness of 125,000 lb- in^2/ft for plywood is assumed to also apply to 15/32" oriented strand board (OSB), with a minimum stiffness of 115,000 lb-in^2/ft.

Note that 1x sheathing is significantly stiffer than either 1/2" or 5/8" plywood (see ASCE Table A2.2 Note 4), even if skip sheathing is used with a 50% coverage (675,000 lb-in^2/ft) / 2 = 338,000 lb-in^2/ft). Therefore, 1x skip sheathing is expected to have greater concentrated load-sharing effects than that assumed in the DCR analysis.

A poll of several lumber suppliers in central and northern California suggests that while 15/32” plywood or OSB has been, and remains, the predominant residential roof sheathing material, about 30% of tract home developments in the Central Valley may use 7/16” plywood or OSB. For this reason, Table D.6.1below and Table D.6.2 include both 7/16” OSB and 15/32” sheathing thicknesses and their associated stiffnesses.

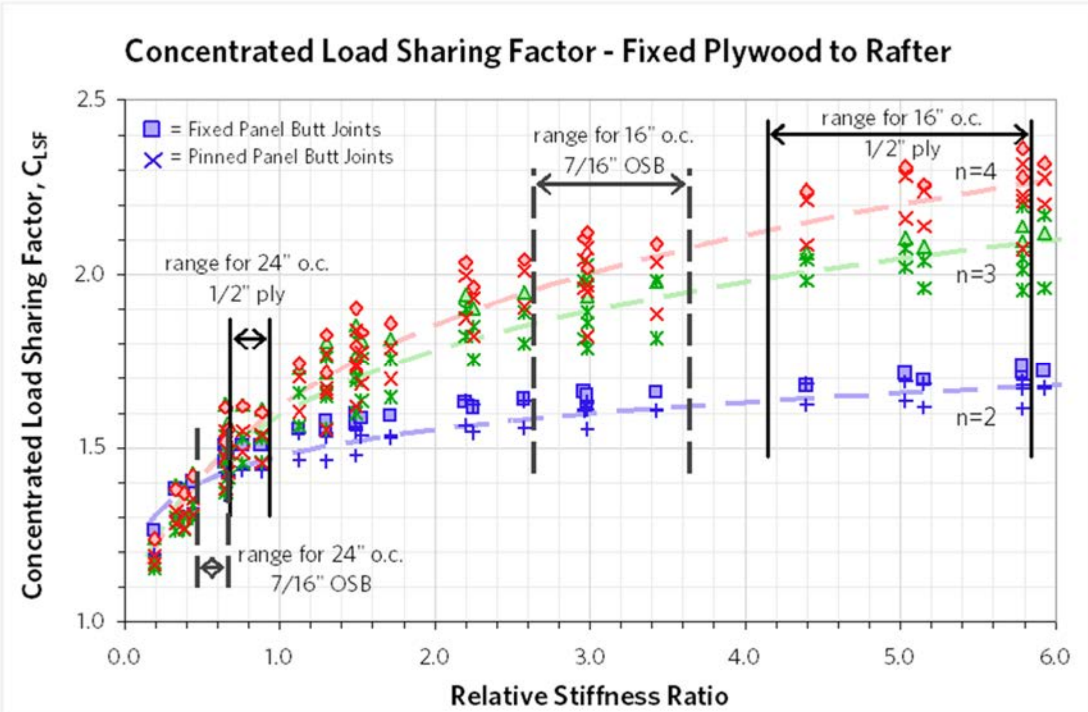
Table D.6.2. Sheathing-to-Rafter Relative Stiffness for Typical Scenarios									
Rafter E (DF #2)(1) = 1,600,000 psi									
Sheathing EI(2) = 15/32" = 125,000 lb-in^2/ft Plywood (1/2" nominal); 7/16" = 78,000 lb-in^2/ft OSB									
Member	I (in^4)	Rafter Spacing (in.)	Max. Rafter Span(3) (ft-in)	(in.)	Rafter EI/L^3 (lb/in)	Sheathing EI/L^3 (lb/in)		Sheathing / Rafter Stiffness Ratio	
						15/32"	7/16"	15/32"	7/16"
2x4	5.36	16" o.c.	9'-10"	118	5.22	30.5	19.0	5.85	3.65
		24" o.c.	8'-0"	96	9.69	9.04	5.64	0.93	0.58
2x6	20.8	16" o.c.	14'-4"	172	6.54	30.5	19.0	4.67	2.91
		24" o.c.	11'-9"	141	11.9	9.04	5.64	0.76	0.47
2x8	47.63	16" o.c.	18'-2"	218	7.36	30.5	19.0	4.15	2.59
		24" o.c.	14'-10"	178	13.5	9.04	5.64	0.67	0.42

Table Notes:

1. Per NDS Supplement Table 4A.
2. Per NDS Manual Table M9-2.1.
3. Per 2013 CBC Table 1308.10.3(1) for Dead Load = 10 psf.

The Concentrated Load Sharing (Redistribution) Factor, C_{LSF}, is a function of the sheathing-to-rafter stiffness ratio. Table D.6.2 shows the sheathing-to-rafter relative stiffness for typical scenarios. For 15/32" plywood, the nondimensional sheathing-to-rafterstiffness ratio ranges from 0.67 to 0.93 for rafters at 24" o.c., and from 4.15 to 5.85 for rafters at 16" o.c.. For 7/16" OSB, the nondimensional sheathing-to-rafter stiffness ratio ranges from 0.42 to 0.58 for rafters at 24" o.c., and from 2.59 to 3.65 for rafters at 16" o.c..

Figure D.6.6, Figure D.6.7, and Figure D.6.8 plot the Concentrated Load Sharing (redistribution) Factor across different ranges of sheathing-to-rafter stiffness ratios (relative stiffness) for anchor-to-rafter spacings n=2, 3 & 4. The sheathing and rafter stiffnesses are proportional to EI/L^3, where L = rafter spacing for calculating sheathing stiffness (sheathing EI/L^3), and L = rafter span for calculating rafter stiffness (rafter EI/L^3).



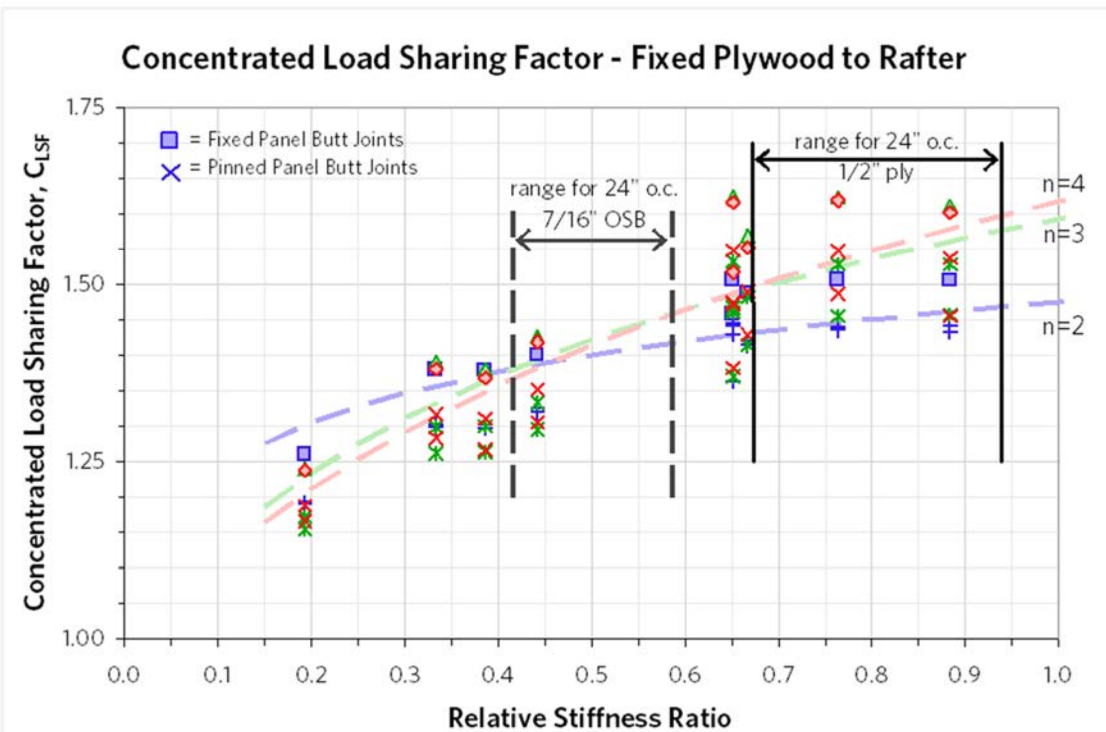


Figure D.6.6. Concentrated Load Sharing Factor as a Function of Sheathing-to-Rafter Relative Stiffness, assuming sheathing-to-rafter field connections are fixed. The upper graph shows the stiffness range of interest for rafters at 16" on center, while the lower graph shows the range for rafters at 24" on center.

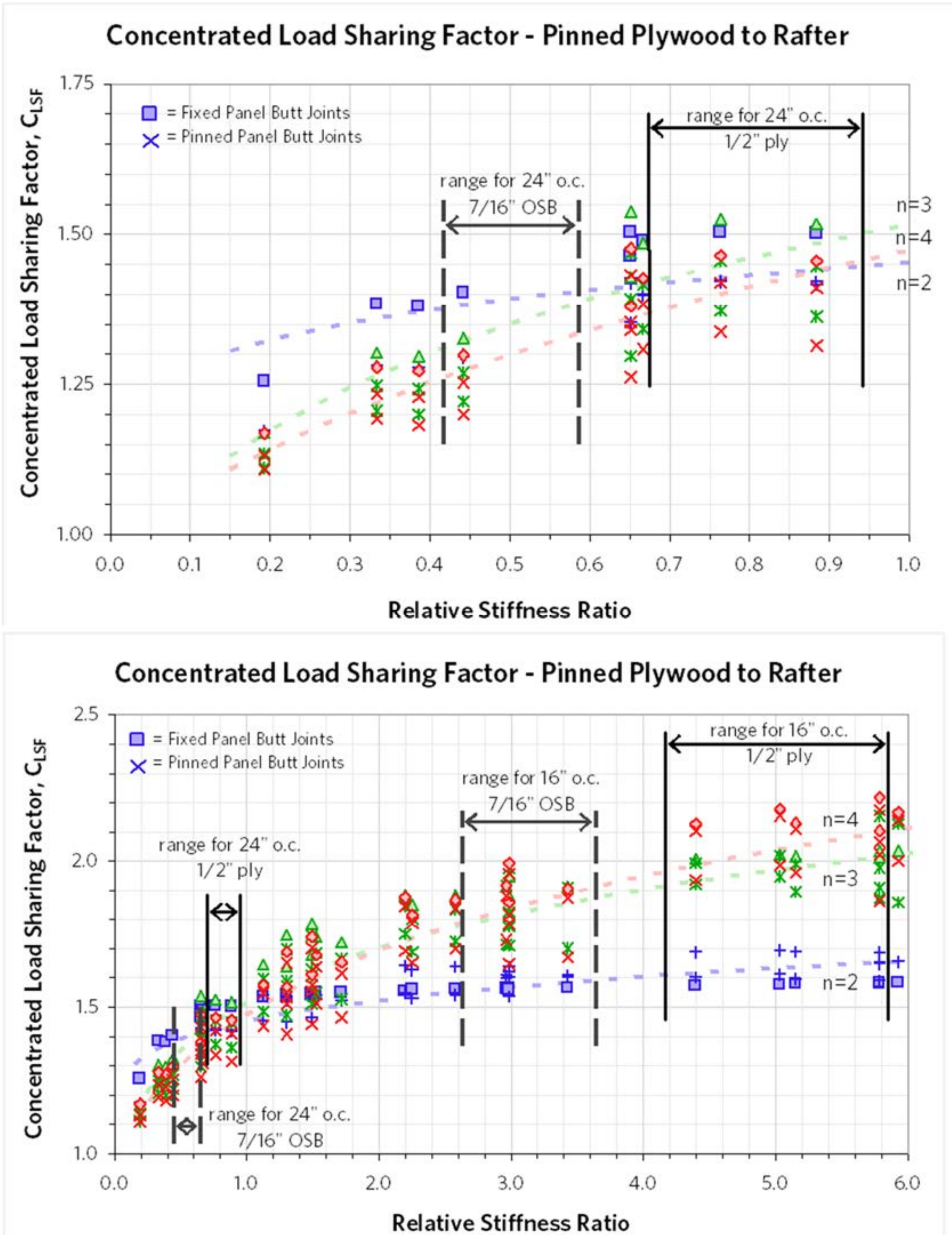
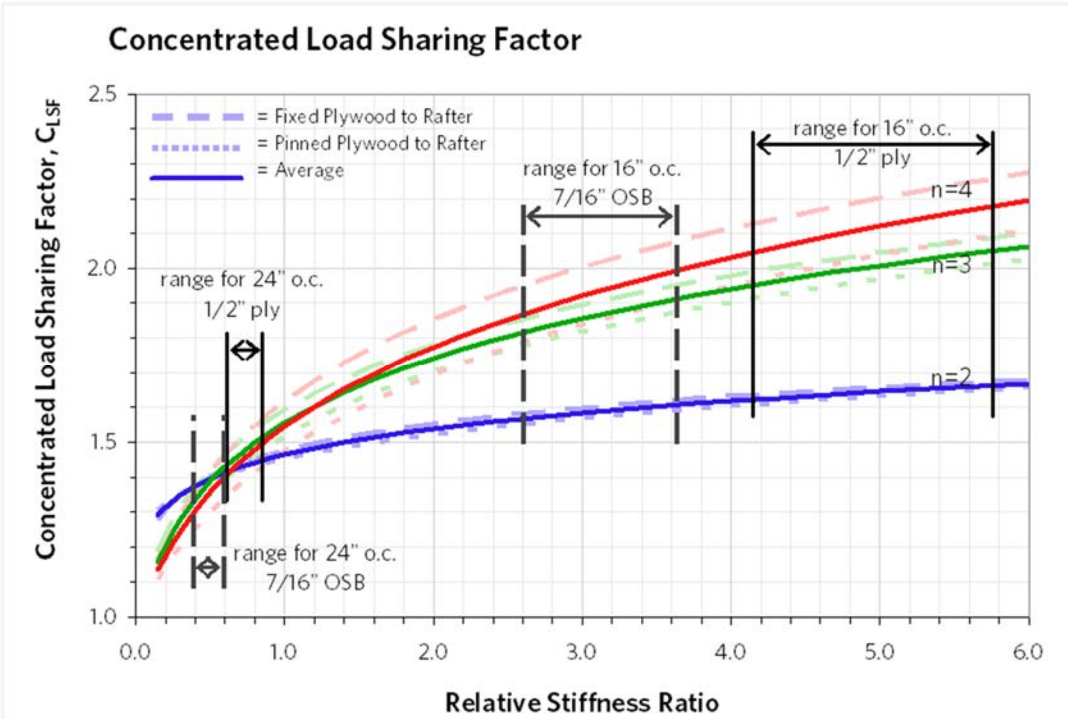


Figure D.6.7. Concentrated Load Sharing Factor as a Function of Sheathing-to-Rafter Relative Stiffness, assuming sheathing-to-rafter field connections are pinned. The upper graph shows the stiffness range of interest for rafters at 16" on center, while the lower graph shows the range for rafters at 24" on center.



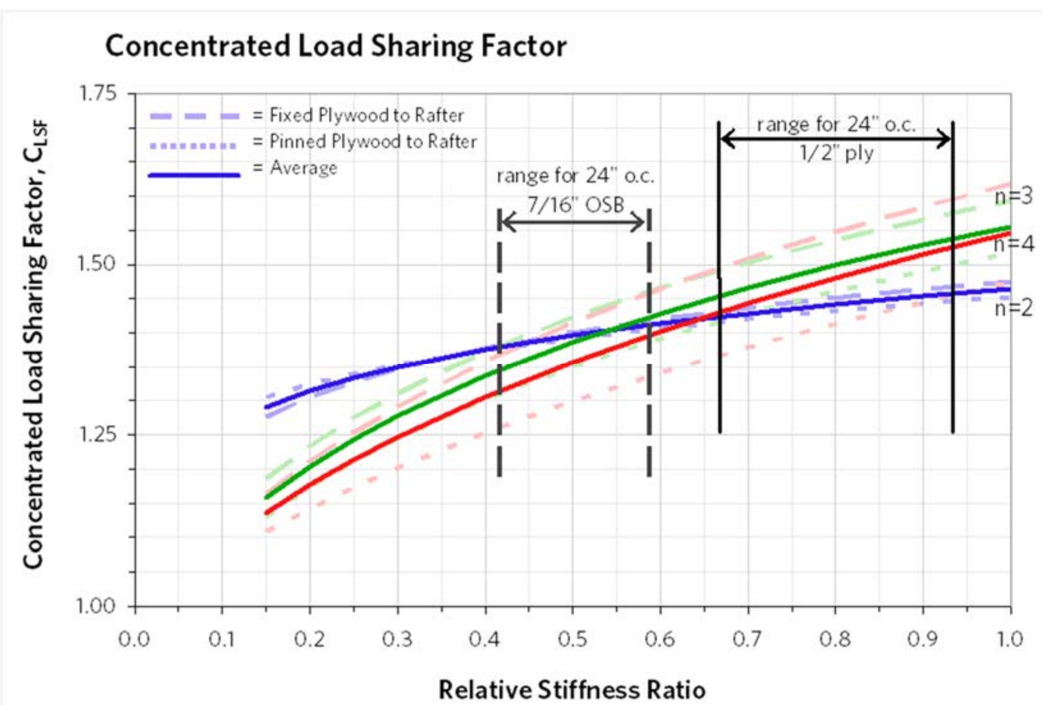


Figure D.6.8. Concentrated Load Sharing Factors taken as the average of the plywood fixed-to-rafter and pinned-to-rafter idealized extremes. The upper graph shows the stiffness range of interest for rafters at 16" on center, while the lower graph shows the range for rafters at 24" on center.

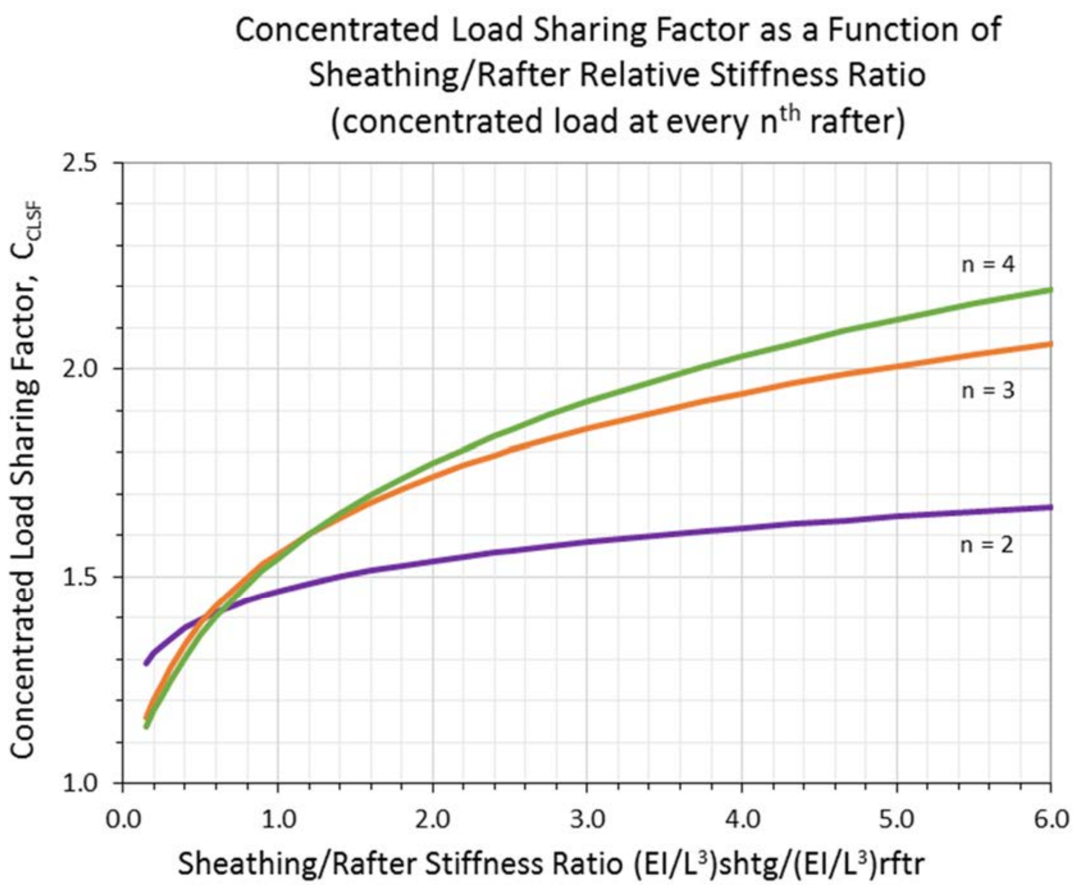


Figure D.6.9. Concentrated Load Sharing Factors as a function of the sheathing-to-rafter relative stiffness ratio, averaging the fixed-vs.-pinned edge assumptions of Figure D.6.6, Figure D.6.7, and Figure D.6.8.

Further Refinements to C_{LSF}

The following are potential future refinements to the Concentrated Load Sharing Factor (C_{LSF}) models. It is expected that such refinements would largely cancel each other out; the assumed load-sharing factors might shift slightly, but the concluding tables would probably remain essentially unchanged. These refinements are:

- Model rafter stiffness as a T-section considering composite action with sheathing. Since this would increase the relative stiffness of the rafters, the current model is slightly unconservative in this regard.
- Increase sheathing stiffness from roofing. Roofing's contribution to sheathing stiffness has not been quantified. Rafter blocking and gypsum board applied to the underside rafters are assumed to be absent, so their potential stiffening effects are also ignored. Because consideration of roofing would increase the relative stiffness of the sheathing, the current model is slightly conservative in this regard.

- Assume that the solar arrays impose more distributed load patterns over the rafter (e.g. third point or quarter point loads) rather than midspan loading. As shown in Figure D.6.4, the current model is conservative in this regard.
- Incorporate partial composite action. As discussed in the next section, composite action is ignored, even though its effect is potentially large, of the order of a 35% increase in capacity (Campos Varela, 2013).
- Incorporate statistical deviation from a constant stiffness/strength ratio. The model currently assumes stiffness (modulus of elasticity) and bending strength (modulus of rupture) have a linear proportional relationship (ref. Green and Kretschmann, 1991). In reality, stiffness and strength are not perfectly correlated (per comm. Brad Douglas, American Wood Council, 9/18/2014; ref. Kretschmann and Bendtsen, 1992). Concentrated loads on a stiff weak rafter will result in a lower effective C_{LSF}, while concentrated loads on a soft strong rafter will underestimate the actual C_{LSF}.

Distinction Between Concentrated Load Sharing Factor and Repetitive Member Factor

The Concentrated Load Sharing Factor is different from the repetitive member factor, C_r. The 2012 NDS Commentary (C4.3.9) states:

> *[Cr] reflects two interactions: [1] [statistical] load-sharing or [statistical] redistribution of load among framing members and [2] partial composite action of the framing member and the covering material. Application of the Cr adjustment requires no assumption as to which of the two types of interaction is involved or predominates. A Cr value of 15 percent is generally considered to be conservative for sawn lumber assemblies.*

In other words, the repetitive member factor is based on two effects:

- Composite action between sheathing and rafters, creating a stronger effective T-section, and
- Averaging of strength variation between three rafters rather than a single rafter, resulting in a statistically predicted higher average strength.

Structural wood experts often use the term "load sharing" to describe the *statistical* load-sharing (strength averaging) effect incorporated in C_r, while describing the Concentrated Load Sharing Factor (C_{LSF}) as "load *redistribution*". To these experts, the Concentrated Load Sharing Factor would be more accurately termed the Concentrated Load *Redistribution* Factor. Future editions of the Structural Commentary may incorporate this nomenclature.

The statistical load-sharing (strength averaging) effect deserves additional explanation. The breaking stress of a single rafter has a standard deviation σ around a mean μ, and an allowable stress at the 5% lower bound tail that is the mean minus 1.645 standard deviations, divided by the factor of safety. For the average of three members, the standard error around the mean is the standard deviation divided by the square root of three. Therefore, the statistically expected strength of three members, compared to one member, is:

$$\frac{\mu - 1.645\sigma / \sqrt{3}}{\mu - 1.645\sigma}$$

ASTM D245 indicates that F_b has a standard deviation of about 570 psi for Douglas Fir. Since for DF-L No. 2:

$$Fb = 900\,psi = [\mu - 1.645(570)]/2.16 \therefore \mu = 2{,}882\,psi$$

For the specific values for Douglas Fir-Larch No. 2, the calculated repetitive member factor based on statistical considerations alone is:

$$Cr = \frac{2{,}882 - 1.645(570)/\sqrt{3}}{2{,}882 - 1.645(570)} = 1.20$$

For Doug-Fir No. 2, the statistical C_r repetitive member factor of 1.15 is conservative without even taking into consideration *any* composite strength action. In general, the higher the design bending strength, the lower the statistical C_r. For instance, DF No 1 & Better, with F_b = 1200, yields C_r = 1.15, while DF No 3, with F_b = 525, yields C_r = 1.35. For non-composite floor assemblies, Rosowsky's Monte Carlo simulations suggest statistical C_r = 1.25 to 1.46.

The 2012 NDS commentary provides further clues about statistical C_r by pointing out that *the repetitive member increase also applies to an assembly of three or more essentially parallel members of equal size* [that*] are in direct contact with each other* [and fastened to each other]. Such an effect would be almost entirely statistical, that is, the standard deviation of three members is tighter (1/sqrt(3)) about the mean than one member, so the lower bound strength (5% tail) is higher.

Based on testing, partial composite action C_r is probably in the 1.25 to 1.40 range for lightly nailed roofs. Campos Varela's full scale tests suggest composite C_r = 1.35. Note that standard structural calculations of effective composite section modulus show that full composite action is in the range of 1.7 to 2.1, and is a function of sheathing thickness and rafter size and spacing; actual composite action is partial rather than full because of the limited capacity of typical sheathing field nailing.

The C_{LSF} is a concentrated load-sharing effect that is distinctly different from either C_r statistical load-sharing effects (more rafters sharing load increases expected lower bound strength) or Cr partial composite action effects. C_{LSF} is based solely on the fact that when you push down on one rafter on a sheathed roof, that rafter does not resist the full load because deformation compatibility causes adjacent rafters pick up part of the load. Fezio (p. 59 of text, p. 72 of pdf) reports that in their FEA analysis, a single rafter in a floor of ten unloaded rafters, under a single concentrated load, only resists 30% of the concentrated load applied directly to it, with adjacent rafters carrying 70%. This is a C_{LSF} of 1/0.30 = 3.33. Note that the Commentary's C_{LSF} calculation is more conservative. Based on the relative stiffness (EI/L^3 ratio) of sheathing and joists used in Fezio's simulation, the Commentary would have predicted C_{LSF} = 2.10, not 3.33.

Calculating Demand-Capacity Ratios (DCRs)

Taking advantage of the displaced roof live load, Demand-Capacity Ratios can be calculated for roof rafters supporting solar arrays with different anchor spacings, considering a solar array's concentrated dead load and wind download effects.

Using Allowable Stress Design (ASD), the Demand-Capacity Ratio for roof rafters, if controlled by bending strength (the typical case), can be expressed as:

$$DCR = \frac{M_{demand}}{M_{capacity}} = \frac{wL^2/8}{C_D F_b S}$$

where:

$w =$ load per unit length (normal to rafter) = $p \cdot s$, where *p* = loading pressure and *s* = tributary width

$L =$ rafter span length (along slope)

$C_D =$ load duration factor per NDS

$F_b =$ rafter allowable bending stress, given its species, grade and size, including all relevant modification terms other than C_D

$S =$ rafter section modulus, $bh^2/6$

Note that s, L, F_b and S are constant for a given roof geometry and rafter type. Thus, for a given structure and loading, the rafter DCR is proportional to the load demand modified for load duration:

$$DCR \propto \frac{w}{C_D} = D$$

If the original rafter was designed to a DCR = 1.0, then the DCR of the rafter supporting a solar array can be calculated as:

$$DCR = \frac{D_{with PV}}{D_{without PV}}$$

where:

$$D_{withPV} = \max\left(D_{PV+DL}, D_{PV+wind\_down+DL}, D_{wind\_up-PV-DL}\right)$$

is the maximum load demand from applicable load combinations on the roof rafter after installation of the solar array, and

$$D_{withoutPV} = \max\left(D_{DL+LLr}, D_{DL+wind\_down}, D_{DL+LLr+wind\_down}, D_{wind\_up-DL}\right)$$

is the maximum load demand from applicable load combinations on the roof rafter before installation of the solar array. The load demands on a roof rafter supporting a solar array are defined as:

$$D_{PV+DL} = \frac{(n/C_{LSF})\cdot\cos\theta\cdot DL_{PV} + \cos\theta\cdot DL_{roof}}{C_{D,DL}}$$

$$D_{PV+wind\_down+DL} = \frac{(n/C_{LSF})(\cos\theta\cdot DL_{PV} + 0.6p_{wind\_down}) + \cos\theta\cdot DL_{roof}}{C_{D,wind}}$$

$$D_{wind\_up-PV-DL} = \frac{0.6\cdot\left((n/C_{LSF})(p_{wind\_up} - \cos\theta\cdot DL_{PV}) - \cos\theta\cdot DL_{roof}\right)}{C_{D,wind}}$$

and the load demands on a roof rafter before installation of a solar array are defined as:

$$D_{DL+LLr} = \frac{\cos\theta\cdot DL_{roof} + \cos^2\theta\cdot LL_{roof}}{C_{D,LLr}}$$

$$D_{DL+wind\_down} = \frac{\cos\theta\cdot DL_{roof} + 0.6p_{wind\_down}}{C_{D,wind}}$$

$$D_{DL+wind\_down+LL_r} = \frac{\cos\theta\cdot DL_{roof} + 0.75\cdot 0.6p_{wind\_down} + 0.75\cos^2\theta\cdot LL_{roof}}{C_{D,wind}}$$

$$D_{wind\_up-DL} = \frac{0.6\cdot(p_{wind\_up} - \cos\theta\cdot DL_{roof})}{C_{D,wind}}$$

where:

$n =$ anchor spacing/rafter spacing

$C_{LSF} =$ Concentrated Load Sharing Factor

$\theta =$ roof slope where 0° = flat

$DL_{PV} =$ dead load of solar array (3.5 psf for photovoltaic arrays, 5 psf for solar-thermal arrays)

$DL_{roof} =$ dead load of roof (10 psf for typical wood-framed roof with composition shingles)

$LL_{roof} =$ roof live load (12 to 20 psf, depending on roof slope, per UBC 97 and CBC 2001 and earlier editions)

$p_{wind\_down} =$ wind downward pressure per ASCE 7-10 Chapter 30 Part 1, $C_{pi} = 0$ (without 16 psf minimum)

$p_{wind\_up} =$ wind upward pressure per ASCE 7-10 Chapter 30 Part 1, $C_{pi} = 0$ (without 16 psf minimum)

$C_L =$ beam stability factor (assumed to be 0.80)

$C_{D,DL} =$ load duration factor for dead load = 0.90

$C_{D,LL_r} =$ load duration factor for roof live load = 1.25

$C_{D,wind} =$ load duration factor for wind = 1.60

For wind upward load combinations, where the bottom of rafter is in compression, a beam stability factor of 0.80 is assumed. This takes into account modest torsional restraint and stiffness from three potential effects: roof sheathing is clamped by sheathing nailing to the top of the rafter, creating torsion stiffness; solar mounting components also brace the rafter against torsional buckling through clamping action; and rafters are sometimes sheathed on the interior side, bracing the bottom of the rafter directly against torsional buckling.

Additional Reserve Strength

The DCRs calculated above are multiplied by 0.90 to account for the following effects:

2013 CBC Chapter 34 "Existing Structures" allows increases in design gravity loads of up to 5 percent (article 3403.3.) without recalculation or re-evaluation.

Modules do not cover the entire slope from eave to ridge. The State Fire Marshall requirement of a three feet or greater set back from the ridge results in bending moments that are 88% for a 12 foot span, and 92% for a 15 feet span compared to a rafter fully and uniformly loaded from roof to ridge.

Discrete incremental rafter sizes (2x4, 2x6 etc.) and spans (16" vs. 24") make it unlikely that a roof framing design will precisely match the most efficient DCR of 1.00. In fact, as Table D.6.3 shows, the average DCR increment between rafter nominal sizes with 16" o.c. and 24" o.c. rafter-spacing options is 0.72. If we assume roof designs are equally distributed between DCR = 0.72 and 1.00, then 50% of the time the expected DCR will be 0.86 or less, and 90% of the time the expected DCR from this effect will be 0.97 or less.

Combining the last two effects suggests that the mean expected DCR is (.88)(.86) = 0.76 where 50% of DCRs are expected to be higher and 50% lower; and the 90% DCR is (.92)(.97) = 0.89 where 90% of DCRs are expected to be lower and 10% higher, showing that the 0.90 multiplier is a reasonable and conservative assumption, even without taking into consideration the existing Building Code's allowance that calculated DCR may be less than 1.05 instead of 1.00. This shifts the crossing point where DCR=1.00 to slightly steeper roof slopes.

Table D.6.3. Rafter Design Strength Steps[1,2]				
Rafter	Depth (in.)	Spacing (in.)	Strength Index	Incremental Relative Strength
2x4	3.5	24 16	0.51 0.77	0.67 0.61
2x6	5.5	24 16	1.26 1.89	0.67 0.86
2x8	7.25	24 16	2.19 3.29	0.67 0.92
2x10	9.25	24 16	3.57 5.35	0.67 -
			Avg:	0.72

Table Notes:

1. Strength Index = $(d^2)/s$ where d = rafter depth and s = rafter spacing

2. Incremental Relative Strength = strength index at row i divided by strength index at row i+1

The Transition from Orthogonal to Staggered Mount Patterns

For an array with mounts that anchor to some rafters, and skip over (span over) other rafters, the loaded rafters will carry a tributary area greater than that for which the rafter was originally designed. Concentrating snow loads on a single rafter can overwhelm its capacity, even after taking into account live load offset, duration of loading, and other factors. A spreadsheet was developed to calculate Demand-Capacity Ratios as a function of roof slope. A Concentrated Load Sharing Factor of 1.44 was assumed for rafters at 24" on center, 1.99 for rafters at 16" on center. Snow loads were incrementally increased until Demand-Capacity Ratios (DCRs) approached and then exceeded 1.00. The graphs for these thresh-hold values are shown below for mounts at 48 inch spacing and rafters at 16" and 24" on center. When this ground snow load threshhold is passed, the mounts should be placed in a staggered pattern to create a quasi-uniform load, thereby avoiding concentrations of loads on some rafters while skipping others. The spreadsheet (and associated figures below) shows that this transition occurs at ground snow loads of 11 psf for rafters at 16" on center, and 12 psf for rafters at 24" on center. The resulting anchoring rule is simple: anchors at 48" on center shall be staggered when ground snow load exceeds 10 psf.

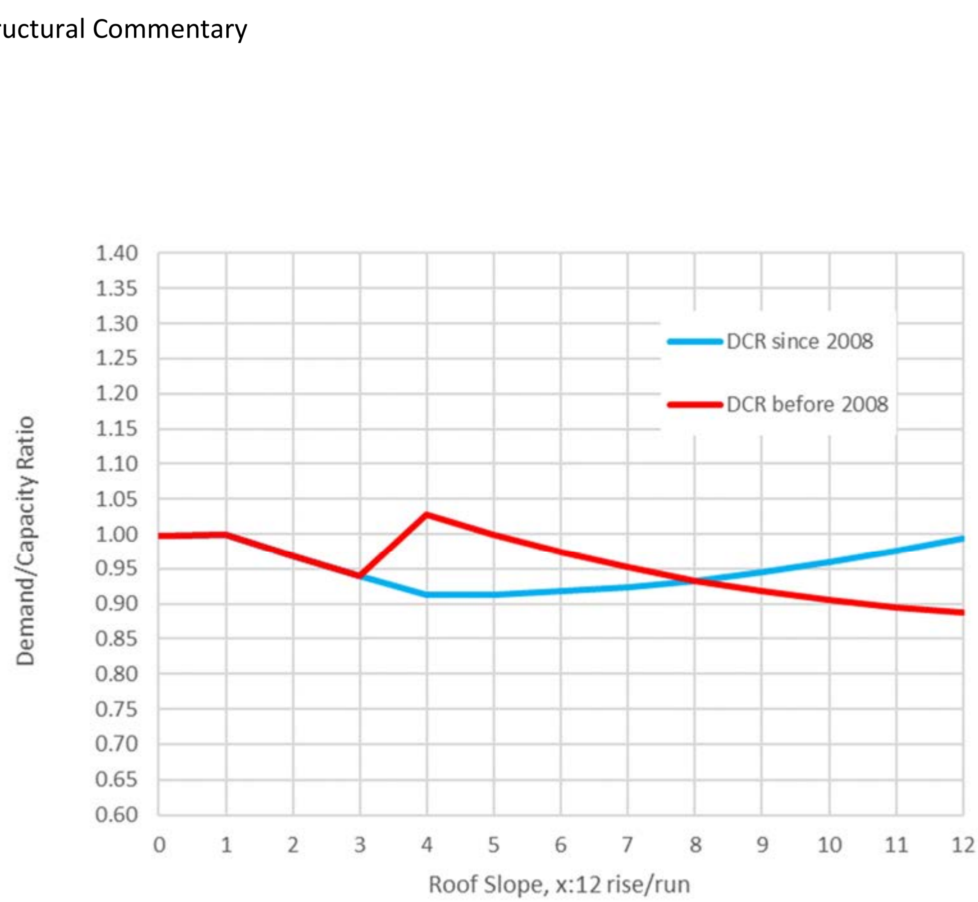


Figure D.6.10. Mounts at 48" o.c., rafters at 16" o.c., under a ground snow load of 11 psf, with mounts in an orthogonal layout (multiple mounts on every other rafter).

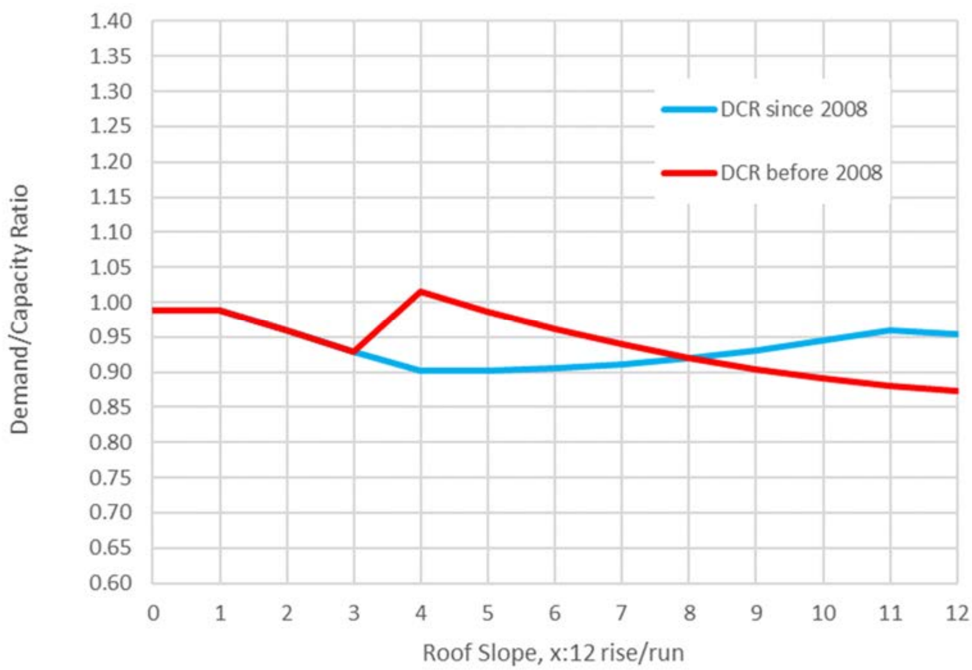


Figure D.6.11. Mounts at 48" o.c., rafters at 24" o.c., under a ground snow load of 12 psf, with mounts in an orthogonal layout (multiple mounts on every other rafter).

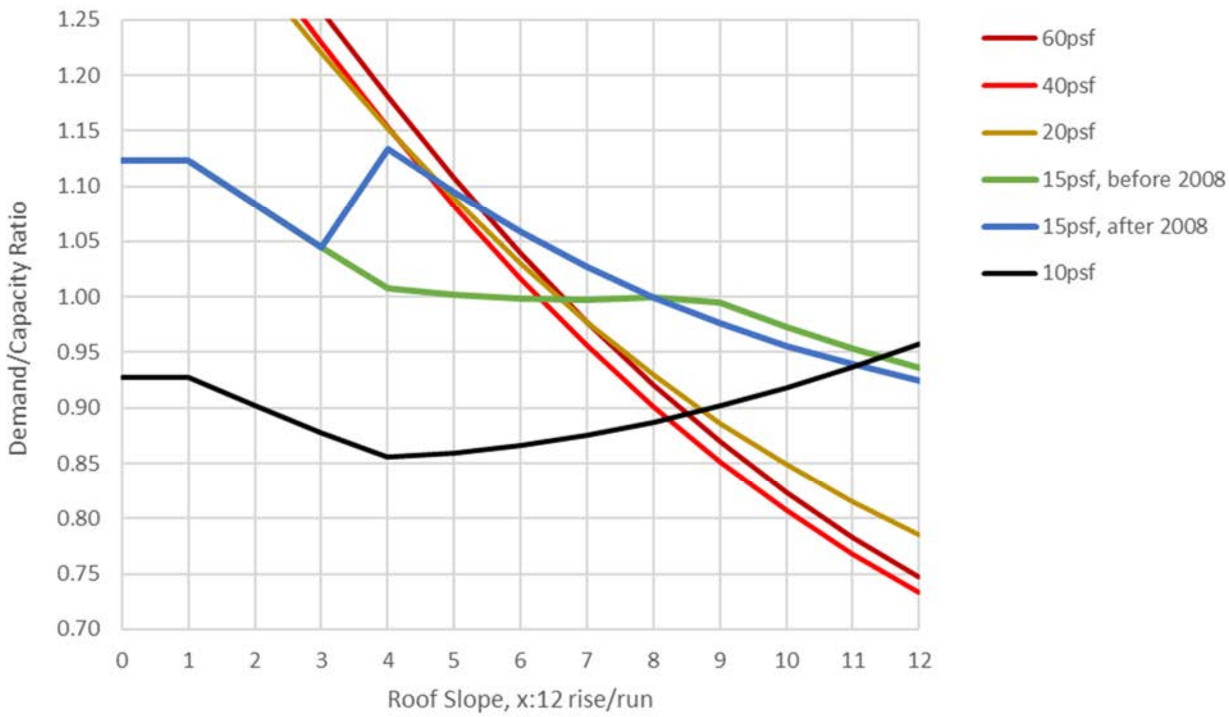


Figure D.6.12. Demand Capacity Ratios (DCRs) under various ground snow loads, for mounts at 48" o.c., rafters at 24" o.c., with mounts in an orthogonal layout (multiple mounts on every other rafter). Note that the DCRs for 10 psf are under 1.00, while the DCRs are over 1.00 for 15 psf and greater ground snow loads. This illustrates why mounts must be in a staggered layout rather than an orthogonal layout when ground snow loads exceed 10 to 12 psf.

The effect of concentrated snow loads, and when mount layout needs to shift from an orthogonal to a staggered pattern, is incorporated in the guideline rules restated below:

D.7. Upslope/downslope anchor spacing follows manufacturer's instructions.

The upslope/downslope anchor spacing does not have a big effect on the bending moment demands imposed on a given rafter. This is because anchors twice as heavily loaded spaced half as far apart will impose essentially the same moment demand as anchors half as heavily loaded spaced twice as far apart. Therefore, while the upslope/downslope anchor spacing is important to meet the support component manufacturer's objectives and recommendations, it does not have a big effect on the flexural demands of the rafters supporting the array. This is unlike the cross-slope anchor spacing, which does have a big effect on the distribution of flexural demands imposed on individual rafters or trusses.

D.8. Anchor fastener

☐ *D.8.a. 5/16" diameter lag screw with 2.5" embedment into structural member; or*

The ASD tensile withdrawal capacity of a 5/16" diameter lag screw embedded 2.5" into lower density Spruce-Pine-Fir lumber is (205 lbs/in)(2.5" – 3/16" tip length)(Cd = 1.6) = 758 lbs. If prying action from the foot configuration halves this value, the uplift capacity may be 379 lbs. For a rail-less system in landscape mode with feet every four feet, this amounts to an uplift demand of (25.7 psf)(40"x48"/144) = 343 lbs, a bit less than the uplift capacity.

Because withdrawal capacity is a function of lumber density taken to the 1.5 power, Douglas Fir (G=0.49) compared to Spruce-Pine-Fir (G=0.42) is $(0.49/0.42)^{1.5}$ = 1.26 times stronger, allowing the lag screw embedment to be 2 inches for Douglas Fir or Southern Pine (G=0.55).

☐ *D.8.b. fastener other than (a.), embedded in structural members in accordance with manufacturer's structural attachment details. Manufacturer's anchor layout requirements must not exceed the anchor spacing requirements shown in Items 5 and 6 above.*

E. Sheathing-Attached Array Requirements

E.1. Array is set back from all roof edges and ridge by at least twice the gap under the modules (or more, where fire access pathways are required).

See the previous discussion under section D.1.

E.2. Array does not cantilever over the perimeter anchors more than 19".

See the previous discussion under section D.1. Note that section E.6 includes tributary area limits. Those tributary areas shall include both half the backspan and any cantilever, so those tributary area provisions place additional limits cantilever lengths.

E.3. Gap under modules (roof surface to underside of module) is no greater than 5".

Wind tunnel research by Drs. Greg Kopp and Sarah Stenabaugh at the University of Western Ontario, Canada, demonstrates that solar arrays act as "air permeable cladding" (Stenabaugh et al, 2015, JWEIA). Arrays with sufficient gaps between modules (variable "g), and within a certain range of heights off the roof (variable "h) exhibit wind uplift pressures significantly less than conventional ASCE 7-10 pressures for solid roof surfaces. D.6.12, taken from Stenabaugh and annotated, shows that for arrays with a height 10" off the roof, with gaps of at least 0.25" between modules, a reduction factor of 0.70 can be justified. For arrays that are even lower, 5 inches off the roof with at least 0.75-inch gaps between modules, the wind uplift reduction factor can be even lower, around 0.50. This is the target configuration for sheathing-attached arrays, since controlling wind uplift is so important to good performance.

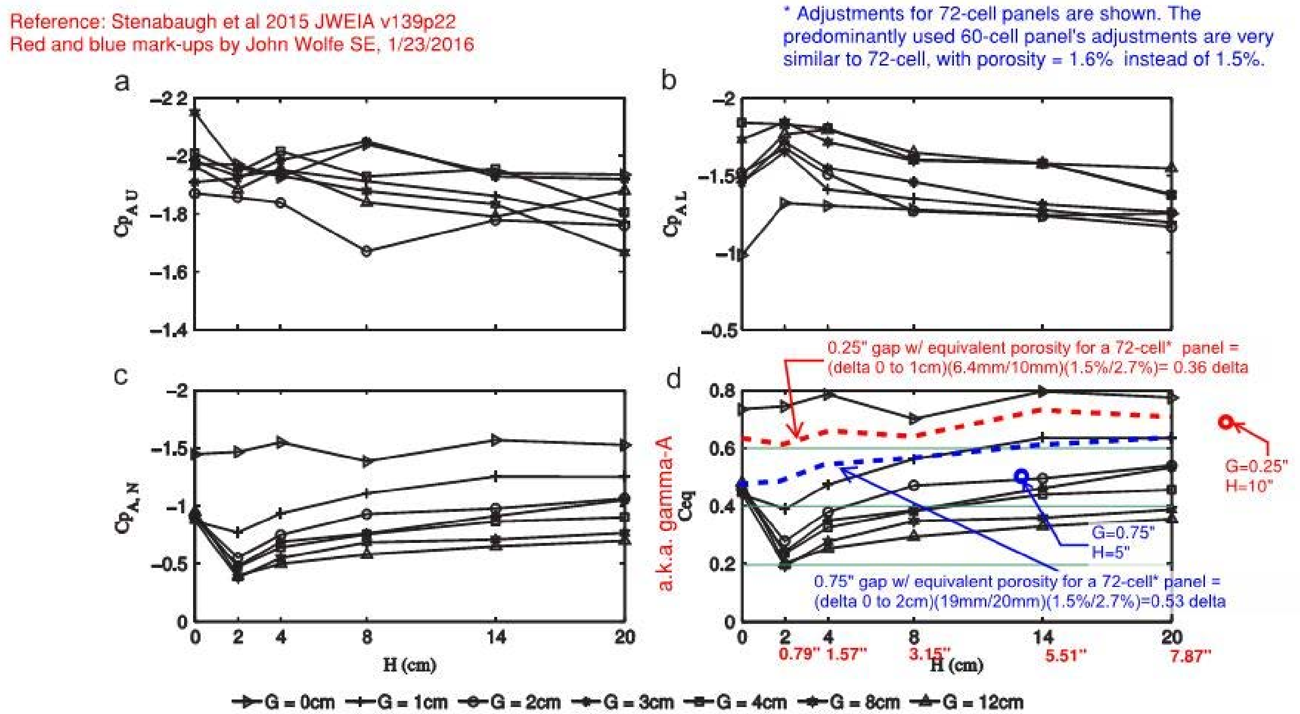


Fig. 7. The worst case values of the peak (suction) external on the (a) upper surface, (b) lower surface, and (c) net pressure coefficients measured, area-averaged over a single PV module and (d) the pressure equalization factor (*Ceq*) with respect to *H;* considering only the interior modules.

Figure D.6.12.

ASCE 7-10 Chapter 31, Article 31.4.3.2 places a lower bound of 65% on reduction factors related to components and cladding, so the reduction factor used in sheathing attached arrays to develop the tables of allowed installation regions is based on 0.65 instead of the even lower empirically-determined reduction factors shown in D.6.12. Because the 65% reduction factor is based on wind tunnel testing, resulting uplift pressures are allowed to drop below the standard code minimum uplift pressure of 16 psf (LRFD) or 10 psf (ASD).

The forthcoming ASCE 7-16 contains provisions for a wind uplift reduction factor of 0.80 for solar arrays with height off the roof less than 10 inches combined with gaps between modules of at least 0.25 inches. The Structural Engineers Association of California (SEAOC) Solar PV Committee has also endorsed a reduction factor of 0.60 for solar arrays with a height off the roof less than 5 inches combined with gaps between modules of at least 0.75 inches. This recommendation is currently in the draft of the forthcoming update to SEAOC PV 2.

E.4. Gap between modules is at least 0.75” on both short and long sides of modules.

See the discussion above under section E.3.

E.5. Roof slope is 2:12 (9 degrees) or greater.

Comparing ASCE 7-10 Figure 30.4-2A for gable roofs less than or equal to 7 degrees to Figure 30.4-2B, it can be seen that Zone 1, 2 and 3 uplift coefficients for the former (near-flat slope case) are somewhat greater than for the latter (low- to mid-slope case). 7 degrees corresponds to a 1.5:12 rise to run. In practical applications, many flashing products for mounts on composition-shingle roofs also require at least a 2:12 slope to address waterproofing concerns.

E.6. Roof Framing and Sheathing Nailing Options

- ☐ *E.6.a. Initially Dry Wood Rafters, or Manufactured Wood Trusses [lumber grade stamps visible and state "SD", "S-DRY" (Surfaced Dry) or "KD" (Kiln-Dried)]; or*
- ☐ *E.6.b. Initially Wet Wood Rafters, meeting one of the following field-verified sheathing nail options. (select i, ii, or iii below):*
  Note: If lumber stamps are not visible, or if lumber stamps state "S-GRN" (Surfaced Green), lumber shall be assumed to have been initially "wet" (MC > 19%) at time of sheathing installation
  - ☐ *i. Deformed shank nails, 6d or greater; or*
  - ☐ *ii. 6d smooth shank common or box nails, nailed into dense lumber, either Douglas Fir (stamp: DF or DF-L) or Southern Pine (stamp: SPIB).*

    *(**NOTE:** sheathing-attached arrays are not allowed with 6d smooth-shank nails and lower density lumber such as Spruce-Pine-Fir (stamp: S-P-F) and Hem-Fir (stamp: HF) .)*

Wet-to-Dry Nail Withdrawal Capacity Analysis

The 2015 NDS has a severe reduction factor of 0.25 for nails fastened to green lumber (moisture content exceeding 19%) that subsequently dries to indoor equilibrium moisture content (typically 8% to 11%). In many cases for existing roofs, the initial moisture content of the lumber is not known. The notable exceptions are where (1) the roof is framed with manufactured wood trusses, since manufactured trusses require dry lumber for quality control and consistent strength of their plated connections, and (2) where "S-Dry" (Surfaced Dry) or "K.D." (Kiln Dried) lumber stamps can be observed on the rafters in open attics.

Table E.6.1. Test Data and Means for Wet-to-Dry and Dry-to-Dry 24" o.c. Test Beds with 15/32" OSB Sheathing

Foot Position	**A_0**		**B_0**		**$A_{1.5}$**		**$B_{1.5}$**	
Initial MC	**Wet (lbs)**	**Dry (lbs)**	**Wet (lbs)**	**Dry (lbs)**	**Wet (lbs)**	**Dry (lbs)**	**Wet (lbs)**	**Dry (lbs)**
	676	941	658	1011	641	631	618	657
	620	884	643	854	586	809	561	1041
	565	1017	552	1108	660	875	599	689
	599	1017	750	920	471	933	584	742
	649	985	709	894	518	654	486	898

	512		625	1022	448	946	533	633
	574		612					
	579		562					
	607		515					
	643		689					
	533		499					
	532		405					
Mean	**591**	**969**	**602**	**968**	**554**	**808**	**564**	**777**
Initial Wet/Dry Ratio	**0.61**		**0.62**		**0.69**		**0.73**	
Average Wet/Dry Ratio	**0.66**							

Because of sheathing bending, the sheathing-to-rafter nails are not loaded in pure tension. Instead, the nails are cranked sideways, jamming the nails against the side of the nail hole and increasing friction between the nail and the loose holes created by drying shrinkage. Figure E.6.1 illustrates the deformation of sheathing nails when adjacent feet pull up on the sheathing.

Figure E.6.1. Sheathing in uplift exerts bending forces on sheathing nails, jamming the nails against the side of the nail hole and providing significant withdrawal resistance even if the hole is enlarged by drying shrinkage. This provides one possible explanation for why test beds built of green lumber appear to have uplift capacities very similar to test beds built of dry lumber, especially for the preferred Foot Positions A and B.

Individual nail pull-out tests conducted by SMASHsolar suggest that the severe NDS withdrawal factor of 0.25 is not only appropriate, but may even apply to lumber with initial moisture content as low as 16%. However, tests on full-scale test beds document significantly higher uplift capacities and a higher withdrawal factor of 0.66. Such full scale testing of the entire sheathing/nailing/rafter assembly under concentrated uplift loads is more accurate, and captures the effect where sheathing nails are under combined bending and withdrawal loads, instead of just pure withdrawal. Code compliance tables for wet-to-dry lumber can be based on the full-scale tests and the associated higher factor of 0.66.

The combination of initial moisture content (dry or wet), lumber density (soft SPF or medium DF-L), and nail size (6d common vs. 8d box) results in a wide range of relative uplift capacity, as shown in Table E.6.2.The difference between initially dry 8d sheathing nails into Douglas Fir versus initially wet 6d sheathing nails into Spruce-Pine-Fir is approximately three-fold (relative capacity product = 1.00 versus 0.35).

Table E.6.2: Sheathing Relative Uplift Capacity									
Moisture	**Density**	**Nail**	**Moisture**	**Density**	**Nail**	**Product**	**wet**	**soft**	**short**
dry	med.	8d	1.00	1.00	1.00	**1.00**			
dry	med.	6d	1.00	1.00	0.78	**0.78**			x
dry	soft	8d	1.00	0.68	1.00	**0.68**		x	
wet	med.	8d	0.66	1.00	1.00	**0.66**	x		
dry	soft	6d	1.00	0.68	0.78	**0.53**		x	x
wet	med.	6d	0.66	1.00	0.78	**0.51**	x		x
wet	soft	8d	0.66	0.68	1.00	**0.45**	x	x	
wet	soft	6d	0.66	0.68	0.78	**0.35**	x	x	x

Table E.6.2. Sheathing-Relative Uplift Capacity

E.7. Anchor Location Restrictions

All anchors must comply with at least one of the options below. Anchors verified to be in “bands of strength” are attached in the middle 16-inch-wide strip centered between the long edges of sheathing panels (i.e., at least 16” from sheathing long edges).

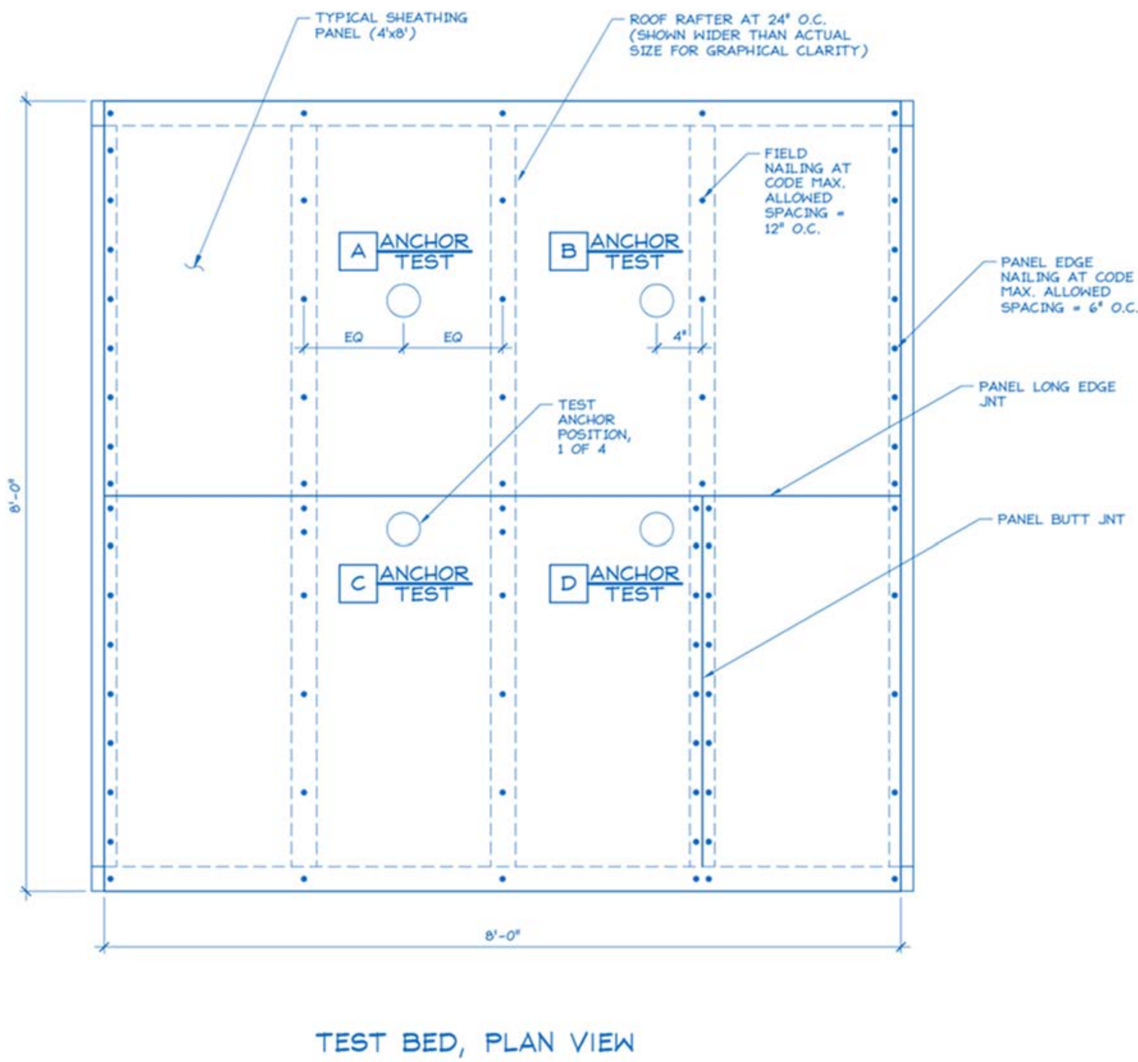


Figure E.7.1. Diagram of full size test bed. One of two positions (A or B) was tested on the upper sheathing panel, and one of two positions (C or D) was tested on the lower panel. Scores of tests were conducted, looking at variations in sheathing (OSB versus plywood), initially dry versus initially wet lumber, and rafter spacing (24" or 16" o.c.).

In residential construction, plywood or oriented strand board (OSB) panels are almost always laid up on a roof starting at the eaves, laying the long panel edge on the eave edge (end of rafters), and laying subsequent courses up the roof. In the National Design Specification (NDS) for wood construction, this sheathing panel layup pattern is called "Case 1". This creates a predictable pattern for the long edges of the panels: sheathing panels are typically installed starting at the eave, with the long edges parallel to the eave, and the bands of strength running parallel to the eaves. The centerline of the first band of strength typically occurs two feet upslope of the eave, with subsequent bands occurring every feet as one moves upslope toward the ridge.

Extensive testing by SMASHsolar found that the uplift capacity within “bands of strength” was twice as great as mounts located outside of these bands. The “bands of strength” are centered about the longitudinal midline of the sheathing panels, midway between the long edges of four feet wide sheathing panels. Figure E.7.3 shows the relative uplift capacity of feet as a function of the distant from the panel longitudinal centerline. Strength falls off rapidly more than 8” from the centerline of the band of strength. Another way of saying this is that the “bands of strength” occur at least sixteen inches away from the long edges of four feet wide sheathing panels.

From SMASHsolar testing, the mean maximum sheathing-to-rafters tested uplift capacity in the A and B positions is 790 lbs., using initially dry Douglas-Fir rafters at 24” on center, and either 15/32” plywood or oriented strand board (OSB).

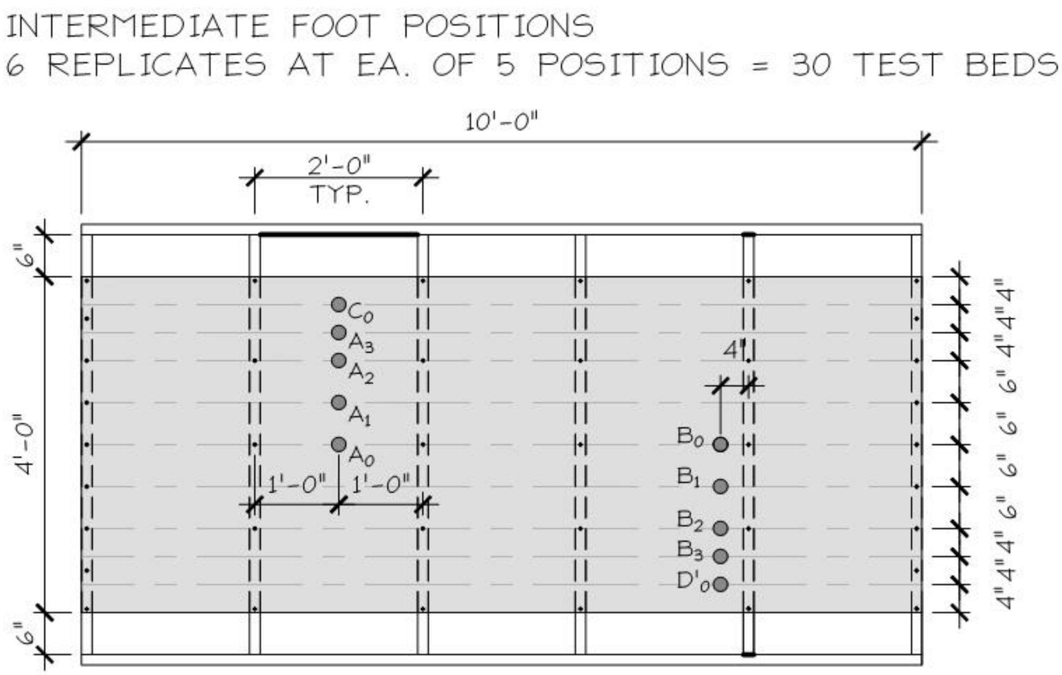


Figure E.7.2. Test protocol to determine the uplift load capacity of intermediate foot positions.

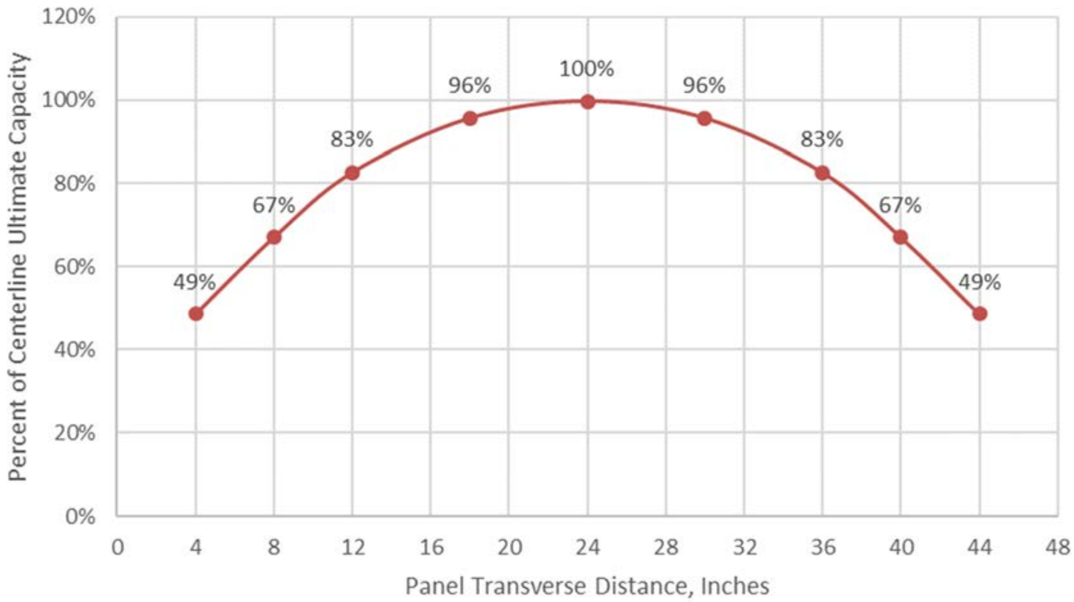


Figure E.7.3. Uplift capacity as a function of distance from longitudinal centerline of 4-foot-wide sheathing panels. Mounts located within 8" to each side of the panel long centerline are said to be "within the band of strength."

E.7.a. Some anchors are <u>not</u> within bands of strength.

All the restrictions below (i., ii. & iii.) apply to this case:

- ☐ *i. Edge of array is more than 3 feet from any roof edge (Wind Zone 1), and*
- ☐ *ii. Tributary area is 9 ft^2 or less (up to half the area of a 60 cell PV module),*
- ☐ *iii. Wind Exposure B only, and design wind speed does not exceed 120 mph.*

If mounts have limited tributary areas and are located within certain wind uplift zones, there are array layouts where mounts may be located outside of bands of strength, if the mount locations are limited to Zone 1 and have tributary areas less than one-half of a typical 60 cell PV module (9 sq.ft.). In addition, wind uplifts need to be restricted so that if the lumber is initially wet (a scenario that often cannot be ruled out), then the array is limited to Exposure B conditions and wind speeds no greater than 120mph. Arrays located on roofs with initially dry lumber have the standard sheathing-attached wind condition requirements: Exposure C, 120 mph, or Exposure B, 140mph.

Anchors located anywhere in Zone 1, regardless of bands of strength, have the two lower-bound conditions: (1) Exposure B, 120 mph, 6d sheathing nails fastened into initially dry, low-density Spruce-Pine-Fir (SPF) rafters; and (2) Exposure B, 120 mph, 6d sheathing nails fastened into initially wet, medium-density Douglas Fir rafters. It was shown in Section E.6 that the second case has a lower capacity, as shown in Table E.6.2.

Table E.7.a.1 shows the DCRs for arrays in this case.

Case (2) has the lower strength, and is therefore the controlling case.

In the table below, an off-center position factor of 0.49 is used to reduce the sheathing uplift capacity. Finally, it is important to note that this table is based on testing where the mounts apply only a vertical upward force to the sheathing, and not a bending moment. At least one manufacturer (SMASHsolar) has mounts that are designed to only impose uplift, and not bending moment, on the sheathing. To be accurate, other types of mounts should have an additional demand increase factor to account for prying imposed on the sheathing-to-rafters nails. To account for this, a capacity reduction factor of 0.8 was used in the following table.

Edge16 DF/SP 6d: Viable Regions of Installation, Generous Roof Edge Distances, Med. Density Lumber										
Roof Edge Distance:		Bottom Side: 16 inches only, Left & Right Sides: 16 inches minimum, Top Side: 36 inches minimum								
Roof Framing Lmbr:		Douglas Fir or Southern Pine (G=0.49 or higher)					Allowable Uplift Capacity:			64 lbs
Wind Speed		Wind Exposure								
		B			C			D		
ASCE 7-05 (equiv.)	ASCE 7-10 (exact)	Slope 0:12-1:12 (0° - 7°)	Slope 2:12-6:12 (8° - 27°)	Slope 7:12-12:12 (28° - 45°)	Slope 0:12-1:12 (0° - 7°)	Slope 2:12-6:12 (8° - 27°)	Slope 7:12-12:12 (28° - 45°)	Slope 0:12-1:12 (0° - 7°)	Slope 2:12-6:12 (8° - 27°)	Slope 7:12-12:12 (28° - 45°)
87	110	0.76	***0.67***	***0.81***	1.17	1.03	1.21	1.43	1.27	1.47
91	115	0.85	***0.75***	***0.90***	1.30	1.15	1.35	1.58	1.41	1.63
95	120	0.95	***0.84***	***1.00***	1.43	1.28	1.48	1.75	1.56	1.79
103	130	1.16	1.03	1.21	1.73	1.54	1.78	2.09	1.87	2.14
111	140	1.39	1.23	1.44	2.04	1.83	2.09	2.47	2.21	2.51
119	150	1.63	1.45	1.68	2.38	2.13	2.43	2.87	2.57	2.92
126	160	1.89	1.69	1.94	2.75	2.46	2.80	3.30	2.95	3.35
134	170	2.17	1.94	2.21	3.13	2.81	3.18	3.76	3.37	3.80

Note: Numbers in table are wind uplift demand/capacity ratios (DCRs).
Viable regions are shown in green. Installation is not permitted in red regions.

Modules:	39.5"x66"		Roof Mean Height (ft):	30	ft.
Orientation:	Portrait		Dead Load (psf):	3.00	psf
Roof Lumber:	Douglas Fir (G=0.49 or higher)		Demand Increase Factor:	1.00	
Kopp Factor:	0.65		Off-Center Factor:	0.49	
Tributary Area:	9	sq.ft.	Initially Wet Lmbr Reduction Fact	0.66	
Rafter Spacing:	24	inches on center	6d Nail Reduction Factor:	0.78	
Toggle Bolt:	1/4"	diameter	Prying Capacity Reduction Factor:	0.80	

Table E.7.a.1: Anchors not within bands of strength, for Case (2): 6d sheathing nails fastened into initially wet, Douglas Fir lumber. As the table shows, the DCRs for Exposure B, 120 mph design wind speed are 0.84 for 2:12 to 6:12 roof slopes, and 1.00 for 7:12 to 12:12 roof slopes. The off-center factor of 0.49 listed in the footnotes reflects the reduction in strength from not locating anchors within bands of strength.

E.7.b. All anchors are within bands of strength, and all the following (i, ii & iii) apply:

- ☐ *i. edge of array is more than 3 feet from any roof edge (Wind Zone 1), and*
- ☐ *ii. tributary area is 14 ft^2 or less (40"x48").*
- ☐ *iii. One of the two cases below (x. or y.) applies:*
  - ☐ x. *Exposure B, and design wind speed does not exceed 140 mph, or*
  - ☐ y. *Exposure C, and design wind speed does not exceed 120 mph.*

When anchors are located within bands of strength, and restricted to Zone 1 on the roof (at least 3 feet from all roof edges), then the standard sheathing-attached restrictions apply. The lower bound case is 6d sheathing nails fastened to initially wet, medium-density Douglas-Fir rafters. The off-position factor is 0.94 showing that anchors located within 8 inches of the panel centerline have twice the uplift capacity of anchors located near the long edges of panels.

Table E.7.b.1 shows the DCRs for arrays with mounts located within the bands of strength for the governing case of initially wet, Douglas Fir lumber (as discussed in Section E.7.a). This table uses a demand increase factor of 1.12 to conservatively reflect some unevenness of uplift on feet located within the bands of strength.

At least one manufacturer (SMASHsolar) has mounts that are designed to only impose uplift, and not bending moment, on the sheathing. To be accurate, other types of mounts should have an additional capacity reduction factor to account for prying imposed on the sheathing-to-rafters nails. The DCRs in Table E.7.b.1 do not include a reduction factor for prying, so mounts in this case must be designed to not impose bending moment loads on the sheathing.

Edge16 DF/SP 6d: Viable Regions of Installation, Generous Roof Edge Distances, Med. Density Lumber										
Roof Edge Distance:		**Bottom Side: 16 inches only, Left & Right Sides: 16 inches minimum, Top Side: 36 inches minimum**								
Roof Framing Lmbr:		**Douglas Fir or Southern Pine (G=0.49 or higher)**					**Allowable Uplift Capacity:**			**153 lbs**
Wind Speed		Wind Exposure								
		B			C			D		
ASCE 7-05 (equiv.)	ASCE 7-10 (exact)	Slope 0:12-1:12 (0° - 7°)	Slope 2:12-6:12 (8° - 27°)	Slope 7:12-12:12 (28° - 45°)	Slope 0:12-1:12 (0° - 7°)	Slope 2:12-6:12 (8° - 27°)	Slope 7:12-12:12 (28° - 45°)	Slope 0:12-1:12 (0° - 7°)	Slope 2:12-6:12 (8° - 27°)	Slope 7:12-12:12 (28° - 45°)
87	110	0.52	***0.45***	***0.54***	0.79	***0.70***	***0.81***	0.97	0.86	0.99
91	115	0.58	***0.51***	***0.60***	0.88	***0.78***	***0.90***	1.08	0.96	1.10
95	120	0.65	***0.57***	***0.67***	0.98	***0.87***	***1.00***	1.19	1.06	1.21
103	130	0.79	***0.70***	***0.81***	1.18	1.05	1.19	1.43	1.27	1.44
111	140	0.95	***0.84***	***0.96***	1.39	1.24	1.41	1.68	1.50	1.69
119	150	1.11	0.99	1.13	1.63	1.45	1.64	1.96	1.75	1.96
126	160	1.29	1.15	1.30	1.88	1.67	1.88	2.25	2.01	2.25
134	170	1.48	1.32	1.49	2.14	1.91	2.14	2.56	2.29	2.56

Note: Numbers in table are wind uplift demand/capacity ratios (DCRs).
Viable regions are shown in green. Installation is not permitted in red regions.

Modules:	39.5"x66"		Roof Mean Height (ft):	30	ft.
Orientation:	Portrait		Dead Load (psf):	3.00	psf
Roof Lumber:	Douglas Fir (G=0.49 or higher)		Demand Increase Factor:	1.12	
Kopp Factor:	0.65		Off-Center Factor:	0.94	
Tributary Area:	13.33	sq.ft.	Initially Wet Lmbr Reduction Fact	0.66	
Rafter Spacing:	24	inches on center	6d Nail Reduction Factor:	0.78	
Toggle Bolt:	1/4"	diameter	Prying Capacity Reduction Factor:	1.00	

Table E.7.b.1: Anchors within bands of strength, for 6d sheathing nails fastened into initially wet, Douglas Fir lumber, for modules within Roof Zone 1 (three feet from all roof edges).

As Table E.7.b.1 shows, the DCRs for Exposure B, 140 mph design wind speed are 0.84 for 2:12 to 6:12 roof slopes, and 0.96 for 7:12 to 12:12 roof slopes. The DCRs for Exposure C, 120 mph design wind speed are 0.87 for 2:12 to 6:12 roof slopes, and 1.00 for 7:12 to 12:12 roof slopes. The off-center factor of 0.94 listed in the footnotes reflects the small reduction in strength from locating anchors no more than 8" from the longitudinal centerline of the roof panels. The demand increase factor of 1.12 reflects the specific properties of the SMASHsolar system, but can be generalized to reflect either (a) a cantilever two or three inches more than the typical 19" cantilever limit, or (b) a standard cantilever of 19 inches or less, projecting a few inches into Zone 2 (i.e. a close as 24" away from the roof edge instead of the Zone 3 limit of 36").

The array shown in Figure E.7.4 is a special case where some of the feet are in bands of strength, while others are not. The array meets these restrictions of section E.7 because (a) all the rows of mounts except for the lowermost row are in Zone 1, and (b) the lowermost row mounts, while extending into Zone 2, are located in a "band of strength". Note that this array would be limited by the requirements of E.7.a., that is Exposure B, 120 mph design wind speed.

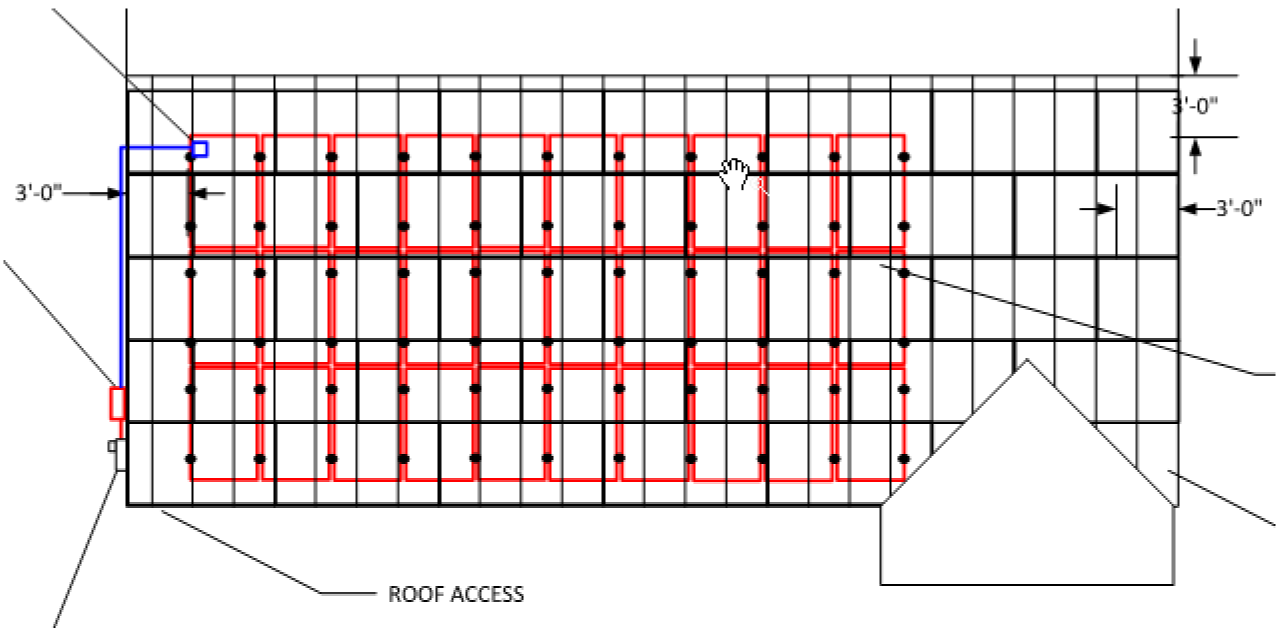


Figure E.7.4. An example of a mixed sheathing-attached array, where the lowest row of mounts is centered in the bands of strength, and the subsequent rows do not align with bands of strength. Assuming these are 60-cell panels, the lower row of mounts meets the edge distance and tributary area restrictions of rule E.7c, while the other rows meet the edge distance and tributary area restrictions of rule E.7a.

E.7.c. All anchors are within bands of strength, and all the following (i, ii and iii) apply:

- ☐ *i. Edge of array meets E.1 and is within 3 feet of a roof edge (Wind Zone 2), and*
- ☐ *ii. Tributary area including cantilevers is 9 ft^2 or less (32.5"X40").*
- ☐ *iii. Wind Exposure B only, and design wind speed does not exceed 120 mph.*

The lower row of mounts shown in Figure E.7.4 illustrate this case, where the modules project into roof wind Zone 2. Table E.7.c.1 shows the associated Demand/Capacity Ratios (DCRs).

Edge10 DF/SP 6d: Viable Regions of Installation, Minimum Roof Edge Distances, Med. Density Lumber										
Roof Edge Distance:		Bottom side: 10 inches only, Left, Right and Top Sides: 10 inches minimum								
Roof Framing Lmbr:		Douglas Fir or Southern Pine (G=0.49 or higher)				Allowable Uplift Capacity:			153 lbs	
		B			C			D		
ASCE 7-05 (equiv.)	ASCE 7-10 (exact)	Slope 0:12-1:12 (0° - 7°)	Slope 2:12-6:12 (8° - 27°)	Slope 7:12-12:12 (28° - 45°)	Slope 0:12-1:12 (0° - 7°)	Slope 2:12-6:12 (8° - 27°)	Slope 7:12-12:12 (28° - 45°)	Slope 0:12-1:12 (0° - 7°)	Slope 2:12-6:12 (8° - 27°)	Slope 7:12-12:12 (28° - 45°)
87	110	0.69	0.61	0.74	1.07	0.95	1.11	1.30	1.16	1.35
91	115	0.78	0.69	0.82	1.19	1.05	1.23	1.45	1.29	1.49
95	120	0.87	0.77	0.91	1.31	1.17	1.36	1.60	1.42	1.64
103	130	1.06	0.94	1.11	1.58	1.41	1.62	1.91	1.71	1.96
111	140	1.27	1.13	1.31	1.87	1.67	1.91	2.25	2.02	2.30
119	150	1.49	1.33	1.53	2.18	1.95	2.22	2.62	2.35	2.67
126	160	1.73	1.54	1.77	2.51	2.25	2.56	3.02	2.70	3.06
134	170	1.98	1.77	2.02	2.87	2.57	2.91	3.43	3.08	3.48

Note: Numbers in table are wind uplift demand/capacity ratios (DCRs).
Viable regions are shown in green. Installation is not permitted in red regions.

Modules:	39.5"x66"		Roof Mean Height (ft):	30	ft.
Orientation:	Portrait		Dead Load (psf):	3.00	psf
Roof Lumber:	Douglas Fir (G=0.49 or higher)		Demand Increase Factor:	1.48	
Kopp Factor:	0.65		Off-Center Factor:	0.94	
Tributary Area:	9	sq.ft.	Initially Wet Lmbr Reduction Fact	0.66	
Rafter Spacing:	24	inches on center	6d Nail Reduction Factor:	0.78	
Toggle Bolt:	1/4"	diameter	16" Rafter Spacing Increase Facto	1.00	

Table E.7.c.1: Anchors within bands of strength, for 6d sheathing nails fastened into initially wet, Douglas Fir lumber, with modules extending into wind roof Zone 2 to within 10 inches of the roof edge.

As Table E.7.c.1 shows, the DCRs for Exposure B, 120 mph design wind speed are 0.77 for 2:12 to 6:12 roof slopes, and 0.91 for 7:12 to 12:12 roof slopes. The off-center factor of 0.94 listed in the footnotes reflects the small reduction in strength from locating anchors no more than 8” from the longitudinal centerline of the roof panels. The demand increase factor of 1.48 reflects the specific properties of the SMASHsolar system, but can be generalized to reflect other systems that project into roof wind zone 2 with cantilevers no more than 19 inches long.

Like case E.7.b, when mounts are only anchored in bands of strength, the mounts are assumed to only impose uplift, and not bending moment, on the sheathing. The DCRs in Table E.7.c.1 do not include a capacity reduction factor for prying, so mounts must be designed to not impose bending moment loads on the sheathing. Other types of mounts should have an additional capacity reduction factor to account for prying imposed on the sheathing-to-rafters nails. Unless testing of mount/sheathing/rafter test beds indicate otherwise, the prying capacity reduction factor should be assumed to be 0.80 or less.

E.7.d. All anchors are within bands of strength, and all the following (i, ii and iii) apply:

- ☐ *i. Edge of array meets E.1 and is within 3 feet of a roof corner (Wind Zone 3), and*
- ☐ *ii. Tributary area including cantilevers is 4.5 ft^2 or less (32.5”X20”).*
- ☐ *iii. Wind Exposure B only, and design wind speed does not exceed 120 mph.*

The reasoning is similar to the previous case, noting that the ratio of Zone 3 to Zone 2 uplift is 2.6/1.7 = 1.53, while the tributary is halved, so the resulting DCRs are less than one.

E.8. Anchor-to-sheathing connection has an allowable stress design (ASD) uplift capacity of at least 166 lbs. under short duration loading, which corresponds to a mean ultimate tested uplift capacity of at least 520 lbs.

To concur with the assumptions behind the calculations and DCR tables that support the E.7 provisions, the mounts shall be anchored to the sheathing with an allowable stress design (ASD) uplift capacity of at least 166 pounds under short duration loading. This corresponds to a mean ultimate tested capacity (6 samples minimum) of at least 520 pounds, based on at least 6 replicates. A factor of safety of 5.0 for single fasteners, increased by the load duration factor of 1.60 yields a short duration ASD allowable capacity of (520 lbs/5.0)(1.60) = 166 lbs.

APPENDIX 1: SHEATHING AND SHEATHING NAILING CODE HISTORY

The demand/capacity calculations for sheathing-attached systems are based on 6d common or 8d box nails with 6" o.c. edge and 12" o.c. field nailing that fasten 15/32" or thicker plywood or OSB to rafters at 24" on center.

Building codes since the late 1990s have required sheathing nails to be at least 8d box (.113" diam. x 2.5" long). Before then, 6d common nails (.113" diam.x3.0") were the minimum allowed, but anecdotal evidence suggests 8d box or 8d common nails were typically used. Because 6d common nails and 8d box nails have the same diameter, but nail embedment into the rafters is shorter (1.5" versus 2.0"), roofs nailed with 6d common nails have 75% of the wind-uplift resistance of 8d box nails.

Appendix 1 Table 1 summarizes the code history of roof sheathing nail requirements for plywood and OSB. Codes since the late 1990s have required sheathing nails to be at least 8d box (.113" diam. x 2.5" long). Before then, 6d common nails (.113" diam.x3.0") were the minimum allowed, but 8d box or 8d common nails were often used

Through the decades, the code has been very consistent regarding nail spacing. Maximum allowed nail spacing for conventionally laid unblocked roof plywood has remained remarkably constant: 6" on center (o.c.) at the supported short edges of panels, and 12" on center "in the field" at intermediate supporting rafters.

Alternative nailing and stapling at closer spacing was also allowed in some codes. To compare these alternatives to the typical 8d box at 12" o.c. field nailing, wind uplift capacity on a per square foot basis was calculated on the basis of rafters at 24" o.c. Hence, for fasteners at 12" o.c., the uplift capacity is the allowable withdrawal of the nail divided by 2 square feet (an area 1 ft. x 2 ft.). Where uplift capacities on a per square foot basis are less than the 8d box nail at 12" o.c., the entries are shown in red.

Since the late 1990s, the codes have also included special nailing provisions for the perimeter 4 or 5 feet of each roof plane. Appendix 1 Table 1 shows the equivalent uplift capacity in these regions, which create field nailed regions with uplift capacities two to four times higher than the typical 8d box nail at 12" o.c. field nailing. Feet located in these regions will have much greater wind uplift capacity. If a home is located in a high wind speed region, 130 mph or higher under the ASCE 7-10 code, and was constructed under the 1996 BOCA, 1997 UBC, 2000 IRC or later codes, then tighter nailing and greater uplift capacity can be expected in roof Zones 2 and 3.

For regions such as the Midwest and Northeast, where low density framing lumber such as Spruce Pine Fir (SPF) is often used, it is important to field verify that sheathing nails are 8d box or larger instead of 6d common. This can be done in one of two ways:

- The roof framing lumber has grade stamps indicating dense wood species such as Douglas Fir or Southern Pine instead of typical Spruce-Pine-Fir, or
- Nails larger than the minimum-sized 6d nails were used. If existing roof framing is visible in an attic, the installer should review the visible framing. If "shiners" (nails missing rafters) are visible, the installer should verify they project at least 2" through the underside of plywood or OSB, thereby verifying that the nails are 8d or greater. If shiners are not visible, a pachometer or similar magnetic field-measuring device can be used to non-destructively locate and measure roofing nails to determine nail length.

Appendix 1 Table 1. Code Minimum Roof Sheathing and Nailing for Rafters at 24" on center.

Code	Plywd. Min. Thk. inches	Nail Size: Nom. Size pennywt.	Nail Size: Nom. Type	Nail Size: Actual Size	Nail Spacing: Edge	Nail Spacing: Field	Uplift Capacity (7) psf	Plywd. Ref. Table No.	Nailing Ref. Table No.	Nailing Ref. Item
IBC 2000-2015 &	7/16"	**8d (1)**	**box (2)**	**.113"x2.50"**	**6"**	**12"**	**28.5**	2304.8(3)	2304.9.1	31
CBC 2007-2013	7/16"	8d	sinker (3)	.113"x2.38""	4"	8"	40.1			
	7/16"	16ga	staple (4)(8)	16ga x1.75"	3"	6"	33.5			
IRC 2006-2012	7/16"	8d	common (5)	.131"x2.50"	6"	12"	33.1	R503.2.1.1(1)	R602.3(1)	
CRC 2010-2013	7/16"	15ga	staple (6)	15ga x1.75"	4"	8"	28.9	R503.2.1.1(1)	R602.3(2)	
	7/16"	16ga	staple (6)	16ga x1.75"	3"	6"	33.5	R503.2.1.1(1)	R602.3(2)	
	7/16"	.097/.099	nail	.098"x2.25"	3"	6"	39.9	R503.2.1.1(1)	R602.3(2)	
IRC 2000-2003	7/16"	8d	common (5)	.131"x2.50"	6"	12"	33.1	R503.2.1.1(1)	R602.3(1)	
	7/16"	15ga	staple (6)	15ga x1.75"	6"	**12"**	**19.3**	R503.2.1.1(1)	R602.3(2)	
	7/16"	16ga	staple (6)	16ga x1.75"	6"	**12"**	**16.8**	R503.2.1.1(1)	R602.3(2)	
	7/16"	.097/.099	nail	.098"x2.25"	3"	6"	39.9	R503.2.1.1(1)	R602.3(2)	
CBC 2001 UBC 1997	7/16"	8d	common (5)	.131"x2.50"	6"	12"	33.1	R503.2.1.1(1) 23-II-E-1	R602.3(1) 23-II-B-2	
UBC 1979-1994 BOCA 1978-1999	15/32"	**6d**	common	.113"x2.00"	6"	12"	**21.4**	23-I-S-1 2307.3.1(2)	23-I-Q 2305.2	26
UBC 1967-1976 BOCA 1965-1975	1/2"	**6d**	common	.113"x2.00"	6"	12"	**21.4**	25-R-1	25-P App. M	
UBC 1961	1/2"	10d	common	.148x3.00"	6"	12"	46.7	25-M	2511(c)	
BOCA 1950-1955	1/2"	**6d**	common	.113"x3.00"	6"	**10"**	**25.7**		App. L	

(1) Table says 6d nail but footnote "l" says that for roofs, use 8d-box = .113"x2.5"

(2) Note that 8d box nails and 6d common nails have the same 0.113" diameter.

(3) Per table footnote "n".

(4) Per table footnote "o".

(5) common or deformed

(6) Staple values per International Staple, Nail and Tool Association's ICC ESR-1539:

20 lb/in for 16ga, 23 lb/in for 15 ga, in G=0.42 lumber, with 1.34 upper bound on load duration factor

(7) Allowable Stress Design (ASD) capacity

Appendix 1 Table 2: Recent Bldg Codes' High Wind Region Special Perimeter Nailing Provisions									
Code	Wind Speed (mph)	ASCE 7 Ed.	Equiv. ASCE 7-10	Nail Size	Nail Type	Extent (2)(3)	Field (3)(4)	Uplift (5) Capacity (psf)	Ref.
IRC 2000-2012 &	110+	7-05	140+	.120"x2.50"	deformed	48" (1)	6" o.c.	121.9	(6)
CRC 2001-2013	100-110	7-05	130-140	8d (.131x2.5")	common	48" (1)	6" o.c.	66.1	(6)
CBC 1998-2001 & UBC 1997 & BOCA 1996	90+	7-95	130+	8d (.131x2.5")	common	48", 60"	6" o.c.	66.1	(7)

(1) Distance from roof gable ends, eaves and ridges

(2) First number is distance from eaves and ridges, second number is distance from gable ends

(3) In all instances, panel edge nailing is 6" o.c. and gable end wall nailing is 4" o.c

(4) Field nailing is referred to "intermediate support" nailing in the IRC and CRC since 2000.

(5) Allowable Stress Design (ASD) uplift capacity for field nailing on a lbs/sq.ft. basis.

(6) Table R602.3(1) w/ footnotes f & g.

(7) Table 23-II-B-2 w/ footnote 4.

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UL 2703: *Standard for Mounting Systems, Mounting Devices, Clamping/Retention Devices and Ground Lugs for Use with Flat-Plate Photovoltaic Modules and Panels,* January 2015. Underwriters Laboratories Inc.

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- 2015 California Solar Permitting Guidebook's Toolkit Structural Document, published by the California Governor's Office of Planning and Research
- 2013 East Bay Green Corridor's Solar Permitting Initiative, published by the Center for Sustainable Energy
- 2011 Expedited Permit Process for PV Systems: A Standardized Process for the Review of Small-Scale PV Systems, published by the Solar America Board for Codes and Standards

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2015 California Solar Permitting Guidebook's Toolkit Structural Document

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Steve Bauer SE, Unirac
Joe Cain PE, DNV GL
Richard Hanson PE, Solar City
James Lai SE, Chair, SEAOC Wind Committee
Joe Maffei SE, Maffei Structural Engineering, Chair, SEAOC Solar PV Committee
Jeremy Rogelstad PE, ZEP Solar
Norm Scheel SE, Normal Scheel Structural Engineer
Andrew Wagner PE, Tipping Structural Engineers
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East Bay Green Corridor 2013 CBC Update

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