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 class rating for the PV system. Fire rating Class (A, B, or C).
4. Specify anchor-to-roof sealing (e.g. flashing, or sealant compatible with roofing)

The building code requires that PV systems meet the minimum required fire class rating that is stated for roofing for the specific building type. The building code does not require that the PV system match the rating of the rating of roofing materials on the building, as some jurisdictions have erroneously interpreted the requirement. The basic building code requirement for residential roofing is class C. Upwards of half of the population of California live in areas where class A roofing is required for dwellings. The only way to comply with a class A requirement is for the PV system, including the racking system and modules, to be evaluated to the fire performance test in UL2703. Currently, well over a dozen mounting system products have achieved class A fire ratings.

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. Several products are available on the market that provide a metal flashing around anchor fasteners. While this may not be explicitly required in the residential and building codes, these flashing products represent best practices for sealing the anchor fasteners for many member-attached mounting systems. Any sealant products that may be used to seal anchor fasteners must be compatible with the roofing materials that the sealant is adhered to. The most successful and long-lasting products used with asphalt shingle roofing materials have been urethane sealant products with over 30 years positive results sealing anchor fasteners.
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 setback distance may be much more where fire access pathways are required. For instance, the International Fire Code and NFPA1 fire code generally require three feet between the ridge and the top of the array, to allow firefighters ample access to the ridge to cut vent holes to vent hot gases during 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.
<table>
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<tr>
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<td>-25lb/ft</td>
<td>-25lb/ft</td>
<td>-25lb/ft</td>
<td>49.7</td>
<td>101.2</td>
</tr>
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</table>
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 run perpendicular to rafters or trusses.

This section addresses the typical case, where rails run perpendicular to rafters in a member-attached system. The unusual case where the rails run upslope/downslope aligned with rafters are covered in F.1. In this D.5 case, sections D.6 and D.7 address the spacing and loading limits, while in the latter case, F.1. addresses the spacing and loading limits by reducing the snow and windspeed limits.

**D.6. The anchor/mount/stand-off spacing perpendicular to rafters or trusses does not exceed 4’-0”, and anchors in adjacent rows are staggered where rafters or trusses are at 24” or less on center.

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, CLSF, 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 CLSF (1.51 in this case). To be conservative, CLSF based on the midspan loading case was used in the subsequent analysis. Note that uniform loading has a CLSF 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>
<thead>
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<th>Anchor/Rafter Spacing, n</th>
<th>Rafter Span (in.)</th>
<th>Rafter Span (ft-in)</th>
<th>Concentrated Load Sharing Factor, C_(LSF)</th>
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<td></td>
<td></td>
<td></td>
<td>7/16&quot; OSB (1)</td>
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Legend:
Green shaded values (7/16" OSB) are the basis of the California 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 or 1x 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, $EI = 78,000 \text{ lb-in}^2/\text{ft}$ 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, $EI = 125,000 \text{ lb-in}^2/\text{ft}$ 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: $EI = 115,000 \text{ lb-in}^2/\text{ft}$.

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

4. 1x sheathing typical stiffness, $EI = (1,600,000) \times (12 \times 0.75^{3/12}) = 675,000 \text{ lb-in}^2/\text{ft}$ if solid sheathed, $\approx 300,000 \text{ lb-in}^2/\text{ft}$ if skip sheathed.

5. Rafter assumed to be Douglas Fir-Larch #2 with $E = 1,600,000 \text{ psi}$ per NDS Supplement Table 4A. Rafter span taken from 2012 IBC 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 ($675,000 \text{ lb-in}^2/\text{ft} / 2 = 338,000 \text{ lb-in}^2/\text{ft}$). 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.6.1 below and Table D.6.2 include both 7/16" OSB and 15/32" sheathing thicknesses and their associated stiffnesses.
The Concentrated Load Sharing (Redistribution) Factor, $C_{LSR}$, 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-rafter stiffness 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 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.
Further Refinements to CLSF

The following are potential future refinements to the Concentrated Load Sharing Factor ($C_{CLSF}$) 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:

\[ C_r \] 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 Distribution 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 \( \sigma \) around a mean \( \mu \), 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:

\[
F_b = 900 \text{ psi} = \left[ \mu - 1.645(570) \right] / 2.16 \therefore \mu = 2,882 \text{ psi}
\]

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

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

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 \( C_r \) 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:

\[
DCR = \frac{M_{\text{demand}}}{M_{\text{capacity}}} = \frac{wL^2}{8} \frac{L^2}{C_D F_b S}
\]

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:

\[
DCR = \frac{D_{\text{with PV}}}{D_{\text{without PV}}}
\]

where:

- \( D_{\text{with PV}} = \max(D_{PV+DL}, D_{PV+\text{wind down}+DL}, D_{\text{wind up}+PV-DL}) \)

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

- \( D_{\text{without PV}} = \max(D_{DL+LLr}, D_{DL+\text{wind down}}, D_{DL+LLr+\text{wind down}}, D_{\text{wind up}-DL}) \)

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

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

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

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

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

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

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

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

where:

- \( n \) = anchor spacing/rafter spacing
- \( C_{LSF} \) = Concentrated Load Sharing Factor
- \( \theta \) = roof slope where \( 0^\circ \) = 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)
- \( p_{\text{wind \_down}} \) = wind downward pressure per ASCE 7-10 Chapter 30 Part 1, \( C_{pi} = 0 \) (without 16 psf minimum)
- \( p_{\text{wind \_up}} \) = wind upward pressure per ASCE 7-10 Chapter 30 Part 1, \( C_{pi} = 0 \) (without 16 psf minimum)
\[ C_L = \text{beam stability factor (assumed to be 0.80)} \]
\[ C_{D,DL} = \text{load duration factor for dead load} = 0.90 \]
\[ C_{D,LL} = \text{load duration factor for roof live load} = 1.25 \]
\[ C_{D,wind} = \text{load duration factor for wind} = 1.60 \]

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:

2012 IBC 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 fire code 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 \((0.88)(0.86) = 0.76\) where 50% of DCRs are expected to be higher and 50% lower; and the 90% DCR is \((0.92)(0.97) = 0.89\) 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\(^{1,2}\)

<table>
<thead>
<tr>
<th>Rafter</th>
<th>Depth (in.)</th>
<th>Spacing (in.)</th>
<th>Strength Index</th>
<th>Incremental Relative Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>2x4</td>
<td>3.5</td>
<td>24</td>
<td>0.51</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>0.77</td>
<td>0.61</td>
</tr>
<tr>
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<td>24</td>
<td>1.26</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>1.89</td>
<td>0.86</td>
</tr>
<tr>
<td>2x8</td>
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<td>24</td>
<td>2.19</td>
<td>0.67</td>
</tr>
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<td></td>
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<td>5.35</td>
<td>-</td>
</tr>
<tr>
<td>Avg</td>
<td></td>
<td></td>
<td></td>
<td>0.72</td>
</tr>
</tbody>
</table>

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 \(i+1\)

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**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 threshold values are shown below for mounts at 48 inch spacing and rafters at 16” and 24” on center. When this ground snow load threshold 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.

**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 PV module manufacture’s requirements for allowable stresses on the module, 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 \( (205 \text{ lbs/in})(2.5” – 3/16” \text{ tip length})(Cd = 1.6) = 758 \text{ lbs} \). 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 \( (25.7 \text{ psf})(40”x48”/144) = 343 \text{ lbs} \), 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.**

Some manufacturers of anchors may use different fastener arrangements than 5/16” lag screws. These manufacturers would need to provide engineering comparison to typical 5/16” lag screws to show equal or greater strength of their fastening system for it to be used by the simplified permit guidelines.

**E. High Wind Member-Attached Array Additional Requirements (all of A. through D. complies and design wind speed does not exceed 180 mph)**

The four additional restrictions placed on locations where the design wind speed can reach 180 mph are based on the same analysis used to generate the general requirements. By limiting the wind pressure by restricting the PV modules to wind zone 1 provides a means to stay within the limitations of both the PV modules and the roof structure for wind speeds up to 180 mph. Further reducing the cantilever for the PV modules to only 6” constrains the loading on the perimeter anchors that generally control both up and down forces on the anchors. Lastly, constraining the anchor spacing to no greater than 2’ generally forces the array to be anchored to every rafter or truss. This additional spacing limitation keeps the uplift forces within the acceptable range for the fasteners specified in D.8.

**F. Low Wind and Low Snow Reduced Member-Attached Array Requirements (design wind speed does not exceed 120 mph and ground snow load no greater than 10 pdf)**

- **F.1. Mounting rail orientation run parallel to rafters and are spaced no more than 4’-0” apart.**

This is an exception to D.5. that allows the rails to be run parallel to rafters if they are no greater than 4 feet apart for low snow and wind design conditions.
F.2.a. Anchor/mount/stand-off spacing perpendicular to rafters or trusses does not exceed 4 feet and anchor layout is orthogonal.

This is an exception to D.6. that allows the anchors layout to be orthogonal. This means that the anchors in adjacent rows need not be staggered to provide for an evenly distributed roof load. In addition to the basic requirements of 10 psf snow load limit and 120 mph wind load limit, a maximum roof slope of 6:12 is also required to allow this arrangement.

F.2.b. Anchor/mount/stand-off spacing perpendicular to rafters or trusses does not exceed 6 feet and anchor layout is orthogonal.

This is an exception to D.6. is similar to F.2.a. except that it allows the rails to be run parallel to rafters if they are no greater than 6 feet apart. The additional requirement for this configuration is that there is no snow load at all.