03 June 2009

C.R. Laurence Co., Inc.
2503 East Vernon
Los Angeles, CA 90058

SUBJ: CR LAURENCE SUN SHADES SERIES 7750

The CRL 7750 Series Aluminum Sun Shades were evaluated in accordance with the 2006 International Building Code and the 2005 Aluminum Design Manual to determine the allowable wind and snow loads.

The sun shades will safely support the following loading:

- Distributed live load = 25 psf over projected horizontal surface
- Concentrated live load = 300#
- Wind load = 50 psf
- Snow load = 50 psf
- Snow load + Wind load = 75 psf

For corner sections:

- Distributed live load = 25 psf over projected horizontal surface
- Concentrated live load = 300#
- Wind load = 38 psf
- Snow load = 38 psf
- Snow load + Wind load = 50 psf

Loading is based on using Hilti HSL-3 concrete anchors size 8mm with 2-3/8" embedment in to concrete with a minimum strength of $f'_c$ = 2,500 psi. The sunshades may be attached to structural steel using 3/8" stainless steel bolts ASTM A276-85a Condition A or stronger.

The supporting structure shall be adequate to support the reactions as shown herein.

Edward Robison, P.E.

REGISTERED PROFESSIONAL ENGINEER
EDWARD C. ROBISON
No. C 65883
CIVIL
STATE OF CALIFORNIA
EXP 12/31/2009
Signed 06/03/2009

Attachments –
Calculations: 7 pages
Shop Drawings: 2 Sheets

*Edward C. Robison, PE*
*10012 Creviston DR NW*
*Gig Harbor, WA 98329*
*253-858-0855*
*FAX 253-858-0856*

WIND LOADING ON SUNSCREENS

Calculated in accordance with SEI/ASCE 7-05 Section 6.4 SIMPLIFIED PROCEDURE.

$K_{zt}$ From Figure 6-4 for the site topography = 1.0.
V = Wind speed (mph) 3 second gust
$p_{net30}$ = from Figure 6-3 Roof overhangs.
$\lambda$ = from Figure 6-3
$w_v = p_{net30} * \lambda$ (uplift)

The wind load will cause a vertical uplift force

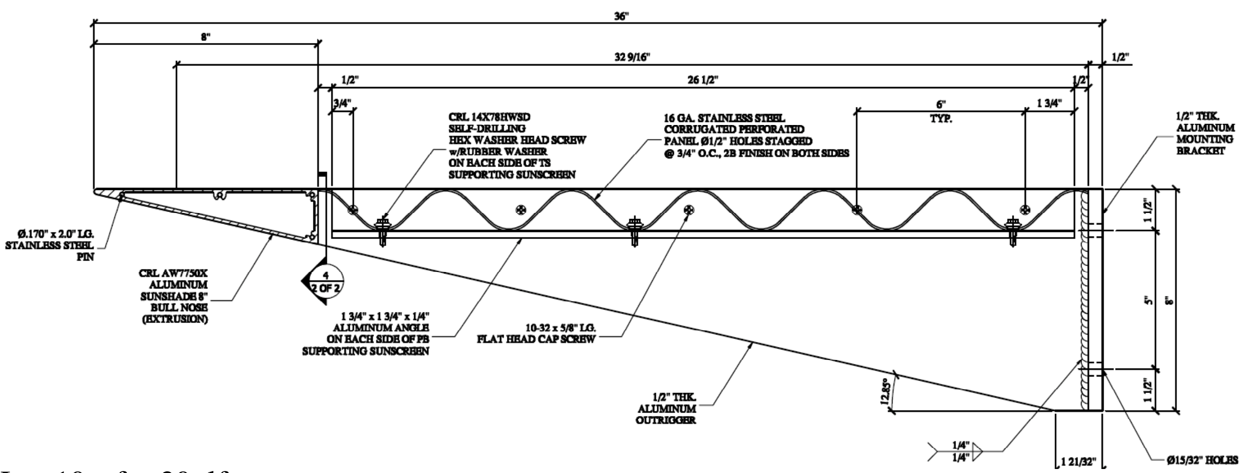
SNOW LOADING

Calculated in accordance with SEI/ASCE 7-05 Section 7.

$p_f = 0.7C_eC_tIp_g = 0.7*1.1*1.2*1.0*p_g = 0.924p_g$ psf
$p_s = C_sp_f = 0.38* p_f =$ psf
$p_{ir}$ = 5psf for icing and rain
$S = p_s + 5.0 =$ psf

ICE LOADING SEI/ASCE 7-05 Section 10

1" Equivalent = 5.2 psf




L = 10psf = 20plf

Dead load

D = 3.2psf*2.33'+3.0plf = 10.5 plf = 3.5psf

36” PROJECTION SUNSHADE
Check based on a standard sun shade length of 48”:
Bending of corrugated metal:
Metal section properties based on perforated panel having 33% perforations and 24 ga minimum:
$I_x = 0.104\ in^4/ft$
$S_x = 0.217\ in^3/ft$

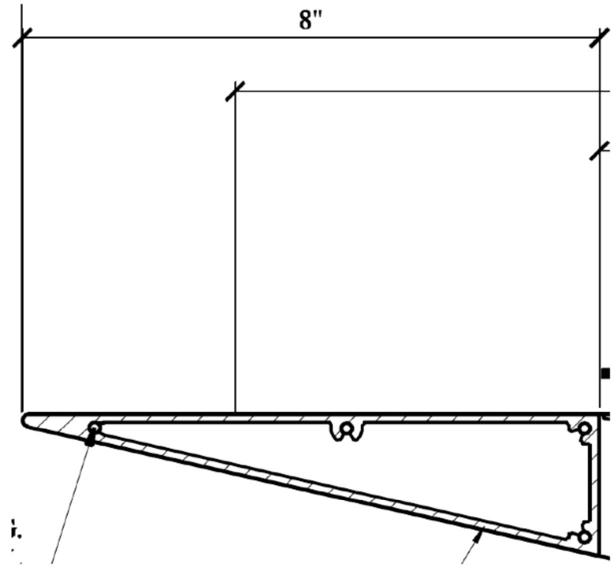
Live concentrated load:
$M_1$ = 48”*300/4 + 0.25pli*48”$^2$/8 = 3,672#”
$f_b$ = 3,672#”/(0.217 in$^3$*2.33’) = 7,263 psi

Determine allowable stress from SEI/ASCE 8-02 Table A1
$F_Y$ = 41 ksi longitudinal compression
$M_a = S*F_c$ = 0.9*2.33*0.217*41ksi/1.6 = 11,660#”

Allowable uniform loads on corrugated metal:
U = 8*$M_a$/$L^2$ =[8*11,660#”/48$^2$]/2.33 = 17.38pli = 208plf

Corrugated metal is attached to end bars by bearing on L1 3/4” x 1 3/4” x 1/4” aluminum angles
Allowable load on angle based on bending of bottom leg:
$F_{bt} = F_{bc}$ = 20 ksi (ADM Table 2-24)
S= 12”*0.25$^2$/6 = 0.125 in$^3$
$M_a$ = 0.125*20ksi = 2,500#”
Load on angle lip:
P = $M_a$/e = 2,500#”/(1.75/2) = 2,857plf
Angle bending won’t control sun shade loading

Check Bull nose section




Check local bending of bull nose:

Concentrated load:
M = 200#*6.625”/5 = 265#”

Resisting width = 2*6.625”+6”
S = 19.25”*$0.125^2$/6 = 0.0501$in^3$
$f_b$ = 265#”/0.0501 = 5,286 psi

Check for wind load:
M = W*$0.67^2$/10 = 0.04489W#”/ft
Local bending won’t control loading.

Allowable stress from ADM Table 2-21 for
6061-T6 extrusion
$F_{bt} = F_{bc}$ = 28 ksi

Check for bending between supports:
$I_{xx}$ = 0.938 $in^4$
$S_{xx}$ = 0.663 $in^3$
determine allowable stress $F_{cb}$:
b/t = 3.36”/0.125 = 26.9
$F_{cb}$ = 27.3-0.292*26.9 = 19.45 ksi

$M_a$ = 0.663*19.45ksi = 16,896#” = 1,075#’
$U_a$ = 1,075#’*8/4’$^2$ = 537plf: 806 psf
Bending of bull nose section will not control sunshade loading

Attachment to end angles with (4) #8 screws:
#8 countersunk screws:
$P_{nov} = (0.27+1.45t_1/D)Dt_1F_{ty1}$ ADM eq 5.4.2.2-2
$P_{nov}$ = (0.27+1.45*0.125/0.1339)0.1339*0.125*25ksi = 679#
$P_a$ = 679/3 = 226#

$Z_a = 2F_{tu1}Dt_1/n_u$ ADM Eq 5.4.3-1
$Z_a$ = 2*30ksi*0.1339*0.125/3 = 335# per screw

Screw shear:
$V_s$ = 0.65*33.7ksi*0.014$in^2$ = 307#

Connection strength = 4*226# = 904# each end
U = 904#/(4’/2) = 452plf
U = 451plf/0.667’ = 676psf
(Will not control loading)

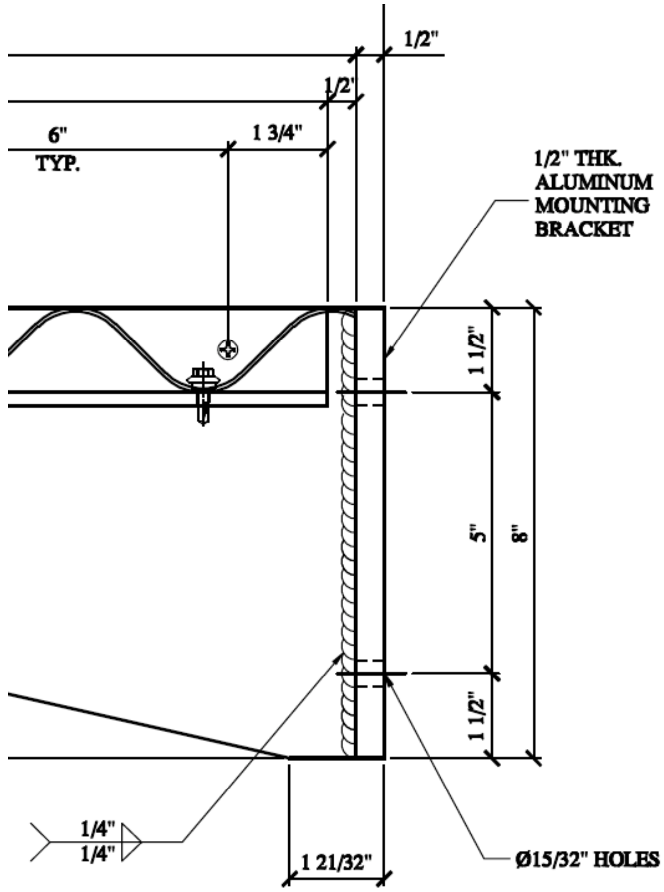
ATTACHEMENT TO WALL:
Out rigger bar is welded to wall plate.
Weld strength in accordance with ADM Section 7.
1/4” fillet weld all around bar, 5356 weld filler

$V_w = F_{sw}L_{we}/n_u$

$V_w$ = 17ksi*8”*2*(0.707*0.25)/1.95 = 49.3k

$S_w = 2*(0.707*0.25)*8^2/6 = 3.77in^3$

$M_{wa} = 3.77in^3*17ksi/1.95 = 32.87k”$




Check strength of weld affected bar:

$S = 0.5”*8^2/6 = 5.33\ in^3$

$M_{bw}$ = 5.33*6.5ksi = 34.67k”

Weld strength will control bar loading.

Allowable uniform load on bar:
3’ projection and 4’ sun shade length:

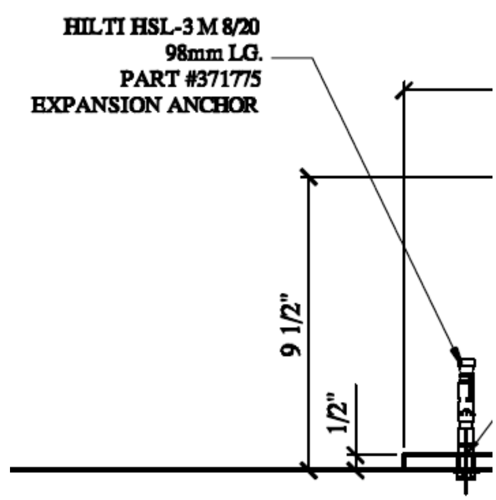
$U = 32.87k”*2/(36^2)$ = 50.7pli = 609plf

u = 609plf/2’ = 304psf

weld strength will not control sun shade loading.

Determine Anchor loads:
For shear:
V = U*3.5’*L/2 for 4’ section: V = 7U psf
where: U = D+ S or W; or U = D+0.75(S+W)
let u = greater of W, S or 0.75(W+S) psf
V = 7*(3.5+u) = 24.5+7u
From ∑M about edge of the wall plate = 0:
0 = 18”*V − (1.5”+5”)*T
solving for T:
T = (18”*V)/6.5 = [18”*(24.5+7u)]/6.5
T = 67.85+19.38u

Anchor strength:
8mm Hilti HSL-3 embed depth 2-3/8”
Allowable loads from Hilti Technical data and ESR 1545:
T = 1,167# (see next page)
V = 0.65*2,107/1.6 = 856#




Substitute into above equations and solve for u:
V = 856# = 24.5+7u
u = (856-24.5)/7 = 118.8 psf
from T:
T = 1,167# = 67.85+19.38u
u = (1,167-67.85)/19.38 = 56.7psf
Tension will control allowable loading

**Table 5 - HSL-3 Allowable Static Tension (ASD), Normal Weight Cracked Concrete (lb)[1,3,4]**

| Nominal Anchor Diameter | Embedment Depth hef mm | (in.) | Concrete Compressive Strength[2] $f'_c$ = 2000 psi Condition A | Condition B | $f'_c$ = 3000 psi Condition A | Condition B | $f'_c$ = 4000 psi Condition A | Condition B | $f'_c$ = 6000 psi Condition A | Condition B |
|---|---|---|---|---|---|---|---|---|---|---|
| M8 | 60 | 2.36 | 1,167 | 1,167 | 1,429 | 1,429 | 1,650 | 1,650 | 2,021 | 2,021 |
| M10 | 70 | 2.76 | 1,867 | 1,867 | 2,286 | 2,286 | 2,640 | 2,640 | 3,233 | 3,233 |
| M12 | 80 | 3.15 | 3,214 | 2,785 | 3,936 | 3,411 | 4,545 | 3,939 | 5,567 | 4,825 |
| M16 | 100 | 3.94 | 4,492 | 3,893 | 5,501 | 4,768 | 6,352 | 5,505 | 7,780 | 6,743 |
| M20 | 125 | 4.92 | 6,277 | 5,440 | 7,688 | 6,663 | 8,877 | 7,694 | 10,873 | 9,423 |
| M24 | 150 | 5.91 | 8,252 | 7,152 | 10,106 | 8,759 | 11,670 | 10,114 | 14,292 | 12,387 |

1 Values are for single anchors with no edge distance or spacing reduction. For other cases, see ESR-1545 Section 4.2 Eq. 5.

2 Values are for normal weight concrete. For sand-lightweight concrete, multiply values by 0.85. For all-lightweight concrete, multiply values by 0.75. See ACI 318-05 Section D.3.4.

3 Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.

4 Allowable static tension loads for 2,500 psi are calculated by multiplying the pullout strength $N_{pn}$ by the strength reduction $\phi$ factor of 0.65 and dividing by an $\alpha$ of 1.4 according to ICC ESR-1545 Section 4.2. See Table 2 for $N_{pn}$. This load may be adjusted for other concrete strengths according to ICC ESR-1545 Section 4.1.3 by using the following equation.

$$N_{pn,cr,f'c} = N_{pn,cr}\sqrt{\frac{f'c}{2500}}$$

Concrete breakout strength in shear:

$V_{cbg} = A_{vc}/A_{vco}(\varphi_{ec,V}\ \varphi_{ed,V}\varphi_{c,V}\varphi_{h,V})V_b$

$A_{vc} = 4.5*2.375^2 = 25.38$

$A_{vco} = 4.5(c_{a1})^2 = 4.5(2.375)^2 = 25.38$

$\varphi_{ec,V} = 1/[1+2e'_v/3c_{a1}] = 1/[1+2*0/(3*2.375)] = 1.0$

$\varphi_{ed,V} = 1.0$ ($c_{a2} \geq 1.5c_{a1}$)

$\varphi_{c,V} = 1.4$ uncracked concrete

$\varphi_{h,V} = \sqrt{(1.5c_{a1}/h_a)} = \sqrt{(1.5*2.375/2.375)} = 1.225$

$V_b = [8(l_e/d_a)^{0.2}\sqrt{d_a}]1\sqrt{f'_c}(c_{a1})^{1.5} = [8(2.375/0.313)^{0.2}\sqrt{0.313}]1.0\sqrt{2500}(2.375)^{1.5} = 1{,}228\#$

$V_{cb} = 25.38/25.38*1.0*1.4*1.225*1{,}228\# = 2{,}107\#$

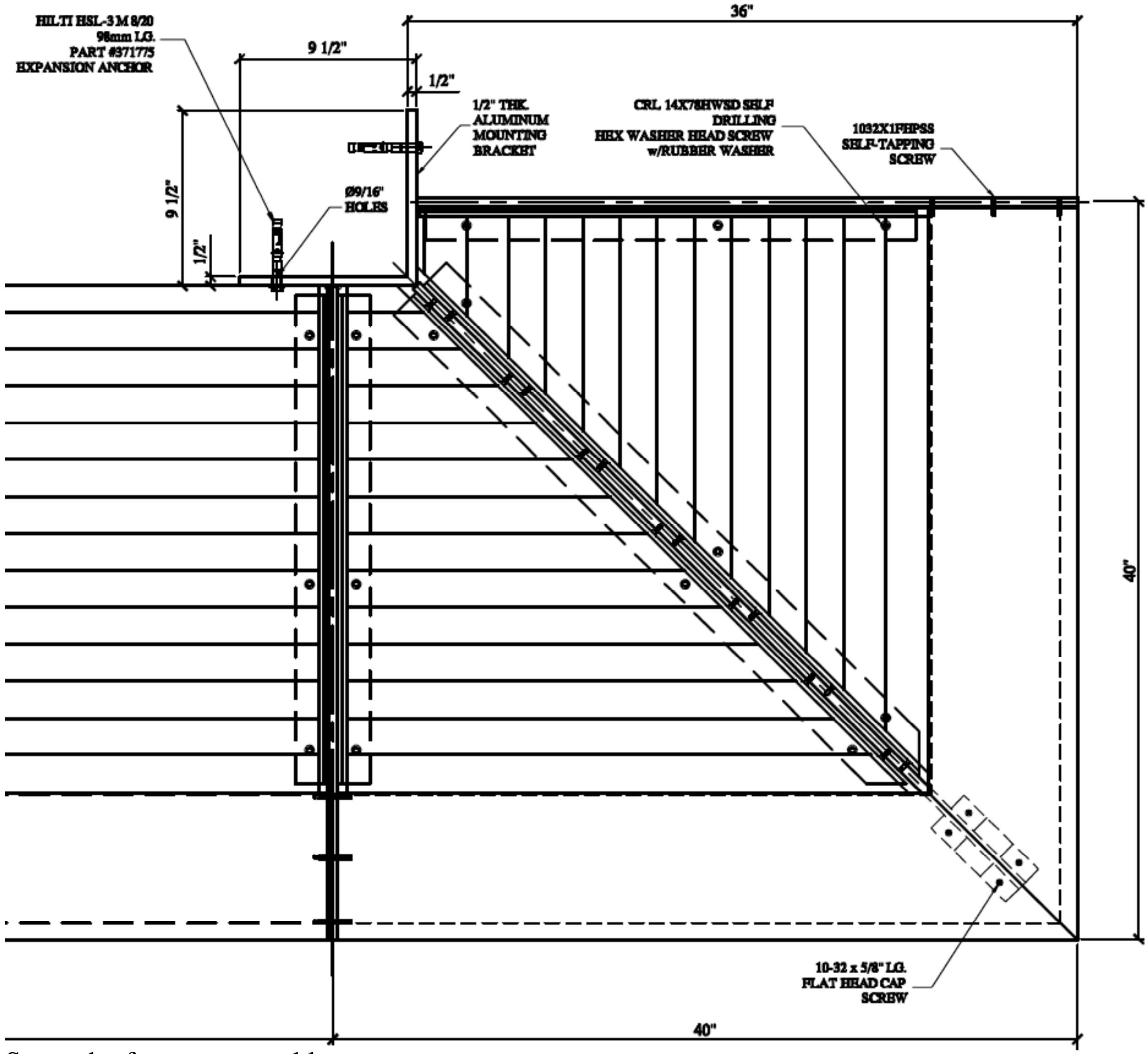
Concrete breakout will control shear strength.

Check bearing strength on bolt holes:
allowable bearing strength from ADM Table 2-24 line 5: $F_B = 31$ ksi
B = 0.5”*0.313”*31ksi = 4,852# (bearing on plate won’t control loading)

MAXIMUM ALLOWABLE LOADS ON SUN SHADE WITH 36” PROJECTION:
L = 300#
S = 50psf
W = 50psf downward and 60psf uplift
S + W = 75 psf

WALL REACTIONS:
Shear: V = 24.5+7u
Tension: T = 67.85+19.38u
where: u = greater of W, S or 0.75(W+S) psf

CORNER SECTIONS:




Strength of corner assembly:

Corner bar tributary load:

Triangular load: (0.33*0.707)U at wall to (3.33*0.707)U at end

$M = 0.233U*4.243'^2/2 + 2.35U*4.243^2/3 = 16.2U$

$U = 3.5+u$

$M = 56.7 + 16.2u$

Bar and weld has same strength as previously checked and therefore will not control the allowable loads.

Corner Assembly Attachment to Wall:
Anchors are 8” from corner so no reduction in strength for the corner is required.

Maximum loading to the corner bracket is:
From front 2’4” tributary
From side 2’4” tributary
From corner: Triangle load varies from 0 to 6’
V = 2.33’*3’*U+0.5*3’*6’*U = 16U
M = 7*3’/2*U + 9*(2/3)*3’*U = 28.5U’

Determine moment strength of bracket anchorage to wall:
Each anchor is loaded in shear from parallel segment and tension by perpendicular segment.
For combined case:

$$T/T_a + V/V_a \leq 1.2$$

When properly installed the tension load will be achieved before the shear load.

From page 6:

$T_a = 1{,}167\#$
$V_a = 856\#$

From $\sum$ of moments about bottom edge of bracket:
On parallel face top anchor is in tension, bottom anchor resists vertical shear,
On perpendicular face both anchors act in shear
Result is that top bolts are loaded in tension with shear load and bottom bolts are loaded in shear only

$M_a = 1.0*T_a*6.5" + 0.2*V_a*\sqrt{(6.5^2+8^2)} + 0.707*1.0V_a*\sqrt{(1.5^2+8^2)} = 6.5T_a+7.82V_a =$
$M_a = 6.5*1{,}167+7.82*856 = 14{,}279\#" = 1{,}190\#'$ each way.

28.5U = 1,190#’
U = 1,190/28.5 = 41.75psf

u = U-3.5psf = 41.75-3.5 = 38.25 psf

MAXIMUM ALLOWABLE LOADS ON SUN SHADE CORNER WITH 36” PROJECTION:
L = 300#
S = 38psf
W = 38psf downward and 43psf uplift
S + W = 50 psf

WALL REACTIONS:
Shear: V = 16u + 56
Tension: T = 100 +28.5u
where: u = greater of W, S or 0.75(W+S) psf