HGA

1200 R St, Suite 100
Sacramento, CA 95811
(916) 787-5100 fax (916) 784-7738

UC DAVIS:
PSSB NETV2 UPGRADE

Structural Calculations

OFFICE OF THE STATE FIRE MARSHAL
APPROVED FIRE AND PANIC ONLY

Will Gilliland
Will Gilliland
Fire Inspector
On behalf of: Stephen Guarino
Lead Designated Campus Fire Marshal

Approval of this plan does not authorize or approve any omission or deviation from applicable regulations. Final approval is subject to field inspection. One set of approved plans shall be available on the project site at all times.

REGISTERED PROFESSIONAL ENGINEER
BRENT A. FORSLIN
S 3124
3-31-2025
STRUCTURAL
STATE OF CALIFORNIA

UC DAVIS HEALTH BUILDING DEPARTMENT
APPROVED
REVIEWED FOR CODE COMPLIANCE
The set of plans and specifications must be kept on the job site at all times and it is unlawful to make any changes or alterations to the approved set without written permission from the Building Department.
The approval of this plan and specifications SHALL NOT be held to permit or approve the violation of any University Policy or State Building Code.
BY: Paul R Menard AIA, CBO DATE: 1/10/2024
PROJECT #: UCDH-2023-0230
UC Davis Health
Sacramento, CA 95817

This approval includes 19 pages.

Commission No: 1500-159-01
4150 V st, Sacramento, CA 95817
Date: June 13, 2023
Resubmittal 01: October 27, 2023

OSHPD

Patient Support Services, 4150 V St, Sacramento, CA 95817, USA

Latitude, Longitude: 38.5561454, -121.4567732

Date	6/13/2023, 1:27:33 PM
Design Code Reference Document	ASCE7-16
Risk Category	IV
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	0.549	MCE_R ground motion. (for 0.2 second period)
S_1	0.248	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.747	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.498	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.36	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.231	MCE_G peak ground acceleration
F_{PGA}	1.369	Site amplification factor at PGA
PGA_M	0.316	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	0.549	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.576	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.248	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.263	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.231	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.953	Mapped value of the risk coefficient at short periods
C_{R1}	0.943	Mapped value of the risk coefficient at a period of 1 s

P2

Type	Value	Description
C_V	1.066	Vertical coefficient

DISCLAIMER

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HGA

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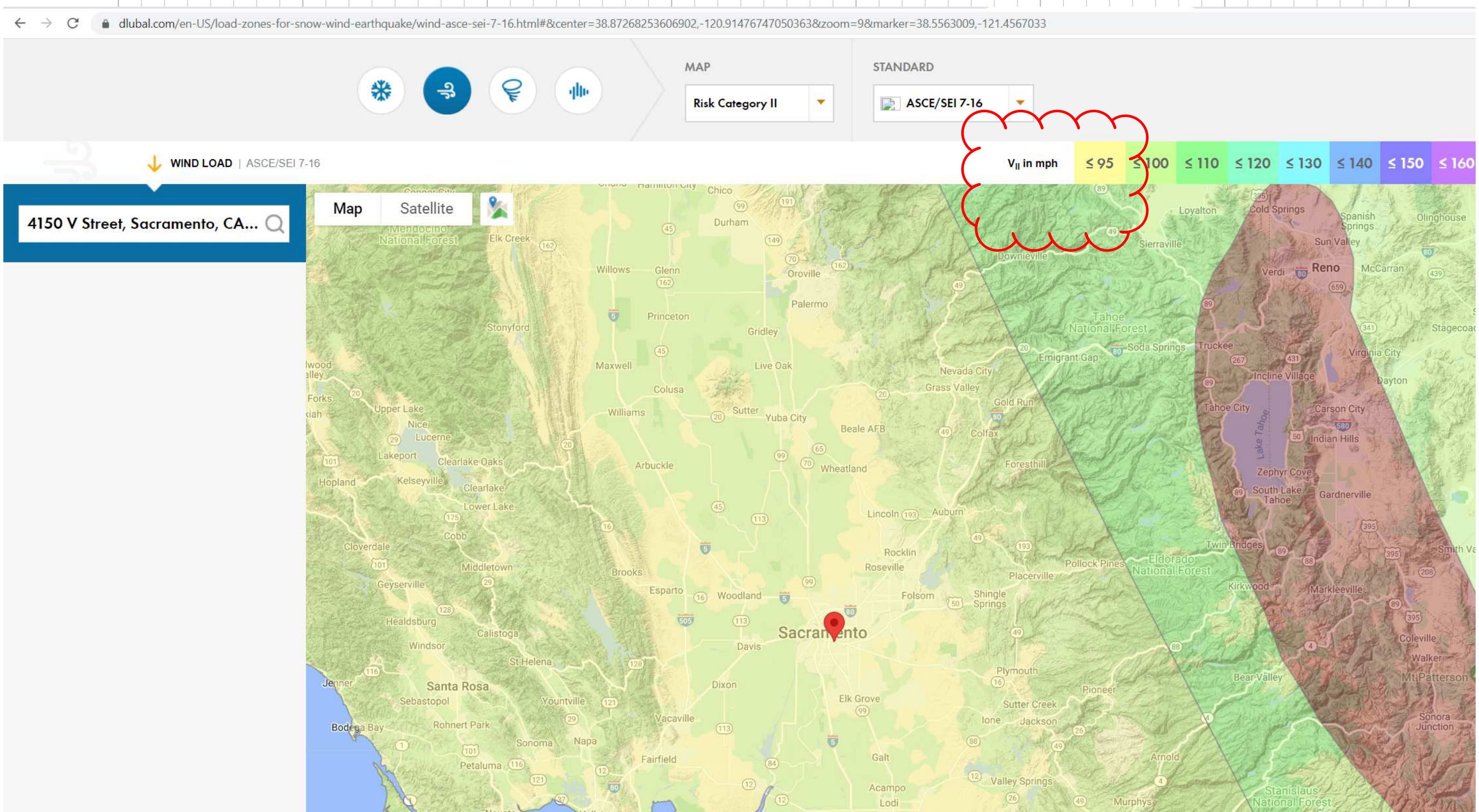
Name ______ Date ______



PLOT DATE: May 09, 2017

P5

EXPANSION ANCHOR ALLOWABLE STRENGTHS

TABLE 1

EXPANSION ANCHORS INSTALLED IN TO THE UNDERSIDE OF STRUCTURAL SAND-LIGHTWEIGHT CONCRETE (f'c MIN=3000 PSI) OVER METAL DECK

ANCHOR DIA. (IN)	EMBED (IN)	SHEAR (LB)	TENSION (LB)
3/8	2	747	604
1/2	2 1/4	1029	610
1/2	3 1/4	1173	1086
5/8	3 1/4	1353	836
5/8	4 1/4	2477	1941

TABLE 2

EXPANSION ANCHORS INSTALLED IN TO THE TOP OF STRUCTURAL SAND-LIGHTWEIGHT CONCRETE (f'c MIN=3000 PSI) OVER METAL DECK

ANCHOR DIA. (IN)	EMBED (IN)	SHEAR (LB)	TENSION (LB)
3/8	2	806	624
1/2	2 1/4	948	660

TABLE 3

EXPANSION ANCHORS INSTALLED IN NORMAL WEIGHT CONCRETE (f'c MIN=3000 PSI)

ANCHOR DIA. (IN)	EMBED (IN)	SHEAR (LB)	TENSION (LB)
3/8	2	1020	961
1/2	2 1/4	1580	1101
1/2	3 1/4	2591	2003
5/8	3 1/4	2579	2150
5/8	4 1/4	3772	3113

SECTION TITLE:
STANDARD PARTITION WALL DETAILS

SHEET TITLE:
EXPANSION ANCHOR ALLOWABLE STRENGTHS

OPD NO.:
ST1.04

HGA

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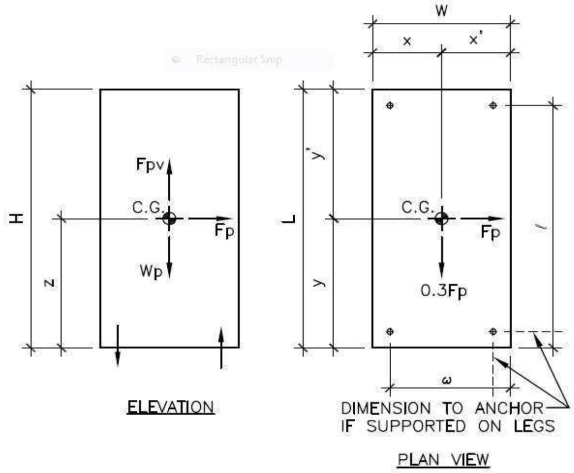
Name:	Voznenko	Date:	6/14/23

2019 CBC & ASCE 7-16 EQUIPMENT ANCHORAGE FORCES - CU-1



Height, H = 53.0 in
Height to center of gravity, z = 27.0 in
Width, W = 13.0 in
Overturning Dimension, ω = 13.0 in
# of anchors in tension, $\#_{T,\omega}$ = 2
x = 6.5 in
x' = 6.5 in
Length, L = 42.0 in
Overturning Dimension, ℓ = 24.0 in
# of anchors in tension, $\#_{T,\ell}$ = 4
y = 21.0 in
y' = 21.0 in
Weight, W_p = 214 lbs
# of anchors in shear, $\#_V$ = 4
Height of component with respect to grade, z = 40 ft
Average roof height, h = 40 ft

Seismic

Seismic design requirements for equipment are based on ASCE 7-16, Chapter 13.

COMPONENT AMPLIFICATION FACTOR
ASCE Section 13.5, 13.6 & ASCE Table 13.5-1, 13.6-1 — a_p = 1.0

COMPONENT RESPONSE MODIFICATION FACTOR
ASCE Section 13.5, 13.6 & ASCE Table 13.5-1, 13.6-1 — R_p = 2.5

DESIGN SPECTRAL RESPONSE ACCELERATION
CBC Section 1613A.3.4 & CBC Equation 16A-39 — S_{DS} = 0.498

COMPONENT IMPORTANCE FACTOR
ASCE Section 13.1.3 — I_p = 1.00

ATTACHMENT FACTOR IN CONCRETE OR MASONRY
ASCE Section 13.4.2.1 and ACI 318-14 sec 17.2.3.4.3 d — Ω factor = 1.0

SEISMIC DESIGN FORCE

ASCE Section 13.3.1 & ASCE Equation 13.3-1	$F_p = 0.4 a_p S_{DS} W_p/(R_p/I_p)(1+2z/h)$	F_p = 0.239 Wp
ASCE Section 13.3.1 & ASCE Equation 13.3-2	$F_{p.max} = 1.6 S_{DS} I_p W_p$	$F_{p,max}$ = 0.797 Wp
ASCE Section 13.3.1 & ASCE Equation 13.3-3	$F_{p,min} = 0.3 S_{DS} I_p W_p$	$F_{p,min}$ = 0.149 Wp

SEISMIC DESIGN FORCES (ASD)

ASCE Section 13.1.7 & 13.3.1	$F_{p,ASD} = 0.7(F_{p,govern})$	$F_{p,ASD}$ = 0.167 Wp
ASCE Section 13.1.7 & 13.3.1	$F_{pv,ASD} = 0.7(0.2 S_{DS} W_p)$	$F_{pv,ASD}$ = 0.070 Wp

DESIGN FORCES

$F_{p,\Omega} = Fp_{,ASD} * W_p * \Omega$ factor = 36 lbs
OTM = $z * F_{p,\Omega}$ = 967 lb-in
$F_{pv,ASD}$ = 15 lbs
DLRM = $(0.6W_p - F_{pv,ASD}) * x_{min}$ = 738 lb-in

$$T = \frac{OTM - DLRM}{\omega * \#_{T,\omega}} + \frac{0.3 * OTM}{\ell * \#_{T,\ell}}$$

T = 12 lbs

$$V = \frac{F_{p,ASD,\Omega} * (2 * y_{max} / L)}{\#_V}$$

V = 9 lbs

(V is approximate when number of anchors exceeds 4)

Use (4) - 3/8"Ø Hilti Kwik Bolt TZ w/ 2" Effective Embedment (Slab on Grade Condition, 4" min)

Updated Sheet P7

R1

Architecture | Engineering | Planning

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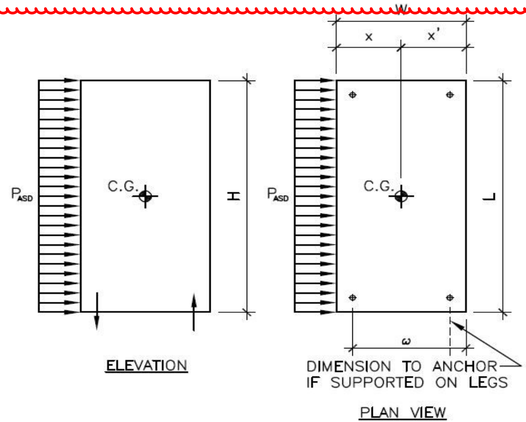
2022 CBC & ASCE 7-16 EQUIPMENT ANCHORAGE FORCES - CU-1



Height, H = 53.0 in
Width, W = 13.0 in
Overturning Dimension, ω = 13.0 in
# of anchors in tension, $\#_{T,\omega}$ = 2
x = 6.5 in
x' = 6.5 in
Length, L = 24.0 in
Weight, W_p = 214 lbs
# of anchors in shear, $\#_V$ = 4
Height of component with respect to grade, z = 40 ft

R1

Wind

Wind design requirements for equipment are based on ASCE 7-16, Section 30.11

RISK CATEGORY (OC)
ASCE TABLE 1.5-1 — RC = II

BASIC WIND SPEED (3 SECOND GUST)
ASCE Figure 26.5-1 — V = 95 mph

EXPOSURE CATEGORY (EC)
ASCE SECTION 26.7 — EC = C

VELOCITY PRESSURE EXPOSURE COEFFICIENT
ASCE Table 26.3.1 — k_z = 1.04

TOPOGRAPHIC FACTOR
ASCE Section 26.8.2 — k_{zt} = 1.0

WIND DIRECTIONALITY FACTOR
ASCE Table 26.6.1 — k_d = 0.85

Note: R1
Load less than calculation shown in P8 in original calc package OK.

FORCE COEFFICIENT R1
ASCE Section 29.4.1 — GC_r = 1.5

Ground Elevation Factor
ASCE Table 26.9-1 — Ke= 1.0

VELOCITY PRESSURE
ASCE Section 29.3.2 — $q_z = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot Ke \cdot V^2$ — q_z = 20.5 psf

DESIGN WIND FORCE
ASCE Section 29.5.1 — $P_{ASD} = 0.6 \cdot (q_z \cdot GC_r)$ — P_{ASD} = 18.4 psf

DESIGN FORCES

$F_{ASD} = P_{ASD} \cdot (H \cdot L/144)$ = 163 lbs
$OTM = F_{ASD} \cdot (H/2)$ = 4318 lb-in
$DLRM = (0.6W_p) \cdot x_{min}$ = 835 lb-in

$$T = \frac{OTM - DLRM}{\omega \cdot \#_T}$$

$$V = \frac{F_{ASD}}{\#_V}$$

T = 134 lbs
V = 41 lbs

Use (4) 3/8"Ø Hilti Kwik Bolt TZ w/ 2" Effective Embedment (Slab on Grade Condition, 4" min)
T_{ALL} = 1154 lbs
V_{ALL} = [illegible] lbs
UNITY CHECK = 0.16 OK

Not used.

R1

HGA

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CONDENSATE UNIT (CU-1) ANCHORAGE

MAX WT = 214 lbs

T_{max} = 191 lbs

V_{max} = 55 lbs

$z = \sqrt{191^2 + 55^2} = 199$ lbs

$\angle = \tan^{-1}(191/55) = 54°$

3/8" Ø x 2 1/2" lag screw

Check lag capacity

W = 214 lbs

W'p = 214 lbs x 2 1/2" = 535 lbs

z_1 = 140 lbs (G = 0.5, 12 Ga) = 140 lbs x 2 1/2" / 8 3/8" = 117 lbs

Please see P12

$$z'_\angle = \frac{1.6 \times (W'p) z'}{(W'p)\cos^2\angle + z'\sin^2\angle} = \frac{1.6 \times 535 \times 117}{535 \times \cos(54°) + 117 \times \sin(54°)} = 244.5 \text{ lbs}$$

$z'_\angle > z$

244.5 lbs > 199 lbs ✓ GOOD

HGA

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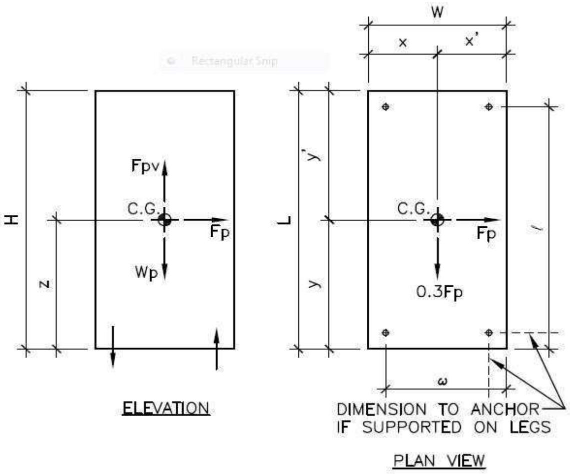
2019 CBC & ASCE 7-16 EQUIPMENT ANCHORAGE FORCES - CU-2



Height, H = 38.0 in
Height to center of gravity, z = 19.0 in
Width, W = 13.0 in
Overturning Dimension, ω = 13.0 in
# of anchors in tension, $\#_{T,\omega}$ = 2
x = 6.5 in
x' = 6.5 in
Length, L = 38.0 in
Overturning Dimension, ℓ = 24.0 in
# of anchors in tension, $\#_{T,\ell}$ = 4
y = 19.0 in
y' = 19.0 in
Weight, W_p = 153 lbs
# of anchors in shear, $\#_V$ = 4
Height of component with respect to grade, z = 40 ft
Average roof height, h = 40 ft

Seismic

Seismic design requirements for equipment are based on ASCE 7-16, Chapter 13.

COMPONENT AMPLIFICATION FACTOR
ASCE Section 13.5, 13.6 & ASCE Table 13.5-1, 13.6-1 — a_p = 1.0

COMPONENT RESPONSE MODIFICATION FACTOR
ASCE Section 13.5, 13.6 & ASCE Table 13.5-1, 13.6-1 — R_p = 2.5

DESIGN SPECTRAL RESPONSE ACCELERATION
CBC Section 1613A.3.4 & CBC Equation 16A-39 — S_{DS} = 0.498

COMPONENT IMPORTANCE FACTOR
ASCE Section 13.1.3 — I_p = 1.00

ATTACHMENT FACTOR IN CONCRETE OR MASONRY
ASCE Section 13.4.2.1 and ACI 318-14 sec 17.2.3.4.3 d — Ω factor = 1.0

SEISMIC DESIGN FORCE

ASCE Section 13.3.1 & ASCE Equation 13.3-1	$F_p = 0.4{*}a_p{*}S_{DS}{*}W_p/(R_p/I_p)(1+2z/h)$	F_p = 0.239 Wp
ASCE Section 13.3.1 & ASCE Equation 13.3-2	$F_{p.max} = 1.6{*}S_{DS}{*}I_p{*}W_p$	$F_{p,max}$ = 0.797 Wp
ASCE Section 13.3.1 & ASCE Equation 13.3-3	$F_{p,min} = 0.3{*}S_{DS}{*}I_p{*}W_p$	$F_{p,min}$ = 0.149 Wp

SEISMIC DESIGN FORCES (ASD)

ASCE Section 13.1.7 & 13.3.1	$F_{p,ASD} = 0.7(F_{p,govern})$	$F_{p,ASD}$ = 0.167 Wp
ASCE Section 13.1.7 & 13.3.1	$F_{pv,ASD} = 0.7(0.2{*}S_{DS}{*}W_p)$	$F_{pv,ASD}$ = 0.070 Wp

DESIGN FORCES

$F_{p,\Omega} = Fp_{,ASD} * W_p *$ Ω factor = 26 lbs
$OTM = z * F_{p,\Omega}$ = 486 lb-in
$F_{pv,ASD}$ = 11 lbs
$DLRM = (0.6W_p - F_{pv,ASD}) * x_{min}$ = 527 lb-in

$$T = \frac{OTM - DLRM}{\omega * \#_{T,\omega}} + \frac{0.3 * OTM}{\ell * \#_{T,\ell}}$$

T = 0 lbs

$$V = \frac{F_{p,ASD,\Omega} * (2 * y_{max} / L)}{\#_V}$$

V = 6 lbs

(V is approximate when number of anchors exceeds 4)

Use (4) - 3/8"Ø Hilti Kwik Bolt TZ w/ 2" Effective Embedment (Slab on Grade Condition, 4" min)

Architecture | Engineering | Planning

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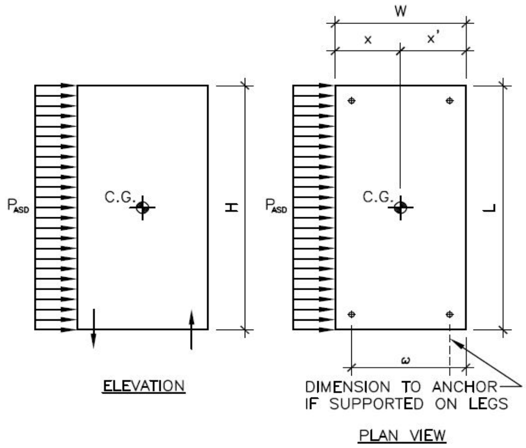
2013 CBC & ASCE 7-10 EQUIPMENT ANCHORAGE FORCES - CU-2



Height, H = 38.0 in
Width, W = 13.0 in
Overturning Dimension, ω = 13.0 in
# of anchors in tension, $\#_{T,\omega}$ = 2
x = 6.5 in
x' = 6.5 in
Length, L = 24.0 in
Weight, W_p = 153 lbs
# of anchors in shear, $\#_V$ = 4
Height of component with respect to grade, z = 40 ft

Wind

Wind design requirements for equipment are based on ASCE 7-10, Section 30.11

RISK CATEGORY (OC)
ASCE TABLE 1.5-1 — RC = II

BASIC WIND SPEED (3 SECOND GUST)
ASCE Figure 26.5-1 — V = 95 mph

EXPOSURE CATEGORY (EC)
ASCE SECTION 26.7 — EC = C

VELOCITY PRESSURE EXPOSURE COEFFICIENT
ASCE Table 29.3.1 — k_z = 1.04

TOPOGRAPHIC FACTOR
ASCE Section 26.8.2 — k_{zt} = 1.0

WIND DIRECTIONALITY FACTOR
ASCE Table 26.6.1 — k_d = 0.90

FORCE COEFFICIENT
ASCE Section 29.5.1 — GC_r = 1.9

VELOCITY PRESSURE
ASCE Section 29.3.2 — $q_z = 0.00256 * K_z * K_{zt} * K_d * V^2$ — q_z = 21.7 psf

DESIGN WIND FORCE
ASCE Section 29.5.1 — $P_{ASD} = 0.6 * (q_z * GC_r)$ — P_{ASD} = 24.7 psf

DESIGN FORCES

$F_{ASD} = P_{ASD} * (H*L/144) =$ 157 lbs
$OTM = F_{ASD} * (H/2) =$ 2977 lb-in
$DLRM = (0.6W_p) * x_{min} =$ 597 lb-in

$$T = \frac{OTM - DLRM}{\omega * \#_T}$$

$$V = \frac{F_{ASD}}{\#_V}$$

T = 92 lbs
V = 39 lbs

Use (4) 3/8"Ø Hilti Kwik Bolt TZ w/ 2" Effective Embedment (Slab on Grade Condition, 4" min)

Please see allowable strength

Govern

HGA

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CONDENSATE UNIT (CU-2) ANCHORAGE

MAX WT = 153 lbs

T_{max} = 92 lbs

V_{max} = 39 lbs

$z = \sqrt{92^2 + 39^2} = 99.9$ lbs

$\angle = \tan^{-1}(92/39) = 36°$

3/8" Ø × 2 1/2" lag screw

Check lag capacity

W = 153 lbs

W'p = 153 lbs × 2 1/2" = 383 lbs

z_1 = 140 lbs (G = 0.5, 12 Ga) = 140 lbs × 2 1/2" / 8 × 3/8" = 117 lbs

Please see P12

$$z'_{\angle} = \frac{1.6 \times (W'p) z'}{(W'p)\cos^2\angle + z' \sin^2\angle} = \frac{1.6 \times 383 \times 117 \text{ lbs}}{383 \times \cos(36°) + 117 \times \sin(36°)} = 189.4 \text{ lbs}$$

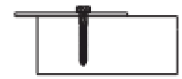
$z'_{\angle} > z$

189.4 lbs > 99.9 lbs ✓ Good

P12

LAG SCREWS

Table 12K LAG SCREWS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2,3,4]



for sawn lumber or SCL with ASTM A653, Grade 33 steel side plate (for t_s<1/4") or ASTM A 36 steel side plate (for t_s=1/4")
(tabulated lateral design values are calculated based on an assumed length of lag screw penetration, p, into the main member equal to 8D)

Side Member Thickness	Lag Screw Diameter	G=0.67 Red Oak		G=0.55 Mixed Maple Southern Pine		G=0.5 Douglas Fir-Larch		G=0.49 Douglas Fir-Larch (N)		G=0.46 Douglas Fir(S) Hem-Fir(N)		G=0.43 Hem-Fir		G=0.42 Spruce-Pine-Fir		G=0.37 Redwood		G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods		G=0.35 Northern Species	
t_s	D	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$
in.	in.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
0.075	1/4	170	130	160	120	150	110	150	110	150	100	140	100	140	100	130	90	130	90	130	90
(14 gage)	5/16	220	160	200	140	190	130	190	130	190	130	180	120	180	120	170	110	170	110	160	100
	3/8	220	160	200	140	200	130	190	130	190	120	180	120	180	120	170	110	170	100	170	100
0.105	1/4	180	140	170	130	160	120	160	120	160	110	150	110	150	110	140	100	140	100	140	90
(12 gage)	5/16	230	170	210	150	200	140	200	140	190	130	190	130	190	120	180	110	170	110	170	110
	3/8	230	160	210	140	200	140	200	130	200	130	190	120	190	120	180	110	180	110	170	110
0.120	1/4	190	150	180	130	170	120	170	120	160	120	160	110	160	110	150	100	150	100	140	100
(11 gage)	5/16	230	170	210	150	210	140	200	140	200	140	190	130	190	130	180	120	180	120	180	110
	3/8	240	170	220	150	210	140	210	140	200	130	200	130	190	120	180	110	180	110	180	110
0.134	1/4	200	150	180	140	180	130	170	130	170	120	160	120	160	110	150	110	150	100	150	100
(10 gage)	5/16	240	180	220	160	210	150	210	140	200	140	200	130	200	130	190	120	180	120	180	120
	3/8	240	170	220	150	220	140	210	140	210	140	200	130	200	130	190	120	190	120	180	110
0.179	1/4	220	170	210	150	200	150	200	140	190	140	190	130	190	130	180	120	170	120	170	120
(7 gage)	5/16	260	190	240	170	230	160	230	160	230	150	220	150	220	150	210	130	200	130	200	130
	3/8	270	190	250	170	240	160	240	160	230	150	220	140	220	140	210	130	210	130	200	130
0.239	1/4	240	180	220	160	210	150	210	150	200	140	190	140	190	130	180	120	180	120	180	120
(3 gage)	5/16	300	220	280	190	270	180	260	180	260	170	250	160	250	160	230	150	230	150	230	140
	3/8	310	220	280	190	270	180	270	180	260	170	250	160	250	160	240	140	230	140	230	140
	7/16	420	290	390	260	380	240	370	240	360	230	350	220	350	220	330	200	330	200	320	190
	1/2	510	340	470	300	460	290	450	280	440	270	430	260	420	260	400	240	400	230	390	230
	5/8	770	490	710	430	680	400	680	400	660	380	640	370	630	360	600	330	590	330	580	320
	3/4	1110	670	1020	590	980	560	970	550	950	530	920	500	910	500	860	450	850	450	840	440
	7/8	1510	880	1390	780	1330	730	1320	710	1280	690	1250	650	1230	650	1170	590	1160	590	1140	570
	1	1940	1100	1780	960	1710	910	1700	890	1650	860	1600	820	1590	810	1500	740	1480	730	1460	710
1/4	1/4	240	180	220	160	210	150	210	150	200	140	200	140	190	130	180	120	180	120	180	120
	5/16	310	220	280	200	270	180	270	180	260	170	250	170	250	160	230	150	230	150	230	140
	3/8	320	220	290	190	280	180	270	180	270	170	260	160	250	160	240	150	240	140	230	140
	7/16	480	320	440	280	420	270	420	260	410	250	390	240	390	230	370	220	360	210	360	210
	1/2	580	390	540	340	520	320	510	320	500	310	480	290	480	290	460	270	450	260	440	260
	5/8	850	530	780	470	750	440	740	440	720	420	700	400	690	400	660	370	650	360	640	350
	3/4	1200	730	1100	640	1060	600	1050	590	1020	570	990	540	980	530	930	490	920	480	900	470
	7/8	1600	930	1470	820	1410	770	1400	750	1360	720	1320	690	1310	680	1240	630	1220	620	1200	600
	1	2040	1150	1870	1000	1800	950	1780	930	1730	900	1680	850	1660	840	1570	770	1550	760	1530	740

1. Tabulated lateral design values, Z, shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Tabulated lateral design values, Z, are for "reduced body diameter" lag screws (see Appendix Table L2) inserted in side grain with screw axis perpendicular to wood fibers; screw penetration, p, into the main member equal to 8D; dowel bearing strengths, F_e, of 61,850 psi for ASTM A653, Grade 33 steel and 87,000 psi for ASTM A36 steel and screw bending yield strengths, F_{yb}, of 70,000 psi for D = 1/4", 60,000 psi for D = 5/16", and 45,000 psi for D ≥ 3/8".
3. Where the lag screw penetration, p, is less than 8D but not less than 4D, tabulated lateral design values, Z, shall be multiplied by p/8D or lateral design values shall be calculated using the provisions of 12.3 for the reduced penetration.
4. The length of lag screw penetration, p, not including the length of the tapered tip, E (see Appendix Table L2), of the lag screw into the main member shall not be less than 4D. See 12.1.4.6 for minimum length of penetration, p_{min}.

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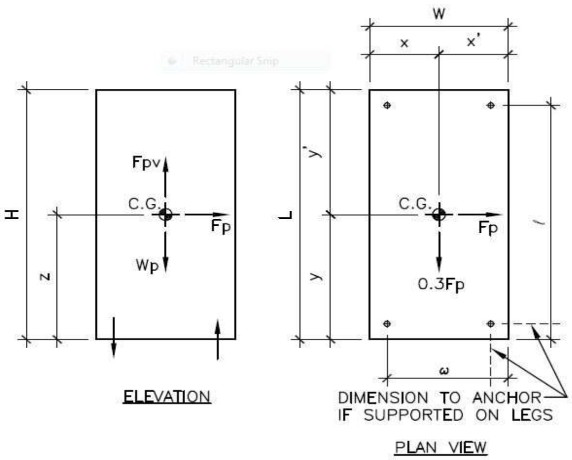
2019 CBC & ASCE 7-16 EQUIPMENT ANCHORAGE FORCES - UPS



Height, H = 78.7 in
Height to center of gravity, z = 35.6 in
Width, W = 39.1 in
Overturning Dimension, ω = 39.1 in
# of anchors in tension, $\#_{T,\omega}$ = 2
x = 19.6 in
x' = 19.6 in
Length, L = 47.4 in
Overturning Dimension, ℓ = 47.4 in
# of anchors in tension, $\#_{T,\ell}$ = 4
y = 23.7 in
y' = 23.7 in
Weight, W_p = 842 lbs
# of anchors in shear, $\#_V$ = 4
Height of component with respect to grade, z = 0 ft
Average roof height, h = 1 ft

Seismic

Seismic design requirements for equipment are based on ASCE 7-16, Chapter 13.

COMPONENT AMPLIFICATION FACTOR
ASCE Section 13.5, 13.6 & ASCE Table 13.5-1, 13.6-1 — a_p = 2.5

COMPONENT RESPONSE MODIFICATION FACTOR
ASCE Section 13.5, 13.6 & ASCE Table 13.5-1, 13.6-1 — R_p = 2.5

DESIGN SPECTRAL RESPONSE ACCELERATION
CBC Section 1613A.3.4 & CBC Equation 16A-39 — S_{DS} = 0.498

COMPONENT IMPORTANCE FACTOR
ASCE Section 13.1.3 — I_p = 1.00

ATTACHMENT FACTOR IN CONCRETE OR MASONRY
ASCE Section 13.4.2.1 and ACI 318-14 sec 17.2.3.4.3 d — Ω factor = 2.0

SEISMIC DESIGN FORCE

ASCE Section 13.3.1 & ASCE Equation 13.3-1	$F_p = 0.4*a_p*S_{DS}*W_p/(R_p/I_p)(1+2z/h)$	F_p = 0.199 Wp
ASCE Section 13.3.1 & ASCE Equation 13.3-2	$F_{p.max} = 1.6*S_{DS}*I_p*W_p$	$F_{p,max}$ = 0.797 Wp
ASCE Section 13.3.1 & ASCE Equation 13.3-3	$F_{p,min} = 0.3*S_{DS}*I_p*W_p$	$F_{p,min}$ = 0.149 Wp

SEISMIC DESIGN FORCES (ASD)

ASCE Section 13.1.7 & 13.3.1	$F_{p,ASD} = 0.7(F_{p,govern})$	$F_{p,ASD}$ = 0.139 Wp
ASCE Section 13.1.7 & 13.3.1	$F_{pv,ASD} = 0.7(0.2*S_{DS}*W_p)$	$F_{pv,ASD}$ = 0.070 Wp

DESIGN FORCES

$F_{p,\Omega} = Fp_{,ASD} * W_p * \Omega$ factor = 235 lbs
$OTM = z * F_{p,\Omega}$ = 8359 lb-in
$F_{pv,ASD}$ = 59 lbs
$DLRM = (0.6W_p - F_{pv,ASD}) * x_{min}$ = 8751 lb-in

$$T = \frac{OTM - DLRM}{\omega * \#_{T,\omega}} + \frac{0.3 * OTM}{\ell * \#_{T,\ell}}$$

T = 8 lbs

$$V = \frac{F_{p,ASD,\Omega} * (2 * y_{max} / L)}{\#_V}$$

V = 59 lbs

(V is approximate when number of anchors exceeds 4)

Use (4) - 3/8"Ø Hilti Kwik Bolt TZ w/ 2" Effective Embedment (Slab on Grade Condition, 4" min)

T_{ALL} = 961 lbs
V_{ALL} = 1020 lbs
UNITY CHECK = 0.07 OK

R1 HGA

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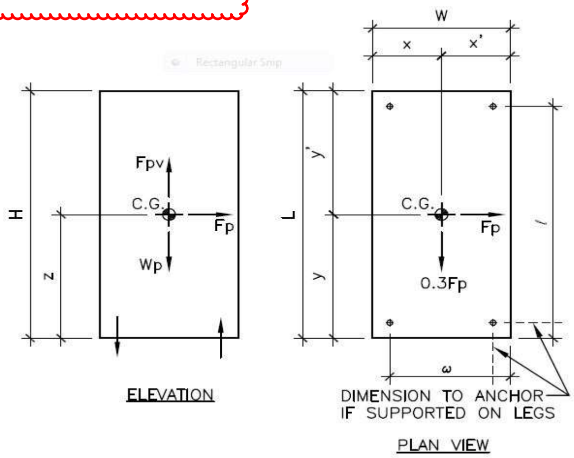
2022 CBC & ASCE 7-16 EQUIPMENT ANCHORAGE FORCES (LRFD) - CFCI



R1

Height, H = 84.0 in
Height to center of gravity, z = 42.0 in
Width, W = 20.0 in
Overturning Dimension, ω = 16.0 in
# of anchors in tension, $\#_{T,\omega}$ = 2
x = 10.0 in
x' = 8.0 in
Length, L = 24.8 in
Overturning Dimension, ℓ = 24.8 in
# of anchors in tension, $\#_{T,\ell}$ = 2
y = 12.4 in
y' = 12.4 in
R1
Weight, W_p = 750 lbs
# of anchors in shear, $\#_V$ = 4
Height of component with respect to grade, z = 0.00 ft
Average roof height, h = 1.00 ft

Seismic

Seismic design requirements for equipment are based on ASCE 7-16, Chapter 13.

COMPONENT AMPLIFICATION FACTOR
ASCE Section 13.5, 13.6 & ASCE Table 13.5-1, 13.6-1 — a_p = 2.5

COMPONENT RESPONSE MODIFICATION FACTOR
ASCE Section 13.5, 13.6 & ASCE Table 13.5-1, 13.6-1 — R_p = 2.5

DESIGN SPECTRAL RESPONSE ACCELERATION
S_{DS} = 0.498

COMPONENT IMPORTANCE FACTOR
ASCE Section 13.1.3 — I_p = 1.50

ATTACHMENT FACTOR IN CONCRETE OR MASONRY
ASCE Tables 13.5-1, 13.6-1 — Ω factor = 2.0

SEISMIC DESIGN FORCE

ASCE Section 13.3.1 & ASCE Equation 13.3-1	$F_p = 0.4 \cdot a_p \cdot S_{DS} \cdot W_p/(R_p/I_p)(1+2z/h)$	F_p = 0.299 Wp
ASCE Section 13.3.1 & ASCE Equation 13.3-2	$F_{p.max} = 1.6 \cdot S_{DS} \cdot I_p \cdot W_p$	$F_{p,max}$ = 1.195 Wp
ASCE Section 13.3.1 & ASCE Equation 13.3-3	$F_{p,min} = 0.3 \cdot S_{DS} \cdot I_p \cdot W_p$	$F_{p,min}$ = 0.224 Wp

SEISMIC DESIGN FORCES

ASCE Section 13.1.8 & 13.3.1	$F_{p,} = F_{p,govern}$	F_p = 0.299 Wp
ASCE Section 13.1.8 & 13.3.1	$F_{pv} = 0.2 \cdot S_{DS} \cdot W_p$	F_{pv} = 0.100 Wp

DESIGN FORCES

$F_{p,\Omega} = Fp \cdot W_p \cdot \Omega$ factor = 448 lbs
$OTM = z \cdot F_{p,\Omega}$ = 18824 lb-in
F_{pv} = 75 lbs
$DLRM = (0.9W_p - F_{pv}) \cdot x_{min}$ = 4802 lb-in

$$T = \frac{OTM - DLRM}{\omega \cdot \#_{T.\omega}} + \frac{0.3 \cdot OTM}{\ell \cdot \#_{T.\ell}}$$

T = 552 lbs

$$V = \frac{F_{p,\Omega} \cdot (2 \cdot y_{max} / L)}{\#_V}$$

V = 112 lbs

(V is approximate when number of anchors exceeds 4)

Use (4) - 1/2"Ø Hilti Kwik Bolt TZ2 w/ 2" Effective Embedment (Slab on Grade Condition, 4" min)

T_{ALL} = 1586 lbs — see HILTI PROFIS
V_{ALL} = 2249 lbs
UNITY CHECK = 0.40 OK

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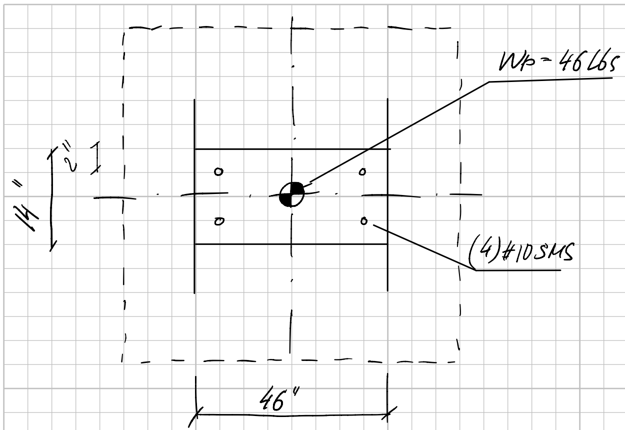


$Sd_s = 0.498$

$a_p = 1.0$

$R_p = 2.5$

$I_p = 1.0$

$W_t = 46\ lbs = F_{pv}$

$F_{pv} = 0.2 \times S_{ds} \times I_p \times W_p =$

$= 0.2 \times 0.498 \times 1.0 \times 46\ lbs = 4.6\ lbs$

$F_p = \frac{0.4 \times a_p \times S_{ds}}{R_p / I_p}(1 + 2\ z/h) =$

$= \frac{0.4 \times 1.0 \times 0.498}{2.5 / 1.0}(1+2) = 0.24\ wp =$

$= 0.24 \times 46\ lbs = 11\ lbs$

$F_{p\,min} = 0.3\ S_{ds} \times I_p \times W_p =$

$= 0.3 \times 0.498 \times 1.0 \times 46 = 6.9\ lbs$

$F_p = 11\ lbs$

Shear allowable strenght check

$V = (wt + 0.7 F_{pv}) / \#\ screw(4) =$

$= (46\ lbs + 0.7 \times 4.6\ lbs) / 4 = 12.3\ lbs$

V / V_{all}

$12.3\ lbs / 69\ lbs = 0.18$

Tension allowable strength check

$T = \frac{(W_t + 0.7 F_{pv}) \times 6.75" + 0.7\ F_p \times 2.5}{(14"-4") \times 2\ screw} =$

$= \frac{(46\ lbs + 0.7 \times 4.6\ lbs) \times 6.75" + 0.7 \times 11\ lbs \times 2.5}{20} = 17.6$

T / T_{all}

$17.6 / 109\ lbs = 0.16$

$V/V_{all} + T/T_{all} \le 1.0$

please see next pages for allowable strength

$0.18 + 0.16$

$0.34 \le 1.0$ GOOD

P16

c) The metal-critical joint may fail in one of two ways. Failure occurs when the resistance of the screw head to embedment is greater than the resistance of the metal to lateral and/or withdrawal load, and the screw tears away from the metal framing. Failure also occurs when thin metal in a metal-to-plywood joint crushes or tears away from the screw.

Tables 1 and 2 present ultimate lateral loads for wood- and sheet-metal-screw connections in plywood-and-metal joints. Loaded end distance in these tests was one inch. Plywood face grain was parallel to the load since this direction yields the lowest lateral loads when the joint is plywood-critical. All wood-screw specimens were tested with a 3/16-in.-thick steel side plate, and values should be modified if thinner steel is used.

TABLE 1

SCREWS: METAL-TO-PLYWOOD CONNECTIONS[a]

Depth of Threaded Penetration (in.)	Ultimate Lateral Load (lbf)[b]					
	Wood Screws			Sheet Metal Screws		
	#8	#10	#12	#8	#10	#12
1/2	415	(500)	590	465	(565)	670
5/8	–	–	–	500	(600)	705
3/4	–	–	–	590	(655)	715

(a) Plywood was C-D grade with exterior glue (all plies Group 1), face grain parallel to load. Side plate was 3/16"-thick steel.

(b) Values in parentheses are estimates based on other tests.

655/6=109

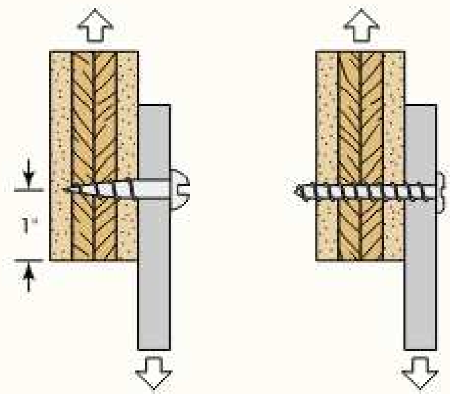


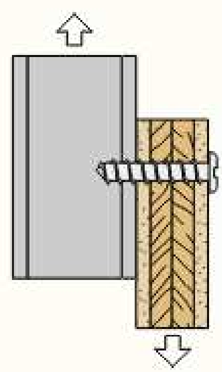
TABLE 2

SHEET METAL SCREWS: PLYWOOD-TO-METAL CONNECTIONS[a]

Framing	Plywood Thickness (in.)	Ultimate Lateral Load (lbf)[b]				
		Screw Size				1/4"-20 Self Tapping Screw
		#8	#10	#12	#14	
0.080-in. Aluminum	1/4	330	360	390	410	590
	1/2	630	850*	860	920	970
	3/4	910*	930*	1250	1330	1440
0.078-in. Galvanized Steel (14 gage)	1/4	360	380	400	410	650
	1/2	700*	890*	900	920	970
	3/4	700*	950*	1300*	1390*	1500

(a) Plywood was A-C EXT (all plies Group 1), face grain parallel to load.

(b) Loads denoted by an asterisk(*) were limited by screw-to-framing strength; others were limited by plywood strength.



P17

Withdrawal:

Tables 3 and 4 present average ultimate withdrawal loads for wood and sheet metal screws in plywood-and-metal joints, based on analysis of test results. Wood screws have a tapered shank and are threaded for only 2/3 of their length. Sheet metal screws typically have higher ultimate load than wood screws in the smaller gages, because of their uniform shank diameter and full-length thread. The difference is not as apparent in the larger gages and lengths because the taper is not as significant.

Values shown in Table 3 for wood screws are based on 1/4-in. protrusion of the wood screw from the back of the panel. This was to assure measurable length of thread embedment in the wood, since the tip of the tapered wood screw may be smaller than the pilot hole. This was not a factor for sheet metal screws due to their uniform shanks.

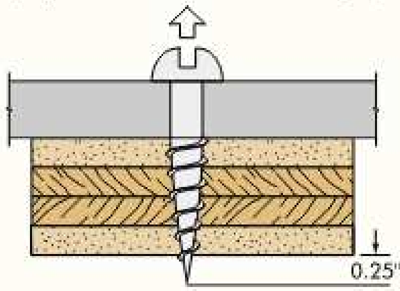
TABLE 3

WOOD AND SHEET METAL SCREWS: METAL-TO-PLYWOOD CONNECTIONS[a]

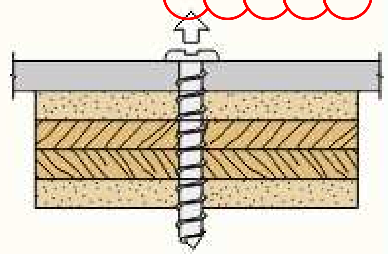
Depth of Threaded Penetration (in.)	Average Ultimate Withdrawal Load (lbf)					
	Screw Size					
	#6	#8	#10	#12	#14	#16
3/8	150	180	205	–	–	–
1/2	200	240	275	315	350	–
5/8	250	295	345	390	440	–
3/4	300	355	415	470	525	–
1	–	–	–	625	700	775
1-1/8	–	–	–	705	790	875
2-1/4	–	–	–	–	1580	–

(a) Plywood was C-D grade with exterior glue (all plies Group 1).

415/6=69



Wood Screw



Sheet Metal Screw

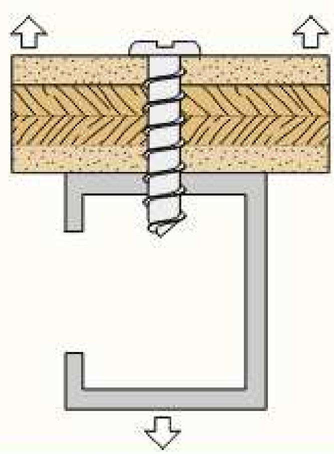
TABLE 4

SHEET METAL SCREWS: PLYWOOD-TO-METAL CONNECTIONS[a]

Framing	Plywood Thickness (in.)	Ultimate Withdrawal Load (lbf)[b]				
		Screw Size				1/4"-20 Self Tapping Screw
		#8	#10	#12	#14	
0.080-in. Aluminum	1/4	130	150	170	180	220
	1/2	350	470	500	520	500
	3/4	660	680	790	850*	790*
0.078-in. Galvanized Steel (14 gage)	1/4	130	150	170	180	220
	1/2	350	470	500	520	500
	3/4	660	680	800	900	850

(a) Plywood was A-C EXT (all plies Group 1).

(b) Loads denoted by an asterisk(*) were limited by screw-to-framing strength; others were limited by plywood strength.



Adjustment for Species Other Than Group 1:

All the ultimate loads presented in Tables 1 through 4 are based on plywood panels of all-Group 1 construction. For plywood panels of other species groups, the ultimate loads in these tables must be adjusted by correction factors presented in Table 5. Correction factors apply for both lateral and withdrawal loading. The adjustment factor for the highest numbered species group present in any veneer should be used.

Fastening Into Plywood Panel Edges

Fastening into plywood panel edges is not normally recommended. For some purposes, however, edge fastening may be necessary. Table 6 presents ultimate lateral and withdrawal loads for various sizes of wood screws in this application.

TABLE 5

LOAD ADJUSTMENT FOR SCREWS INTO PLYWOOD FOR SPECIES GROUPS NOTED(a)(b)

Types of Loading	All-Group 1	All-Group 2	All-Group 3, 4, 5
Lateral	100%	78%	78%
Withdrawal	100%	60%	47%

(a) Adjustments based on the species groups for plywood shown in Voluntary Product Standard PS 1 and the equations in U.S. Agricultural Handbook No. 72.

(b) Face, back, and core veneer must be of the same species group. When species group is unknown, assume all-Group 4.

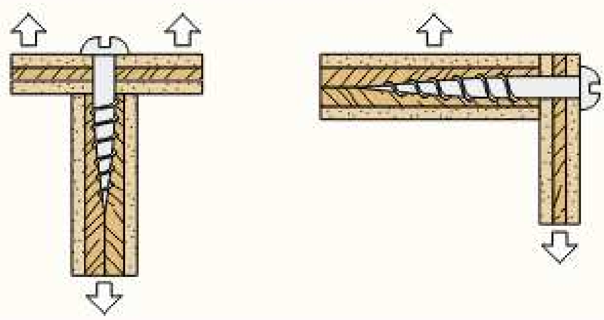
TABLE 6

WOOD SCREWS: PLYWOOD-TO-PLYWOOD EDGE CONNECTIONS(a)

Depth of Threaded Penetration	Ultimate Lateral Load (lbf)(b)			Ultimate Withdrawal Load (lbf)(b)		
(in.)	#8	#10	#12	#8	#10	#12
1	180	(185)	195	360	(405)	450
1-1/2	180	(185)	195	410	(455)	500

(a) Plywood receiving screw thread was 3/4"-thick C-D grade with exterior glue (Group 2 inner plies).

(b) Values in parentheses are estimates based on other tests.



ESTIMATING ALLOWABLE DESIGN LOADS

It is the responsibility of the designer to select a working load suitable for the particular application. A high degree of variability is inherent in individual fastener test results. Therefore, for screws in withdrawal, a working load of about one-sixth of the ultimate load has traditionally been used for long-duration loads. For normal load duration, the long-term working load may be increased by 10 percent. Normal load duration contemplates fully stressing the connection for approximately ten years, either continuously or cumulatively.

For laterally loaded screws, a working load of normal duration may be approximated by dividing the tabulated ultimate load by 5 or 6. For practically all laterally loaded screw connections shown, the normal-duration working load will correspond to a joint slip of less than 0.01 inch.

Adjustments for shorter or longer duration of load apply to design values for mechanical fasteners where the strength of the wood (i.e., not the strength of the metal fastener) determines the load capacity. Adjustments of design values for varying durations of load and combinations of load should be in accordance with the current AF&PA *National Design Specification for Wood Construction.*