ZFA STRUCTURAL ENGINEERS

san francisco
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SLAC Optics Lab
Structural Calculations

Menlo Park, California

ZFA Project Number: 25649

SLAC F&O Engineers Drawing Review

☑ REVIEWED ☐ REVISE AS NOTED ☐ REVISE AND RESUBMIT

REVIEWER/S: Jahan Danesh, Amy Ancheta

DATE: 05/22/26 SIGNATURE: Amy Ancheta

The above items have been reviewed for general compliance with project requirements and industry standards within the reviewer's area of expertise. This review does not constitute final approval or acceptance of the submittal. The contractor remains responsible for ensuring accuracy of dimensions, field conditions, and coordination with other trades.

Revision F Calculations

May 15, 2026

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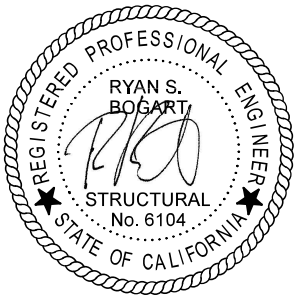
Prepared For:

Vektor Engineering and Consulting

San Ramon, California



Prepared By:

Dalton Muck, Designer

Ryan S. Bogart, SE, Principal-in-Charge

San Francisco, California

SLAC BUILDING INSPECTION OFFICE

Construction Authorization (IFC)

BY: cnadler

DATE: 5/31/2026 PRS#: 26-118

Revised Authorization ☑ No ☐ Yes

These plans have been reviewed for compliance with applicable building codes and standards, and ESH program requirements.

The authorization of these plans shall not be construed to be a permit or an approval for violation of any **applicable codes, standards, or ESH programs.**

THESE PLANS SHALL BE KEPT ON THE JOB SITE FOR ALL CONSTRUCTION INSPECTIONS.

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STRUCTURAL NARRATIVE

The project is located at 2575 Sand Hill Road, Menlo Park, CA. It consists of constructing a new optics lab, including modifications to nonstructural partition walls and anchorage/bracing of equipment and distribution systems. No modifications to the existing building structure. Design is in accordance with CBC 2025 and ASCE 7.

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Design Criteria

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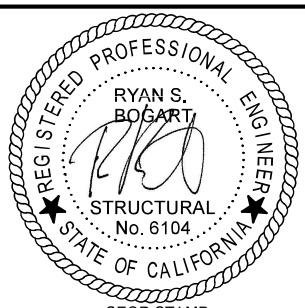
DETAILED DESIGN CRITERIA

BUILDING CODE

Governing Code:	2025 California Building Code
Authority Having Jurisdiction:	City of Menlo Park
Local Codes or Amendments:	2025 Menlo Park Building Code Amendments



SEOR STAMP

BUILDING SYSTEM DESCRIPTION

Mean Building Height:	29 ft
Building Use:	Research Lab
Gravity System:	Steel framing
Diaphragm System:	Concrete over metal deck
Foundation System:	Concrete Footings

GENERAL DESIGN CRITERIA

Latitude =	37.4198 deg		
Longitude =	-122.2055 deg		
Building Risk Category:	II	Research Lab	*Table 1.5-1*
Diaphragm =	Rigid Diaphragm	Concrete over metal deck	

SEISMIC DESIGN PARAMETERS

ASCE 7-22 Reference UNO:

I_E =	1.00	Importance Factor, Seismic	*Table 1.5-2*
I_P =	1.00	Importance Factor, Nonstructural Components	*Section 13.1.3*
Soil Site Class =	Default	Per Geotech Report, Site Class C, CD, or D otherwise	*Section 11.4.2*
S_{DS} =	1.800 g	Design spectral response acceleration parameter	*ASCE 7 Hazards*
Seismic Design Category:	E		*ASCE 7 Hazards*

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MATERIAL STRENGTH AND SPECIFICATIONS

COLD FORMED METAL FRAMING:

33 & 43 mils (20 & 18 ga), f_y =	33 ksi	ASTM A1003, Grade ST33H or ST33L
54, 68, & 97 mils (16, 14, & 12 ga), f_y =	50 ksi	ASTM A1003, Grade ST50H or ST50L
CFS Weld, F_{EXX} =	60 ksi	CFS Weld Strength

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GRAVITY / SEISMIC FLAT WEIGHT TAKEOFF (PSF)

Typical CFS Ceiling

CBC Live Load Category: - [Table 1607.1]

Material	Thickness (in)	Deck	Joists	Girders	Columns	Seismic
Framing: 400S162-43 @ 16"oc		1.0	1.0	1.0	1.0	1.0
Ceiling: 1/2 in. Gypsum Board		2.2	2.2	2.2	2.2	2.2
MEP: Typical		2.0	2.0	2.0	2.0	2.0
Miscellaneous		1.5	1.5	1.5	1.5	1.5
Dead Load		**6.7**	**6.7**	**6.7**	**6.7**	**6.7**
Total Load		**6.7**	**6.7**	**6.7**	**6.7**	**6.7**

GRAVITY / SEISMIC WALL WEIGHT TAKEOFF (PSF)

CFS Interior Wall

Material	Weight	
	Gravity	Seismic
General: Interior CFS Wall	10.0	10.0
TOTAL	**10.0**	**10.0**

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Gravity & Out-of-Plane Analysis & Design

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CBC 2025, ASCE 7-22 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

DSA Project?	NO			
Soil Site Class	Default	Section 11.4.2		
Response Spectral Acc. (0.1 sec) S_{DS} =	1.800g	= 180.0%g	(at 5% Damped Design)	ASCE 7 Hazards
Building Risk Category:	II			Table 1.5-2
I_E, Seismic =	1.0			
Seismic Design Category	E			ASCE 7 Hazards

13.3 Seismic Demands on Nonstructural Components

Component Name: Typical Interior Wall
Component Description: Interior nonstructural walls and partitions- Light frame > 9ft in height

Component Location: At or below grade

$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_u}\right]\left[\frac{C_{AR}}{R_{po}}\right]$ S_{DS} = 1.800 (13.3-1)

C_{AR} = 1.4 R_{po} = 1.5 T-13.5-1 or 13.6-1
Ω_{op} = 2.00 T-13.5-1 or 13.6-1
I_p = 1.0 13.1.3
R_μ = 1.00 Bldg R = 6.5 Bldg Ω_o = 2.5
z = 0 ft h = 29 ft
T_a = 0.458 Sec
H_f = 1.0 a_1 = 2.18 a_2 = 0.24 (13.3-4)
H_f = 1.0 (13.3-5)

F_p = 0.672Wp
Max F_p = $1.6 S_{DS} I_p W_p$ = 2.880Wp (13.3-2)
Min F_p = $0.3 S_{DS} I_p W_p$ = 0.540Wp (13.3-3)
F_p = 0.672 Wp
$F_p \Omega_{op}$ = 1.344 Wp T-13.5-1 footnote b or 13.6-1 footnote c

W_p = 10.0 psf
F_p = **6.7** psf (LRFD)
$F_p \Omega_o$ = **13.4** psf (LRFD)

F_p = **4.7** psf (ASD) < 5 psf
$F_p \Omega_o$ = **9.4** psf (ASD)

5psf Live Load is greater than Out-of-Plane Seismic load. Use 5psf in CFS typical framing design

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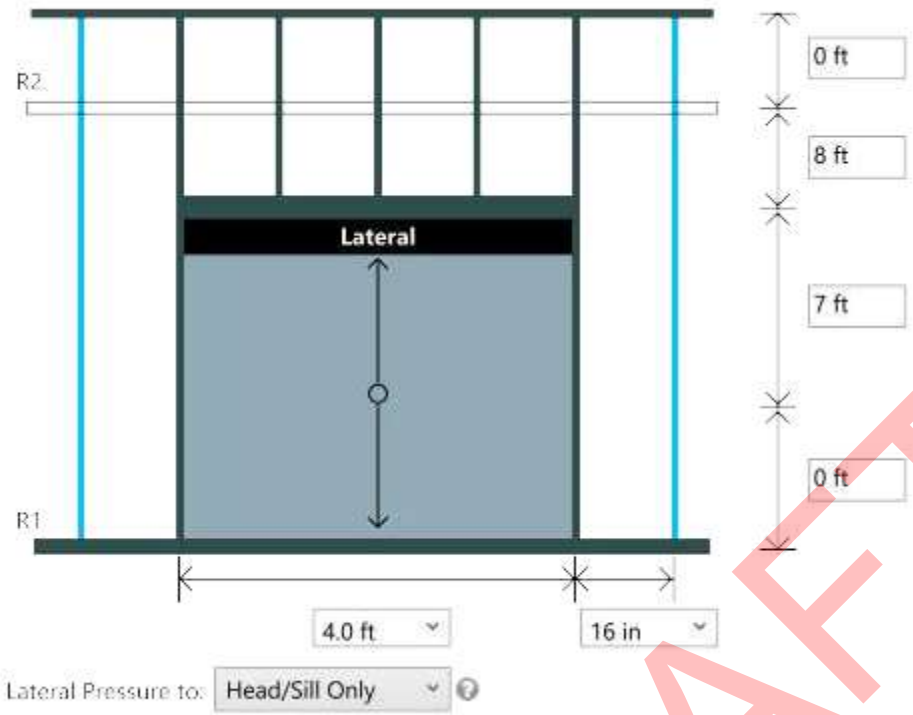

Design Loads

Wall Lateral Pressure :	**5 psf**
Parapet Lateral Pressure :	
RO Lateral Pressure :	**Head/Sill Only**
Lateral element force multiplier	
Strength :	**1.0**
Deflection :	**0.7**
Header:	**Single Member**
Gravity Load at Header:	**10 psf**

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Brace Settings

Component(s)	Members(s)	Flexural Bracing	Axial KyLy	Axial KtLt	Distortional K-Phi(lb-in/in)	Distortional Lm	Interconnection Spacing
Wall Studs	400S162-43(33), Single@16 in o/c	60 in	60 in	60 in	0	None	N/A
Jamb Studs	400S162-43(33), Single	60 in	60 in	60 in	0	None	N/A
Vertical Header	400T200-68(50), Y-Y Axis	Full	N/A	N/A	0	None	N/A
Lateral Header	400T200-68(50), Single	Full	N/A	N/A	0	None	N/A

Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max KL/r	Max. Moment (ft-lb)	Max. Shear (lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Wall Studs	400S162-43(33), Single@16 in o/c	0.0	N/A	187.5	50.0	50.0	50.0
Jamb Studs	400S162-43(33), Single	160.0	114	373.3	65.0	100.0	60.0
Vertical Header	400T200-68(50), Y-Y Axis	N/A	N/A	160.0	160.0	N/A	160.0
Lateral Header	400T200-68(50), Single	N/A	N/A	75.0	75.0	N/A	75.0

Design Results

		Deflection		Interaction				
Component(s)	Members(s)	Span	Parapet	Defl.	A + M	V + M	Web Stiffners	Design OK
Wall Studs	400S162-43(33), Single@16 in o/c	L/891	L/0	-	0.30	0.03	No	Yes
Jamb Studs	400S162-43(33), Single	L/526	L/0	-	0.70	0.55	No	Yes
Vertical Header	400T200-68(50), Y-Y Axis	L/465	NA	-	0.76	0.76	No	Yes
Lateral Header	400T200-68(50), Single	L/13223	NA	-	0.05	0.05	No	Yes
Combined Header				-	0.81	0		

Simpson Strong-Tie® Connectors @ Studs

	Connector	Anchor

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Interaction	Interaction
R2	50.00	0.00	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	8.20 %	4.48 %
R1	50.00	200.00	400T150-43 (33) & (1) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	10.70 %	17.45 %

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	60.00	0.00	400SLT250-54 (50) & (1) .157", 1.25" embed SST PDPA/PDPAT to 2500 nw concrete	18.16 %	26.04 %
R1	100.00	306.67	400T125-33 (33) & (2) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	46.34 %	22.71 %

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Studs

Span/Parapet	Bracing Length(in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Span	60	N/A	0.0	No Soln	No Soln	No Soln	No Soln	No Soln	No Soln

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/Parapet	Bracing Length(in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Span	60	3	0.0	No Soln	No Soln	No Soln	No Soln	No Soln	No Soln

Notes:

1) Values in parentheses are stress ratios.
2) Bridging connectors are not designed for back-back, box, or built-up sections.
3) Reference www.strongtie.com for latest load data, important information, and general notes.
4) CFS Designer will not select bridging connectors unless all flexural and axial bracing settings are the same.
5) If the bracing length is larger than the span length, bridging connectors are not designed.

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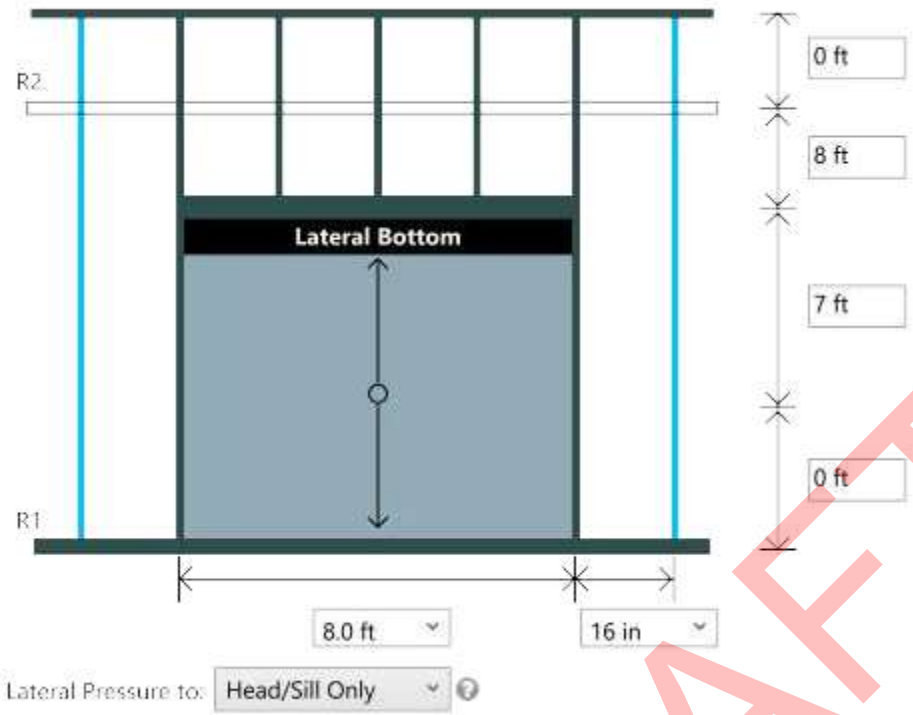

Design Loads

Wall Lateral Pressure :	**5 psf**
Parapet Lateral Pressure :	
RO Lateral Pressure :	**Head/Sill Only**
Lateral element force multiplier	
Strength :	**1.0**
Deflection :	**0.7**
Header:	**Box (lateral top, bottom)**
Gravity Load at Header:	**10 psf**

Back-to-Back Member L/6 Interconnection Spacing per AISI S100 I1.1

Member	Span	Cantilever
Jamb Studs	30.0 in	0.0 in

See AISI S100 I1.1 for Add'nl Requirements

Brace Settings

Component(s)	Members(s)	Flexural Bracing	Axial KyLy	Axial KtLt	Distortional K-Phi(lb-in/in)	Distortional Lm	Interconnection Spacing
Wall Studs	400S162-43(33), Single@16 in o/c	60 in	60 in	60 in	0	None	N/A
Jamb Studs	400S162-43(33), Back-To-Back	60 in	60 in	60 in	0	None	12 in
Vertical Header	400S162-33(33), Boxed	Full	N/A	N/A	0	None	N/A
Lat. Top Head	400T150-43(33), Single	Full	N/A	N/A	0	None	N/A
Lat. Bottom Head	400T150-43(33), Single	Full	N/A	N/A	0	None	N/A

Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max KL/r	Max. Moment (ft-lb)	Max. Shear (lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Wall Studs	400S162-43(33), Single@16 in o/c	0.0	N/A	187.5	50.0	50.0	50.0
Jamb Studs	400S162-43(33), Back-To-Back	320.0	114	653.3	105.0	175.0	95.0
Vertical Header	400S162-33(33), Boxed	N/A	N/A	640.0	320.0	N/A	320.0
Lat. Top Head	400T150-43(33), Single	N/A	N/A	160.0	80.0	N/A	80.0
Lat. Bottom Head	400T150-43(33), Single	N/A	N/A	140.0	70.0	N/A	70.0

Design Results

		Deflection		Interaction				
Component(s)	Members(s)	Span	Parapet	Defl.	A + M	V + M	Web Stiffners	Design OK
Wall Studs	400S162-43(33), Single@16 in o/c	L/891	L/0	-	0.30	0.06	No	Yes
Jamb Studs	400S162-43(33), Back-To-Back	L/618	L/0	-	0.55	0.48	No	Yes
Vertical Header	400S162-33(33), Boxed	L/532	NA	-	0.65	0.65	R1	Yes
Lat. Top Head	400T150-43(33), Single	L/1579	NA	-	0.33	0.33	No	Yes
Lat. Bottom Head	400T150-43(33), Single	L/1804	NA	-	0.29	0.05	No	Yes

Simpson Strong-Tie® Connectors @ Studs

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Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	50.00	0.00	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	8.20 %	4.48 %
R1	50.00	200.00	400T125-33 (33) & (1) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	11.59 %	22.71 %

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	95.00	0.00	400SLT250-54 (50) & (1) .157", 1.25" embed SST PDPA/PDPAT to 2500 nw concrete	26.76 %	41.23 %
R1	175.00	466.67	400T150-43 (33) & (1) .157", 1.25" embed SST PDPA/PDPAT to 2500 nw concrete	35.22 %	61.09 %

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Studs

Span/Parapet	Bracing Length(in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Span	60	N/A	0.0	No Soln	No Soln	No Soln	No Soln	No Soln	No Soln

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/Parapet	Bracing Length(in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Span	60	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

1) Values in parentheses are stress ratios.
2) Bridging connectors are not designed for back-back, box, or built-up sections.
3) Reference www.strongtie.com for latest load data, important information, and general notes.
4) CFS Designer will not select bridging connectors unless all flexural and axial bracing settings are the same.
5) If the bracing length is larger than the span length, bridging connectors are not designed.

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Miscellaneous

Anchorage of a transformer on a steel base and a rail connection to the concrete over a metal deck supporting new shelving.

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Anchorage for Transformer and steel base (8/S1.00)

CBC 2025, ASCE 7-22, §13.3 Seismic Demands on Nonstructural Components

EQUIPMENT:

Description = Transformer to Base

Unit Weight, $1.1W_p$ = 700.0 lbs I_p = 1.0 13.1.3

Anchored to Concrete and using Ω_o for anchorage = NO Table 13.5-1 or Table 13.6-1 footnote C

Vibration Isolators with air gap > 0.25" = NO

Fp FORCE:

Component Location: Above grade plane

$$F_p = 0.4SDSIpWp\left[\frac{H_f}{R_u}\right]\left[\frac{C_{AR}}{R_{po}}\right]$$

S_{DS} = 1.800 (13.3-1)

C_{AR} = 1.0 R_{po} = 1.5 T-13.5-1 or 13.6-1

Ω_{op} = 2.00 T-13.5-1 or 13.6-1

I_E = 1.00

R_μ = 1.34 Bldg R = 3.25 Bldg Ω_o = 2

z/h = 1

H_f = 3.5 (13.3-5)

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F_p = 1.257Wp

Max F_p = $1.6S_{DS}I_pW_p$ = 2.880Wp (13.3-2)

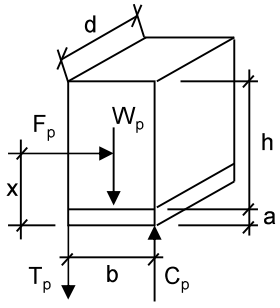
Min F_p = $0.3S_{DS}I_pW_p$ = 0.540Wp (13.3-3)

Fp = 1.257 Wp

Fp = 0.880 kips

OVERTURNING FORCES:

Method: LRFD



Distance between anchors, b = 15 in short side of MU

Distance between anchors, d = 28.5 in long side of MU

Height, h = 40 in

Frame base, a = 0 in

C.G., x = a + h/2 = 20 in

Fp = 0.88 kips

Mo = x*Fp = 18 kip-in overturning moment

Mrb(T) = Wp(0.9-0.2SDS)b/2 = 3 kip-in resisting moment in 'b' direction

Mrb(C) = Wp(1.2+0.2SDS)b/2 = 8 kip-in resisting moment in 'b' direction

Mrd(T) = Wp(0.9-0.2SDS)d/2 = 5 kip-in resisting moment in 'd' direction

Mrd(C) = Wp(1.2+0.2SDS)d/2 = 16 kip-in resisting moment in 'd' direction

Cb = (Mo+Mrb(C))/b/d = **0.72 klf** compression along 'b' side

Cd = (Mo+Mrd(C))/d/b = **0.93 klf** compression along 'd' side

ANCHORS:

Anchor Type = 1/2"Ø MB

N_b = 2 anchors along b side

N_d = 2 anchors along d side

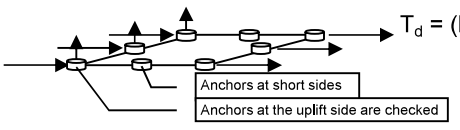
Total Anchors, N_{tot} = 4 = $2N_b + 2N_d - 4$

$T_b = (Mo-Mb(T))/b/N_d$ = 0.49 kips/anchor along b side

$T_d = (Mo-Mrd(T))/d/N_b$ = 0.21 kips/anchor along d side

$Tp = T_b + 0.3T_d$ = 0.56 kips/anchor at corner

$V = 1.30Fp/N_{tot}$ = 0.29 kips/anchor



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BOLT CAPACITY

Loading:

Design Methodology		LRFD	
Shear Demand	Vu =	0.3	k
Tension Demand	Tu =	0.6	k

Design Strength (§J3.6 & §J3.7): ϕ = 0.75

Bolt Description		A307	
Nominal Tensile Stress *(§Table J3.2)*	F_{nt} =	45	ksi
Nominal Shear Stress *(§Table J3.2)*	F_{nv} =	27	ksi
Bolt Diameter	d =	0.50	in
Bolt Area	A_b =	0.20	in^2

Design Shear Strength	**(§J3-1)**	**ϕ Fnv Ab =**	**4.0**	**k**
Shear Utilization Ratio		**DCR =**	**0.07**	

Required Shear Stress		frv =	1.5	ksi
Modified Tensile Stress (§J3-3a)	Fnt' = min[Fnt, 1.3 Fnt - (Fnt / ϕ Fnv) frv] =		45.0	ksi

Available Tensile Strength (§J3-2)	ϕ Fnt' Ab =	**6.6**	**k**
Tension Utilization Ratio	**DCR =**	**0.08**	

Design Bearing Strength (§J3.11): ϕ = 0.75

Minimum Tensile Strength of Connected Material	Fu =	58	ksi
Clear Distance from Hole to Hole or Edge	lc =	1.00	in
Thickness of Connected Material	t =	0.50	in

Bearing Condition: Deformation at bolt hole at service load is considered

Nominal Bearing Strength	Rn = min[1.2 lc t Fu, 2.4 d t Fu] =	34.8	k

Available Bearing Strength	**ϕ Rn =**	**26.1**	**k**
Bearing Utilization Ratio	**DCR =**	**0.01**	

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Transformer Base Post

Steel Column

Project File: SLAC B999 MFX.ec6

DESCRIPTION: Transformer Base Post

Code References

Governing Code : IBC 2018, CBC 2019
Referenced Design Standard(s) : AISC 360-22*
Load Combinations Used : ASCE 7-16 (1)

*User-selected, not referenced by : IBC 2018, CBC 2019

General Information

Steel Section Name : **HSS3-1/2x3-1/2x3/16**

Analysis Method :	Load Resistance Factor
Steel Stress Grade	
Fy : Steel Yield	36.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height	4.33 ft
Top & Bottom Fixity	Top Free, Bottom Fixed
Brace condition :	

Unbraced Length for buckling ABOUT X-X Axis = 4.33 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 4.33 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 35.290 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 4.330 ft, D = 0.1750 k
BENDING LOADS . . .
Lat. Point Load at 4.330 ft creating Mx-x, E = 0.290 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.1709** : 1

Load Combination	+1.560D+E
Location of max.above base	0.0 ft
At maximum location values are . . .	
Pu	0.3281 k
0.9 * Pn	67.131 k
Mu-x	-1.256 k-ft
0.9 * Mn-x :	7.452 k-ft
Mu-y	0.0 k-ft
0.9 * Mn-y :	7.452 k-ft

PASS Maximum Shear Stress Ratio = **0.01727** : 1

Load Combination	+1.560D+E
Location of max.above base	0.0 ft
At maximum location values are . . .	
Vu : Applied	0.290 k
Vn * Phi : Allowable	16.789 k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.290 k

Maximum Load Deflections . . .

Along Y-Y 0.1149 in at 4.330 ft above base
for load combination : E Only

Along X-X 0.0 in at 0.0 ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.004	PASS	0.00 ft	1.67	1.00	38.49	38.49	0.000	PASS	0.00 ft
+1.20D	0.004	PASS	0.00 ft	1.67	1.00	38.49	38.49	0.000	PASS	0.00 ft
+0.90D	0.003	PASS	0.00 ft	1.67	1.00	38.49	38.49	0.000	PASS	0.00 ft
+1.560D+E	0.171	PASS	0.00 ft	1.67	1.00	38.49	38.49	0.017	PASS	0.00 ft
+0.540D+E	0.169	PASS	0.00 ft	1.67	1.00	38.49	38.49	0.017	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial (k) @ Base	Rx (k) @ Base	Rx (k) @ Top	Ry (k) @ Base	Ry (k) @ Top	Mx (k-ft) @ Base	Mx (k-ft) @ Top	My (k-ft) @ Base	My (k-ft) @ Top
D Only	0.210								
+0.60D	0.126								
+D+0.70E	0.210			-0.203		0.879			
+D+0.5250E	0.210			-0.152		0.659			
+0.60D+0.70E	0.126			-0.203		0.879			
E Only				-0.290		1.256			

DRAFT

DESCRIPTION: Transformer Base Post

Extreme Reactions

Item	Extreme Value	Axial (k) @ Base	Rx (k) @ Base	Rx (k) @ Top	Ry (k) @ Base	Ry (k) @ Top	Mx (k-ft) @ Base	Mx (k-ft) @ Top	My (k-ft) @ Base	My (k-ft) @ Top
Axial, Base Max		0.210								
Axial, Base Min										
Rx, Base Max										
Rx, Base Min										
Ry, Base Max										
Ry, Base Min					-0.290					
Mx, Base Max							1.256			
Mx, Base Min										
My, Base Max										
My, Base Min										

Maximum Deflections for Load Combinations

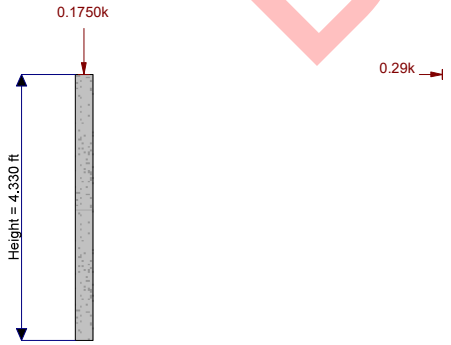
Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	0.0000 in	0.000 ft	0.080 in	4.330 ft
+D+0.5250E	0.0000 in	0.000 ft	0.060 in	4.330 ft
+0.60D+0.70E	0.0000 in	0.000 ft	0.080 in	4.330 ft
E Only	0.0000 in	0.000 ft	0.114 in	4.301 ft

Steel Section Properties : HSS3-1/2x3-1/2x3/16

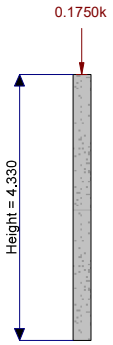
Depth	=	3.500 in	I xx	=	4.05 in^4	J	=	6.560 in^4
Design Thick	=	0.174 in	S xx	=	2.31 in^3			
Width	=	3.500 in	R xx	=	1.350 in			
Wall Thick	=	0.187 in	Zx	=	2.760 in^3			
Area	=	2.240 in^2	I yy	=	4.050 in^4	C	=	3.830 in^3
Weight	=	8.150 plf	S yy	=	2.310 in^3			
			R yy	=	1.350 in			
Ycg	=	0.000 in						

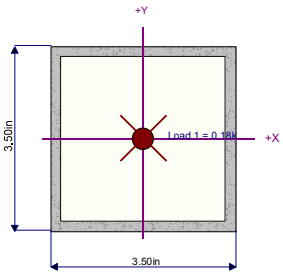
Sketches

My Loads - Looking along Y-Y Axis



Mx Loads - Looking along X-X Axis





DRAFT

CBC 2025, ASCE 7-22, §13.3 Seismic Demands on Nonstructural Components

EQUIPMENT:

Description = Transformer + Base to Conc o/ mtl deck

Unit Weight, $1.1W_p$ = 700.0 lbs I_p = 1.0 13.1.3

Anchored to Concrete and using Ω_o for anchorage = YES Table 13.5-1 or Table 13.6-1 footnote C

Vibration Isolators with air gap > 0.25" = NO

Fp FORCE:

Component Location: Above grade plane

$$F_p = 0.4 SDS I_p W_p \left[\frac{H_f}{R_u}\right]\left[\frac{C_{AR}}{R_{po}}\right]$$

S_{DS} = 1.800 (13.3-1)

C_{AR} = 1.0 R_{po} = 1.5 T-13.5-1 or 13.6-1

Ω_{op} = 2.00 T-13.5-1 or 13.6-1

I_E = 1.00

R_μ = 1.34 Bldg R = 3.25 Bldg Ω_o = 2

z/h = 1

H_f = 3.5 (13.3-5)

F_p = 1.257Wp

Max F_p = $1.6 S_{DS} I_p W_p$ = 2.880Wp (13.3-2)

Min F_p = $0.3 S_{DS} I_p W_p$ = 0.540Wp (13.3-3)

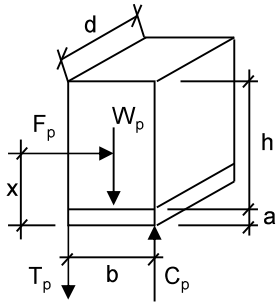
Ωo Fp = **2.513 Wp**

Ωo Fp = **1.759 kips**

SLAC BUILDING INSPECTION OFFICE
Construction Authorization (IFC)
BY: cnadler
DATE: 5/31/2026 PRS#: 26-118
Revised Authorization ☑ No ☐ Yes
These plans have been reviewed for compliance with applicable building codes and standards, and ESH program requirements.
The authorization of these plans shall not be construed to be a permit or an approval for violation of any applicable codes, standards, or ESH programs.
THESE PLANS SHALL BE KEPT ON THE JOB SITE FOR ALL CONSTRUCTION INSPECTIONS.

DRAFT

OVERTURNING FORCES:



Method: LRFD

Distance between anchors, b = 25 in short side of MU

Distance between anchors, d = 28 in long side of MU

Height, h = 40 in

Frame base, a = 52 in

C.G., x = a + h/2 = 72 in

Ωo Fp = 1.76 kips

Mo = x*Ωo Fp = 127 kip-in overturning moment

Mrb(T) = Wp(0.9-0.2SDS)b/2 = 5 kip-in resisting moment in 'b' direction

Mrb(C) = Wp(1.2+0.2SDS)b/2 = 14 kip-in resisting moment in 'b' direction

Mrd(T) = Wp(0.9-0.2SDS)d/2 = 5 kip-in resisting moment in 'd' direction

Mrd(C) = Wp(1.2+0.2SDS)d/2 = 15 kip-in resisting moment in 'd' direction

Cb = (Mo/Ωo+Mrb(C))/b/d = **1.32 klf** compression along 'b' side

Cd = (Mo/Ωo+Mrd(C))/d/b = **1.35 klf** compression along 'd' side

ANCHORS:

Anchor Type = 1/2"Ø Exp Anchor

N_b = 2 anchors along b side

N_d = 2 anchors along d side

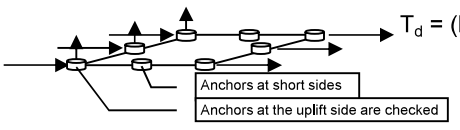
Total Anchors, N_{tot} = 4 = $2N_b + 2N_d - 4$

$T_b = (Mo - Mb(T))/b/N_d$ = 2.44 kips/anchor along b side

$T_d = (Mo - Mrd(T))/d/N_b$ = 2.17 kips/anchor along d side

$Tp = T_b + 0.3T_d$ = 3.09 kips/anchor at corner

$V = 1.30Fp/N_{tot}$ = 0.57 kips/anchor



Transformer Base Anchorage to Conc o/ Metal Deck

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Transformer Anchorage	Date:	2/2/2026
Fastening point:			

Specifier's comments:

1 Input data

Anchor type and diameter:	**Kwik Bolt TZ2 - CS 1/2 (2) hnom2**
Item number:	2210254 KB-TZ2 1/2x3 3/4
Specification text:	Hilti ∅ 1/2 in Kwik Bolt TZ2 - CS with 2.5 in nominal embedment depth per ICC-ES ESR-4266 , Hammer drill bit installation per MPII
Effective embedment depth:	$h_{ef,act}$ = 2.000 in., h_{nom} = 2.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-4266
Issued I Valid:	10/1/2024 | 12/1/2025
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.375 in.
Anchor plate[R]:	l_x x l_y x t = 8.000 in. x 8.000 in. x 0.375 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC), HSS3-1/2X3-1/2X.125; (L x W x T) = 3.500 in. x 3.500 in. x 0.125 in.
Base material:	cracked lightweight concrete, 3000, f_c' = 3,000 psi; h = 4.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d)) Shear load: yes (17.10.6.3 (c))

[R] - The anchor calculation is based on a rigid anchor plate assumption.

DRAFT

SLAC BUILDING INSPECTION OFFICE

Construction Authorization (IFC)

BY: cnadler

DATE: 5/31/2026 **PRS#:** 26-118

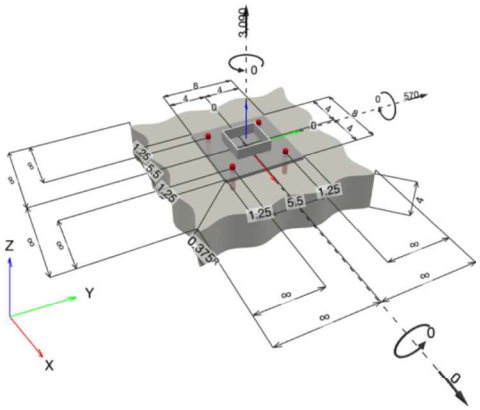
Revised Authorization ☑ No ☐ Yes

These plans have been reviewed for compliance with applicable building codes and standards, and ESH program requirements.

The authorization of these plans shall not be construed to be a permit or an approval for violation of any **applicable codes, standards, or ESH programs.**

THESE PLANS SHALL BE KEPT ON THE JOB SITE FOR ALL CONSTRUCTION INSPECTIONS.

Geometry [in.] & Loading [lb, in.lb]



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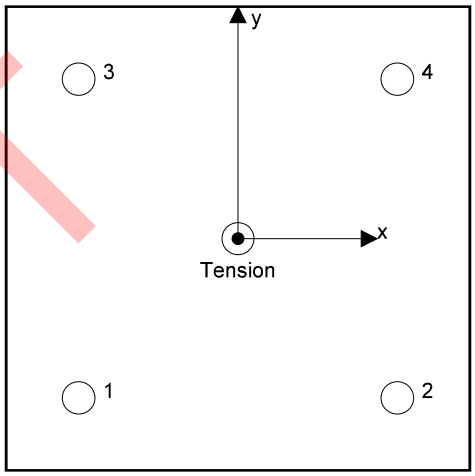
Design:	Transformer Anchorage	Date:	2/2/2026
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 3,090; V_x = 0; V_y = 570; M_x = 0; M_y = 0; M_z = 0;	yes	89

2 Load case/Resulting anchor forces



Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	772	143	0	143
2	772	143	0	143
3	772	143	0	143
4	772	143	0	143

Max. concrete compressive strain: - [‰]
Max. concrete compressive stress: - [psi]
Resulting tension force in (x/y)=(0.000/0.000): 3,090 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	772	8,433	10	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	3,090	3,496	89	OK

* highest loaded anchor **anchor group (anchors in tension)

DRAFT

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Transformer Anchorage	Date:	2/2/2026
Fastening point:			

3.1 Steel Strength

N_{sa} = ESR value — refer to ICC-ES ESR-4266
$\phi\ N_{sa} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.10	114,004

Calculations

N_{sa} [lb]
11,244

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
11,244	0.750	8,433	772

3.2 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
2.000	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
5.500	21	0.600	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
132.25	36.00	1.000	1.000	1.000	1.000	1,952

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
7,171	0.650	0.750	1.000	3,496	3,090

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Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Transformer Anchorage	Date:	2/2/2026
Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	143	3,599	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	570	5,020	12	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-4266
ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.10	114,004	1.000

Calculations

$V_{sa,eq}$ [lb]
5,537

Results

$V_{sa,eq}$ [lb]	ϕ steel	ϕ $V_{sa,eq}$ [lb]	V_{ua} [lb]
5,537	0.650	3,599	143

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Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Transformer Anchorage	Date:	2/2/2026
Fastening point:			

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\Psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\Psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\Psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
1	2.000	0.000	0.000	∞

$\Psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	5.500	21	0.600	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\Psi_{ec1,N}$	$\Psi_{ec2,N}$	$\Psi_{ed,N}$	$\Psi_{cp,N}$	N_b [lb]
132.25	36.00	1.000	1.000	1.000	1.000	1,952

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕV_{cpg} [lb]	V_{ua} [lb]
7,171	0.700	1.000	1.000	5,020	570

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.884	0.114	1.000	84	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Transformer Anchorage	Date:	2/2/2026
Fastening point:			

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- 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.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- "An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-19, Chapter 17, Section 17.10.5.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.10.5.3 (b), Section 17.10.5.3 (c), or Section 17.10.5.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.10.6.3 (a), Section 17.10.6.3 (b), or Section 17.10.6.3 (c)."
- Section 17.10.5.3 (b) / Section 17.10.6.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.10.5.3 (c) / Section 17.10.6.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.10.5.3 (d) / Section 17.10.6.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

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Fastening meets the design criteria!

PROFIS Engineering (c) 2003-2026 Hilti AG, FL-9494 Schaan Hilti is a registered Trademark of Hilti AG, Schaan

TABLE 5—HILTI KH-EZ, KH-EZ P, KH-EZ PM, KH-EZ PL, KH-EZ C AND KH-EZ CRC TENSION AND SHEAR DESIGN DATA FOR INSTALLATION IN THE UNDERSIDE OF CONCRETE-FILLED PROFILE STEEL DECK ASSEMBLIES[1,5,6,7]

Characteristic	Symbol	Units	Lower Flute											Upper Flute				
			Anchor Diameter															
			1/4		3/8			1/2			5/8		3/4	1/4		3/8		1/2
Head Style and Coating	-	-	Hex, P, PM, PL, C Head		Hex, C Head	Hex Head (including CRC)								Same as previous				
Embedment	h_{nom}	in.	1 5/8	2 1/2	1 5/8	2 1/2	3 1/4	2 1/4	3	4 1/4	3 1/4	5	4	1 5/8	2 1/2	1 5/8	2 1/2	2 1/4
		(mm)	(41)	(64)	(41)	(64)	(83)	(57)	(76)	(108)	(83)	(127)	(102)	(41)	(64)	(41)	(64)	(57)
Minimum Hole Depth	h_o	in.	2	2 7/8	1 7/8	2 3/4	3 1/2	2 5/8	3 3/8	4 5/8	3 5/8	5 3/						2 5/8
		(mm)	(51)	(73)	(47)	(70)	(83)	(67)	(86)	(117)	(92)	(13						(67)
Effective Embedment Depth	h_{ef}	in.	1.18	1.92	1.11	1.86	2.50	1.52	2.16	3.22	2.39	3.88	2.92	1.18	1.92	1.11	1.86	1.52
		(mm)	(30)	(49)	(28)	(47)	(64)	(39)	(55)	(82)	(61)	(99)	(74)	(30)	(49)	(28)	(47)	(39)
Pullout Resistance, (uncracked concrete)[2]	$N_{p,deck,uncr}$	lbf.	1,210	1,875	1,300	2,240	3,920	1,305	3,060	5,360	4,180	9,495	4,180	1,490	1,960	1,490	2,920	1,395
		(kN)	(5.4)	(8.3)	(5.8)	(10.0)	(17.4)	(5.8)	(13.6)								(13.0)	(6.2)
Pullout Resistance (cracked conc. / seismic loads)[3]	$N_{p,deck,cr}$	lbf.	620	930	810	1,590	2,780	820	1,930	3,375	2,630	5,980	2,630	760	975	1,185	2,070	985
		(kN)	(2.8)	(4.1)	(3.6)	(7.1)	(12.4)	(3.6)	(8.6)	(15.0)	(11.7)	(26.6)	(11.7)	(3.4)	(4.3)	(5.3)	(9.2)	(4.4)
Steel Strength in Shear[4]	$V_{sa,deck}$	lbf.	1,205	2,210	1,510	1,510	3,605	1,605	2,920	3,590	3,470	4,190	3,760	1,205	3,265	3,670	6,090	7,850
		(kN)	(5.4)	(9.8)	(6.7)	(6.7)	(16.0)	(7.1)	(13.0)	(16.0)	(15.4)	(18.6)	(16.7)	(5.4)	(14.5)	(16.3)	(27.1)	(34.9)
Steel Strength in Shear, Seismic	$V_{sa,deck,eq}$	lbf.	905	1,990	905	905	2,165	965	1,750	2,155	2,080	2,515	2,610	1,080	2,940	3,670	3,650	4,710
		(kN)	(4.0)	(8.9)	(4.0)	(4.0)	(9.6)	(4.3)	(7.8)	(9.6)	(9.3)	(11.2)	(11.6)	(4.8)	(13.1)	(16.3)	(16.2)	(21.0)

Anchorage for Cabinet Bracket (9/S1.00)

DCR = 600#/(2)1,590# = 19% OK

For **SI:** 1 inch = 25.4 mm, 1 ft-lbf = 1.356 N-m, 1 psi = 6.89 kPa, 1 in^2 = 645 mm^2, 1 lb/in = 0.175 N/mm.

[1]Installation must comply with Sections 4.1.10 and 4.3 and Figure 12 of this report.
[2]The values listed must be used in accordance with Section 4.1.4 of this report.
[3]The values listed must be used in accordance with Section 4.1.4 and 4.1.8.2 of this report.
[4]The values listed must be used in accordance with Section 4.1.5 and 4.1.8.3 of this report.
[5]The values for ϕ_p in tension and the values for ϕ_{sa} in shear can be found in Table 3 of this report.
[6]For the 1/4-inch-diameter (KH-EZ) at 2 1/2-inch nominal embedment and the 3/8-inch- through 3/4-inch-diameter anchors the characteristic pullout resistance for concrete compressive strengths greater than 3,000 psi may be increased by multiplying the value in the table by $(f'_c/3{,}000)^{1/2}$ for psi or $(f'_c/20.7)^{1/2}$ for MPa.
[7]For the 1/4-inch-diameter anchors (KH-EZ and KH-EZ P, PM, PL) at 1 5/8-inch nominal embedment characteristic pullout resistance for concrete compressive strengths greater than 3,000 psi may be increased by multiplying the value in the table by $(f'_c/3{,}000)^{0.3}$ for psi or $(f'_c/20.7)^{0.3}$ for MPa.

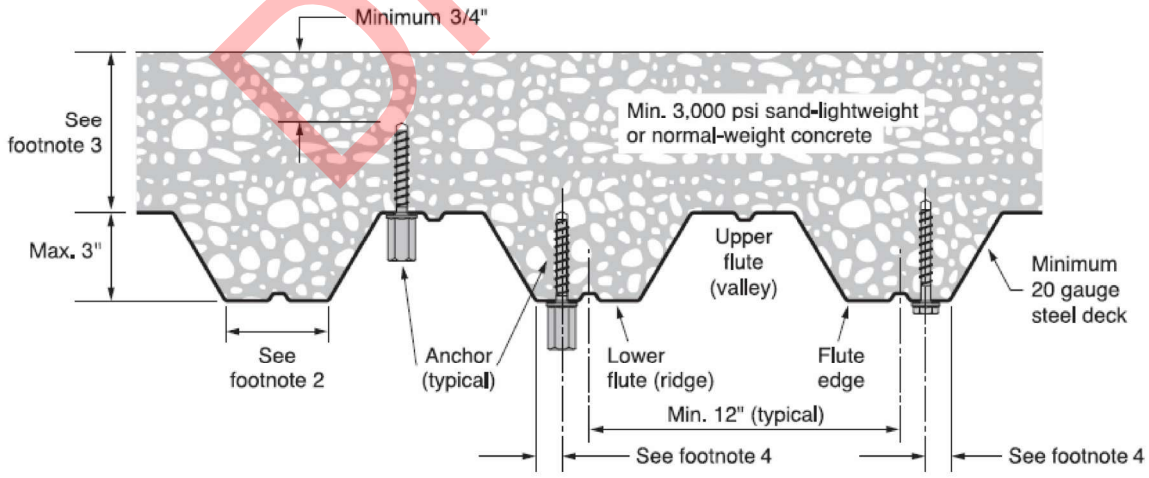

FIGURE 12—INSTALLATION OF KH-EZ, KH-EZ CRC AND KH-EZ I IN SOFFIT OF CONCRETE OVER STEEL DECK FLOOR AND ROOF ASSEMBLIES[1]

[1]Anchors may be placed in the upper or lower flute of the steel deck profile provided the minimum hole clearance is satisfied. Anchors in the lower flute may be installed with a maximum 1-inch offset in either direction from the center of the flute. The offset distance may be increased proportionally for profiles with lower flute widths greater than those shown provided the minimum lower flute edge distance is also satisfied.
[2]Minimum flute width for 1/4-inch diameter KH-EZ and KH-EZ I and 3/8-inch diameter KH-EZ I is 3 7/8 inches. Minimum flute width for KH-EZ 3/8-, 1/2-, 5/8- and 3/4-inch diameters is 4 1/2 inches.
[3]Minimum concrete thickness above upper flute for 1/4-inch diameter KH-EZ and 3/8-inch KH-EZ I is 2 1/2 inches. Minimum concrete thickness above upper flute for KH-EZ 3/8-, 1/2-, 5/8- and 3/4-inch diameter is 3-1/4 inches.
[4]Minimum distance from edge of flute to centerline of anchor for KH-EZ and KH-EZ I 1/4-inch diameter is 1-inch. Minimum distance from edge of flute to centerline of anchor for KH-EZ 3/8-, 1/2-, 5/8- and 3/4-inch diameter is 1 1/4 inches.

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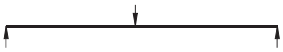
P1000 - Beam Loading

CHECK UNISRTRUT SUPPORTING HANGER

Span In	Max. Allowable Uniform Load Lbs	Defl. at Uniform Load In	Uniform Loading at Deflection Span/180 Lbs	Span/240 Lbs	Span/360 Lbs
24	1,690	0.06	1,690	1,690	1,690
36	1,130	0.13	1,130	1,130	900
48	850	0.22	850	760	500
60	680	0.35	650	480	320
72	560	0.50	450	340	220
84	480	0.68	330	250	160

Load and Support Condition		Load Factor	Deflection Factor
1. Simple Beam, Uniform Load		**1.00**	**1.00**
2. Simple Beam, Concentrated Load at Center		**.50**	**.80**

DCR = 300#/845# = (0.5)1,690# = 0.3 OK

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SLAC BUILDING INSPECTION OFFICE

Construction Authorization (IFC)

BY: cnadler

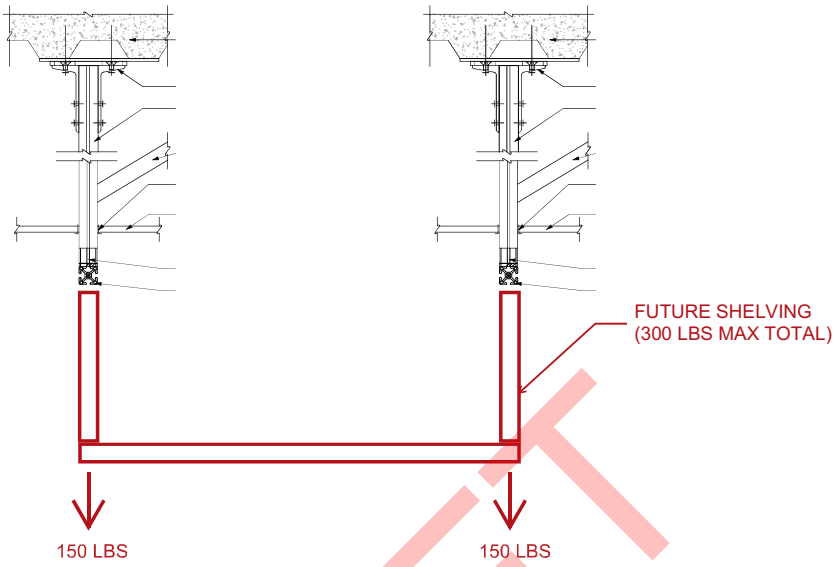
DATE: 5/31/2026 **PRS#:** 26-118

Revised Authorization ☑ No ☐ Yes

These plans have been reviewed for compliance with applicable building codes and standards, and ESH program requirements.

The authorization of these plans shall not be construed to be a permit or an approval for violation of any **applicable codes, standards, or ESH programs.**

THESE PLANS SHALL BE KEPT ON THE JOB SITE FOR ALL CONSTRUCTION INSPECTIONS.



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ASSUME POINT LOAD AT MID-SPAN.

300 LBS SPLIT BETWEEN (2) STRUTS.

150 LBS EACH.

8020 40-4040 STRUT (DEPTH = 40 mm).

SPANNING 4'-0"

MODULUS OF ELASTICITY, E = 10,000 KSI

MOMENT OF INERTIA, I = 0.331 in^4

YIELD STRENGTH, Fy = 25 KSI MIN.

SECTION MODULUS, S = 0.42 in^3

CHECK STRENGTH:

DEMAND: $M = PL/4 = 150\ lb(48") / 4 = 1.8k"$

CAPACITY: $M/\Omega = FyS/\Omega = \frac{(25\ ksi)(0.42\ in^3)}{1.67}$

$= 6.3k" > 1.8k"$

CHECK DEFLECTION:

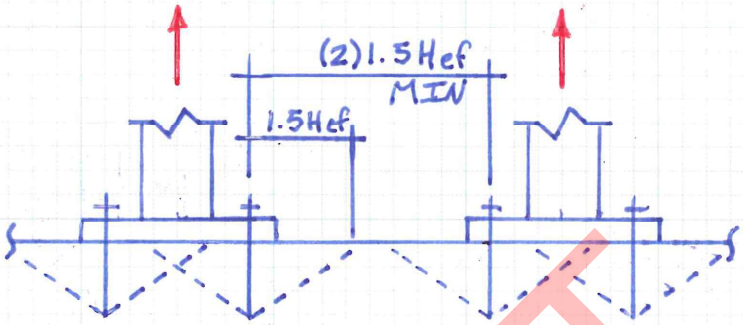
$S = PL^3/48EI = (150\ lb)(48")^3 / (48 \times 10{,}000{,}000\ psi \times 0.33\ in^4)$

$= 0.105\ in$

$48" / 0.105" \rightarrow L/457$

∴ 4040 STRUT IS ADEQUATE.

5. CONCRETE BREAK OUT GOVERNS.
CHECK THAT THE ANCHORS INFLUENCE AREA DOES NOT IMPACT (E) ANCHORS INFLUENCE AREA.



ANCHOR TYPE: ½"ϕ EXP ANCHOR (H_{ef} = 2")

MIN SPACING BETWEEN (N) & (E) FRAME SUPPORT = (2)1.5(2") = 6"

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SLAC BUILDING INSPECTION OFFICE

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BY: cnadler

DATE: 5/31/2026 **PRS#:** 26-118

Revised Authorization ☑ **No** ☐ **Yes**

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CBC 2025, ASCE 7-22 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

DSA Project?	NO			
Soil Site Class	Default	Section 11.4.2		
Response Spectral Acc. (0.1 sec) S_{DS} =	1.800g	= 180.0%g	(at 5% Damped Design)	ASCE 7 Hazards
Building Risk Category:	II			Table 1.5-2
I_E, Seismic =	1.0			
Seismic Design Category	D			ASCE 7 Hazards

13.3 Seismic Demands on Nonstructural Components

Component Name: Cable Tray Support
Component Description: Electrical conduit and cable trays

Component Location: Above grade plane

$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_u}\right]\left[\frac{C_{AR}}{R_{po}}\right]$ S_{DS} = 1.800 (13.3-1)

C_{AR} = 1.0	R_{po} = 1.5		T-13.5-1 or 13.6-1
Ω_{op} = 2.00			T-13.5-1 or 13.6-1
I_p = 1.0			13.1.3
R_μ = 1.69	Bldg R = 6.5	Bldg Ω_o = 2.5	
z = 10 ft	h = 10 ft		
H_f = 3.5			(13.3-5)

F_p = 0.971Wp			
Max F_p = $1.6 S_{DS} I_p W_p$	= 2.880Wp		(13.3-2)
Min F_p = $0.3 S_{DS} I_p W_p$	= 0.540Wp		(13.3-3)
F_p = 0.971 Wp			
$F_p \Omega_{op}$ = 1.942 Wp			T-13.5-1 footnote b or 13.6-1 footnote c

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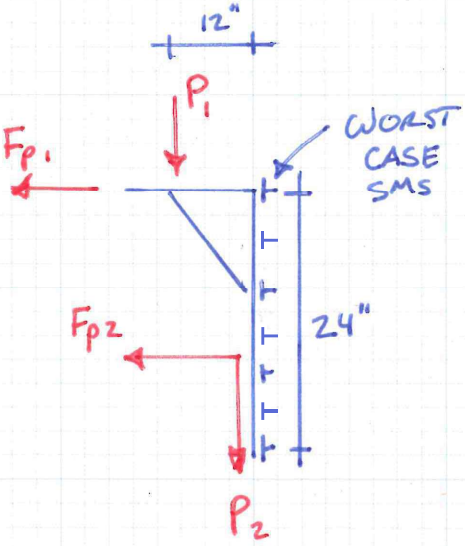
ZFA STRUCTURAL ENGINEERS

SLAC OPTICS
project name

PIPE & CONDUIT SUPPORT TO CFS WALL	DJM	25648	4/28	1/1
section	engineer	job #	date	page

CALCS TO JUSTIFY THE CONNECTION TO THE CFS STUD (4/E0.01)



LOADING:

$P_1 = 10plf(8') = 80^{\#}$

$F_{p1} = 0.971(80^{\#}) = 77.7^{\#}$

$P_2 = 31.5plf(8') = 252^{\#}$

$F_{p2} = 0.971(252^{\#}) = 245^{\#}$

CHECK 1/4 SMS THROUGH 2 LAYERS OF GYP TO CFS STUD

SHEAR: $80^{\#} + 252^{\#} = 332^{\#}$

$DCR = \frac{332^{\#}}{(7)166^{\#}} = 0.29$

TENSION: $\frac{77.7^{\#} + 245^{\#}}{4} + \frac{80^{\#} \cdot 12"}{24"} = 120^{\#}$

$DCR = \frac{120^{\#}}{261^{\#}} = 0.46$

COMBINED DCR = 0.29 + 0.46 = 0.47 < 1.0 ✓

SLAC BUILDING INSPECTION OFFICE	
Construction Authorization (IFC)	
BY: cnadler	
DATE: 5/31/2026	PRS#: 26-118
Revised Authorization	☑ No ☐ Yes
These plans have been reviewed for compliance with applicable building codes and standards, and ESH program requirements.	
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OPD-0001-13

SHEET METAL SCREW (SMS) ALLOWABLE STRENGTHS (CONTINUED)

TABLE 3 – NON-PRYING CONDITION

SHEET METAL SCREW ALLOWABLE STRENGTHS FOR STEEL TO STEEL CONNECTIONS WITH TWO LAYERS OF 5/8" GYP BOARD BETWEEN STEEL SURFACES.

F_y (KSI)	MIL (STEEL GA)	FASTENER SIZE									
		NO. 14 0.250 IN		NO. 12 0.216 IN		NO. 10 0.190 IN		NO. 8 0.164 IN		NO. 6 0.138 IN	
		SHEAR (LB)	TENSION (LB)	SHEAR (LB)	TENSION (LB)	SHEAR (LB)	TENSION (LB)	SHEAR (LB)	TENSION (LB)	SHEAR (LB)	TENSION (LB)
50	97 (12)	166	275	130	205						
	68 (14)	166	275	130	205	100	159				
	54 (16)	166	261	130	205	100	159	80	118		
33	43 (18)	166	144	130	124	100	109	80	94	50	79
	33 (20)					70	84	50	72	40	61

NOTES:
1. SEE GENERAL NOTES ON ST1.06 FOR MORE INFORMATION.
2. ALLOWABLE STRENGTH VALUES DO NOT ACCOUNT FOR EFFECTS FROM PRYING. RDP IN RESPONSIBLE CHARGE TO PROVIDE ADEQUATE BLOCKING/RESTRAINT TO PREVENT PRYING ACTION. WHERE PRYING OCCURS, THE VALUES AND CONSTRAINTS OF TABLE 4 SHALL BE USED.

CALCULATIONS JUSTIFY THE CONNECTION TO CFS STUD IN THE LONGITUDINAL DIRECTION (4/E0.01)

SEISMIC FORCE:

$0.7F_p = 0.7(77.7^{\#} + 245^{\#}) = 226^{\#}$

$DCR = \frac{226^{\#}}{(7)40^{\#}} = 0.81$

SHEAR DUE TO DEAD LOAD:

$P = 80^{\#} + 252^{\#} = 332^{\#}$

$DCR = \frac{332^{\#}}{(7)166^{\#}} = 0.29$

COMBINED SHEAR DCR $= \sqrt{0.81^2 + 0.29^2} = 0.86$

TENSION DUE TO DEADLOAD ASSUME TOP (3) SCREWS RESIST TENSION

$\frac{80^{\#} \cdot 12''}{20''} = 48\#$

$DCR = \frac{48\#}{(3)261^{\#}} = 0.07$

COMBINED TENSION & SHEAR DCR

$DCR = 0.86 + 0.07 = 0.93 < 1.0$ OK

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TABLE 4 – PRYING CONDITION (SEE DETAILS BELOW – STRUT CAN BE HORIZONTAL OR VERTICAL)
SHEET METAL SCREW ALLOWABLE STRENGTHS FOR STEEL TO STEEL CONNECTIONS WITH ONE OR TWO LAYERS OF 5/8" GYP BOARD BETWEEN STEEL SURFACES AND MAXIMUM PRYING MOMENT ARM OF 1 5/8".

F_y (KSI)	MIL (STEEL GA)	FASTENER SIZE									
		NO. 14 0.250 IN		NO. 12 0.216 IN		NO. 10 0.190 IN		NO. 8 0.164 IN		NO. 6 0.138 IN	
		SHEAR (LB)	TENSION (LB)	SHEAR (LB)	TENSION (LB)	SHEAR (LB)	TENSION (LB)	SHEAR (LB)	TENSION (LB)	SHEAR (LB)	TENSION (LB)
50	97 (12)	40	275	30	205						
	68 (14)	40	275	30	205	25	159				
	54 (16)	40	261	30	205	25	159	20	118		
33	43 (18)	40	144	30	124	25	109	20	94	10	79
	33 (20)					15	84	15	72	10	61

ZFA STRUCTURAL ENGINEERS SLAC OPTICS

project name

CONDUIT TRAPEZE SUPPORT	DJM	25649	4/28	1/1
section	engineer	job #	date	page

SLAC BUILDING INSPECTION OFFICE
Construction Authorization (IFC)
BY: cnadler
DATE: 5/31/2026 PRS#: 26-118
Revised Authorization ☑ No ☐ Yes
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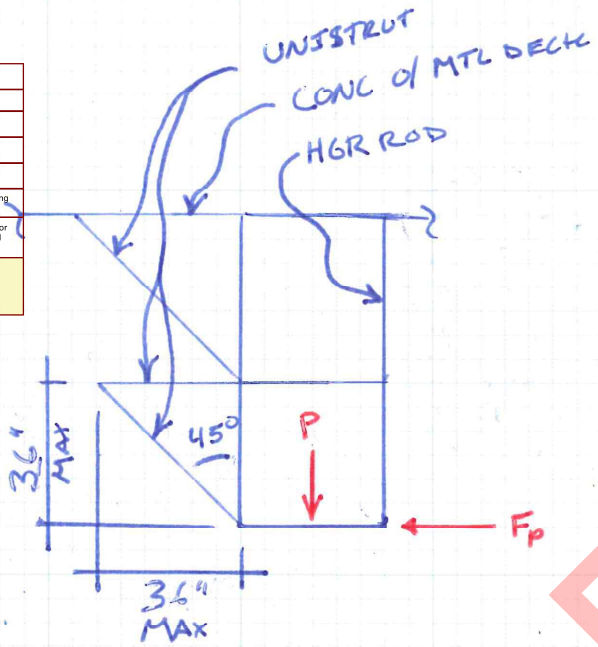

CALCS TO JUSTIFY DOUBLE TRAPEZE SYSTEM. (14/S1.00)
CAP FROM UNISTRUT CATALOG.

LOADING

$P = 21\,plf \cdot 8\,ft_{oc} = 168\#$

$F_p = 0.971\,W_p = 0.971(168\#) = 164\#$

LOAD IN BRACE

$F_B = \sqrt{2}\,(164\#) = 232\#$

CANTILEVER BEAM (Pg 18 & 25)

P1000 LOAD TABLE = 1,130# (3' SPAN)

BEAM FACTOR FOR CANT = 0.12

$LOAD = (0.12)(1{,}130\#) = 135\#$

$DCR = \frac{0.7(164\#)}{135\#} = 0.81 < 1.0$ OK

BRACE CAP (Pg 25)

CAP 12' UNBRACED LENGTH = 1,150#

$DCR = \frac{232}{1150} = 0.21 < 1.0$ Ok

CHANNEL NUT

CHANNEL NUT SLIP RESISTANCE = 1,500# AND OK BY INSPECTION

P1000 - Beam Loading

Span In	Max. Allowable Uniform Load Lbs	Defl. at Uniform Load In	Uniform Loading at Deflection Span/180 Lbs	Span/240 Lbs	Span/360 Lbs
24	1,690	0.06	1,690	1,690	1,690
36	1,130	0.13	1,130	1,130	900

	Load and Support Condition	Load Factor	Deflection Factor
7. Cantilever Beam, Concentrated Load at End		.12	3.20

P1000 - Column Loading

Unbraced Height In	Max. Allowable Load at Slot Face Lbs	Maximum Column Load Applied at C.G. K = 0.65 Lbs	K = 0.80 Lbs	K =1.0 Lbs	K = 1.2 Lbs
24	3,550	10,740	9,890	8,770	7,740
36	3,190	8,910	7,740	6,390	5,310
48	2,770	7,260	6,010	4,690	3,800
60	2,380	5,910	4,690	3,630	2,960
72	2,080	4,840	3,800	2,960	2,400
84	1,860	4,040	3,200	2,480	1,980
96	1,670	3,480	2,750	2,110	1,660
108	1,510	3,050	2,400	1,810	**
120	1,380	2,700	2,110	**	**
144	1,150	2,180	1,660	**	**

P1000 - Column Loading

Unbraced Height In	Max. Allowable Load at Slot Face Lbs	Maximum Column Load Applied at C.G. K = 0.65 Lbs	K = 0.80 Lbs	K =1.0 Lbs	K = 1.2 Lbs
24	3,550	10,740	9,890	8,770	7,740
36	3,190	8,910	7,740	6,390	5,310
48	2,770	7,260	6,010	4,690	3,800
60	2,380	5,910	4,690	3,630	2,960
72	2,080	4,840	3,800	2,960	2,400
84	1,860	4,040	3,200	2,480	1,980
96	1,670	3,480	2,750	2,110	1,660
108	1,510	3,050	2,400	1,810	**
120	1,380	2,700	2,110	**	**
144	1,150	2,180	1,660	**	**

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