STRUCTURAL ENGINEERING CALCULATIONS
FOR
AMERICAN WALK-IN COOLERS
SAMPLE
PROJECT NUMBER: 24-23100

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PROJECT INFORMATION

Tamarack Grove Engineering:

Address:

Date:
Firm Registration Number:
TGE Engineer of Record:
Project Manager:
Direct Phone:
Office Phone:
Office Fax:
Email:

Project Client Information:

Company:	American Walk-in Coolers
Project Number:	

Project Site Information:

Name:
Address:

Client Reference Number:

Local Jurisdiction Information:

Jurisdiction:	Jackson County
Enforced Code Used:	2022 Oregon Structural Specialty Code
Contact Info:	jacksoncountyor.gov

Project Scope of Work:

Tamarack Grove Engineering is providing structural engineering calculations for the walk-in manufactured by American Walk-in Coolers to verify the structural integrity of the panels and anchorage to the slab. The design of the slab/foundation is to be provided by others. Redline drawings provided to the client as needed based on calculations and code requirements.

SYMBOLS AND NOTATION

BSC = Building Site Class
C_e = Exposure Factor
C_T = Thermal Factor
DL_{panel} = Total Panel Dead Load
DL_{roof} = Dead Load Roof
EC = Exposure Category
F_a = Short Period Site Coefficient
F_v = Long Period Site Coefficient
I_E = Seismic Importance Factor
I_S = Snow Importance Factor
$L_{internal}$ = Minimum Indoor Lateral Live Load
LL_{panel} = Total Panel Live Load
LL_{panel_acc} = Total Panel Live Load (Accessible)
LL_{roof} = Live Load Roof
p_g = Ground Snow Load
P_{LL} = Maintenance Worker Live Load
R = Response Modification Coefficient
S_1 = Mapped MCE_R Spectral Response Acceleration Parameter at a Period of 1 s
S_{D1} = Design Spectral Response Acceleration Parameter at a Period of 1 s
SDC = Seismic Design Category
S_{DS} = Design Spectral Response Acceleration Parameter at Short Periods
S_{M1} = MCE_R Spectral Response Acceleration Parameter at a Period of 1 s
S_{MS} = MCER Spectral Response Acceleration Parameter at Short Periods Adjusted For Site
SRC = Surface Roughness Category
S_S = Mapped MCE_R Spectral Response Acceleration Parameter at Short Periods
T_L = Long Period Transition Period
V = Basic Wind Speed

GENERAL STRUCTURAL NOTES

1. General Structural Notes
   - A. Contractor to verify all openings, building dimensions, column locations and dimensions with owner prior to setting of any cooler boxes or construction.
   - B. The engineer of record is not responsible for any deviations from these plans unless such changes are authorized in writing to the engineer of record.
   - C. The contractor is responsible for providing safe and adequate shoring and/or temporary structural stability for all parts of the structure during construction. The structure shown on the drawings has been designed for final configuration.
   - D. Notching and/or cutting of any structural member in the field is prohibited, unless prior consent is given by the engineer of record.
   - E. All future roof/ceiling mounted equipment not currently shown on the approved shop drawings shall be coordinated with the EOR prior to any installation, typ.
   - F. The assumed thickness of existing concrete will be 4” with an f’c of 2,500 psi, unless otherwise noted in calculations.

2. Special Inspections & Testing (Quality Assurance Plan)
   - A. General:
     1. Independent testing lab shall be retained by owner to provide inspections and special inspections as described herein.
     2. The contractor is responsible for coordinating and providing on site access to all required inspections and notifies testing lab in time to perform such inspections prior.
     3. Do not cover work required to be inspected prior to inspection being made. If work is covered, contractor will be responsible for uncovering as necessary.
     4. The contractor shall correct all deficiencies as noted within the special inspection reports and/or the engineer of record's field observation (structural observations) reports to bring the construction into compliance with the contract documents, addendums, revisions, RFI's and/or written instructions. The contractor is responsible to request summary reports from the special inspector and engineer of record at the time of the project substantial completion. Prior to requesting the summary of structural observation reports from the engineer of record, the contractor shall submit to the architect and engineer of record a letter stating that all outstanding items noted on previous structural observation reports have been completed in accordance with the contract documents, addendums, revisions, RFI's and/or written instructions.

B. Special Inspections:

1. All special inspections shall be performed to meet the requirements of the 2022 Oregon Structural Specialty Code (2022 OSSC), as recommended by the local building jurisdiction.
2. Required special inspections shall be performed by an independent certified testing laboratory employed by the owner per section 1704 of the 2022 OSSC.
3. The independent certified testing laboratory and inspectors shall be a qualified person who shall show competence to the satisfaction of the local building official, owner, architect and engineer of record for the particular operation. All special inspection reports shall be submitted to the building department, architect and engineer of record stating the project name and address.

4. The contractor and special inspector shall notify the engineer of record of any items not complying with the project specifications, contract documents and/or applicable codes before proceeding with any work involving that item. The engineer of record will review the item and determine its acceptability. If work involving that item proceeds without prior approval from the engineer of record, then the work will be considered non-compliant.

DESIGN CRITERIA INFORMATION

Building Risk Category: **II**

Panel Specification:

Manufacturer:	American Walk-in Coolers
Analysis Method / Report Used:	LARR 25898

Dead/ Live Load Information Per ASCE 7:

DL_{panel} =	5.00 psf		
LL_{panel} =	10.00 psf		
LL_{panel_acc} =	20.00 psf		
$L_{internal}$ =	5.00 psf		
P_{LL} =	300 lbf		

<u>Dead Load Calculation</u>

Steel Facing (ASTM-A-646) Weight =	1.80 psf
Insulation Weight =	0.75 psf
Rail Weight =	0.45 psf
Miscellaneous =	2.00 psf

Seismic Load Information Per ASCE 7:

BSC =	D - Default		F_a =	1.290	Table 11.4-1
I_E =	1.0	Table 1.5-2	F_v =	1.933	Table 11.4-2
SDC =	D	Sec. 11.6	S_{MS} =	0.822	$S_{MS} = F_a * S_S$, Equation 11.4-1
S_S =	0.637	Per ASCE Hazard Tool	S_{M1} =	0.709	$S_{M1} = F_v * S_1$, Equation 11.4-2
S_1 =	0.367	Per ASCE Hazard Tool	S_{DS} =	0.548	$S_{DS} = 2/3 * S_{MS}$, Equation 11.4-3
T_L =	16	Per ASCE Hazard Tool	S_{D1} =	0.473	$S_{D1} = 2/3 * S_{M1}$, Equation 11.4-4

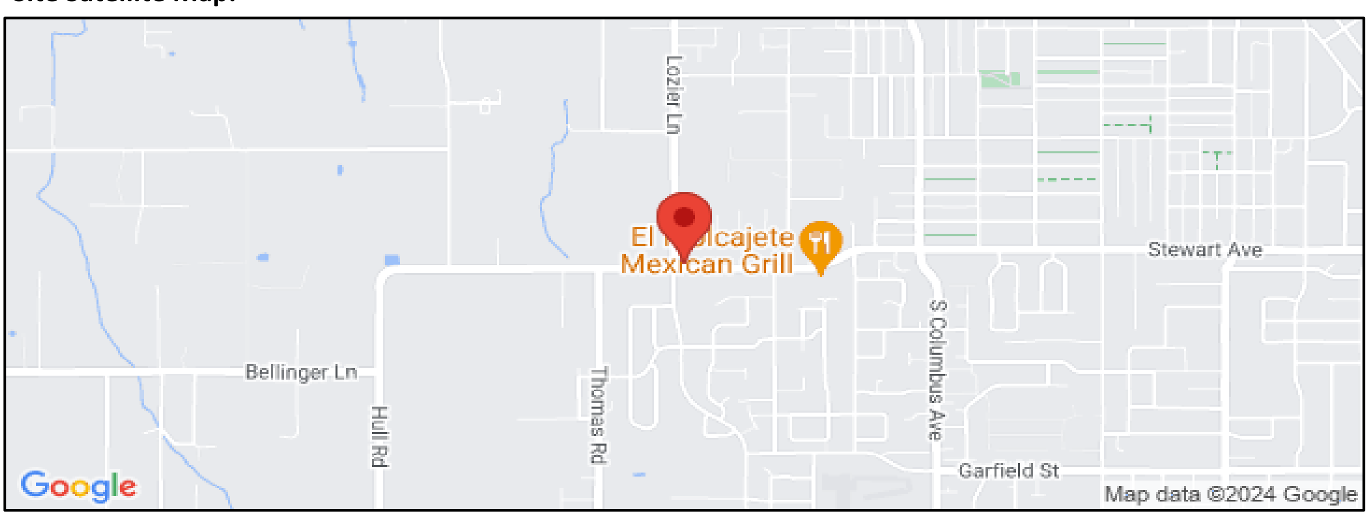
Snow Design Information Per ASCE 7:

Not Applicable

Wind Design Information:

Not Applicable

Site Satellite Map:



STRUCTURAL CALCULATIONS CBX (1)

JURISDICTION INFORMATION

JURISDICTION:	MEDFORD , OREGON	PASS = 1.0	FAILURE = 0
STRUCTURAL CODE:	2022 OREGON STRUCTURAL SPECIALTY CODE		

DESIGN CRITERIA

LOAD DESIGN VALUES:

$DL_{panel} := 5\ psf$	Panel Dead Load
$LL_{panel} := 10\ psf$	Panel Live Load - Not Accessible
$LL_{panel\_2} := 20\ psf$	Panel Live Load - Accessible
$P_{LL} := 300\ lbf$	Maintenance Worker Live Load
$P_{internal} := 5\ psf$	Minimum Transverse Load (ASCE 7 1.4.5)

NOTE: SEISMIC DESIGN DATA IS GIVEN IN THE LATERAL ANALYSIS SECTION BELOW.

ASD LOAD COMBINATIONS (ASCE 7-16)

$LC_3 := DL_{panel} + LL_{panel} = 15\ psf$	Load Combination 3: D+(Lr, S, or R)
$LC_{3\_acc} := DL_{panel} + LL_{panel\_2} = 25\ psf$	Load Combination 3: D+(Lr, S, or R)

WALK-IN DESIGN CRITERIA

$Width := 45.17\ ft$	Unit Width
$Length := 57.00\ ft$	Unit Length
$H := 8.00\ ft$	Unit Height
$H_w := 7.67\ ft$	Wall Height

NOTE: REFERENCE THE LOS ANGELES RESEARCH REPORT (LARR) AND/OR TESTING REPORTS PROVIDED IN THE APPENDIX FOR PANEL ALLOWABLES UTILIZED.

NON-ACCESSIBLE CEILING PANEL ANALYSIS

$L := 11.17\ ft$ — Ceiling Panel Span

$T_{width\_panel} := 3.83\ ft$ — Tributary Width of Panel

LOADS:

$P_{LL} := 300\ lbf$ — Maintenance Worker Live Load

$LL_{panel} := 10\ psf$ — Panel Live Load - Not Accessible

$w_{design\_ceiling} := LL_{panel} \cdot T_{width\_panel} = 38.3\ plf$ — Distributed Live Load

$$LL_{gov} := \text{if}\left(\frac{P_{LL} \cdot L}{4} \geq \frac{w_{design\_ceiling} \cdot L^2}{8}, P_{LL}, LL_{panel} \cdot T_{width\_panel}\right) = 300\ lbf$$ Governing Load

$Wt_{unit} := 115\ lbf$ — Unit Weight

$a := \frac{L}{2} = 5.585\ ft$ — Unit Location

TYPICAL NON-REINFORCED PANEL:

$L_{all} := 12\ ft$ — Allowable Span (Testing Report)

$m_{max} := 991\ ft \cdot lbf$ — Maximum Moment (Enercalc)

$w_{all} := 20\ psf \cdot T_{width\_panel} = 76.6\ plf$ — Allowable Panel Load (Testing Report)

$M_{allow} := \frac{w_{all} \cdot L_{all}^2}{8} = 1378.8\ ft \cdot lbf$ — Allowable Moment

CHECK $M_{allow} \geq m_{max} = 1$

<u>SUMMARY: USE 4" THICK BASF HIGH DENSITY FOAM RAILS CEILING PANELS.</u>

CDS LEGACY DOOR ANALYSIS (WORST CASE)

$H_w = 7.67\ ft$ — Design Height

$T_{width\_panel} := 3.92\ ft$ — Tributary Width of Panel

$T_{width\_wall} := \frac{L}{2} = 5.59\ ft$ — Tributary Width of Ceiling Panel Acting on Wall

$T_{width\_header} := 1.07\ ft$ — Height of Header above Anthony Door

AXIAL LOADS:

$V_{max} := 177.5\ lbf$ — Axial Load from Ceiling Panel (Enercalc)

$DL_{panel} := 5\ psf$ — Panel Dead Load

$$p_{design\_wall} := \frac{V_{max}}{T_{width\_panel}} + DL_{panel} \cdot \left(T_{width\_header} + T_{width\_wall}\right) = 78.56\ plf$$

Ceiling Panel Total Axial Load

$P_{all\_anthony} := 150\ plf$ — Allowable Axial Load for Anthony Door(Testing Report)

CHECK $P_{all\_anthony} \geq p_{design\_wall} = 1$

SUMMARY: USE CDS LEGACY DOOR.

WALL PANEL ANALYSIS

$H_w = 7.67\ ft$ — Design Height

$T_{width\_panel} = 3.92\ ft$ — Tributary Width of Panel

$T_{width\_wall} := \frac{L}{2} = 5.59\ ft$ — Tributary Width of Roof Panel Acting on Wall

AXIAL LOADS:

$P_{unit} := 28\ lbf$ — Coil Load on Wall Panel (Enercalc)

$p_{design\_wall} := \frac{P_{unit}}{T_{width\_panel}} + \max\left(\frac{P_{LL}}{T_{width\_panel}}, LL_{panel} \cdot T_{width\_wall}\right) + DL_{panel} \cdot T_{width\_wall} = 111.6\ plf$ — Ceiling Panel Axial Load

$H_{all\_axial} := 10\ ft$ — Allowable Height for Axial Load (Testing Report)

$P_{all\_axial} := 763\ plf$ — Allowable Axial Load (Testing Report)

TRANSVERSE LOADS:

$w_{wall} := P_{internal} \cdot T_{width\_panel} = 19.6\ plf$ — Transverse Load on Wall

$m_{max} := \frac{w_{wall} \cdot H_w^{\ 2}}{8} = 144.13\ ft \cdot lbf$ — Maximum Moment

$H_{all\_trans} := 20\ ft$ — Allowable Height for Transverse Load (Testing Report)

$P_{all\_trans} := 5\ psf \cdot T_{width\_panel} = 19.6\ plf$ — Allowable Transverse Load (Testing Report)

$M_{allow} := \frac{P_{all\_trans} \cdot H_{all\_trans}^{\ 2}}{8} = 980\ ft \cdot lbf$ — Allowable Moment

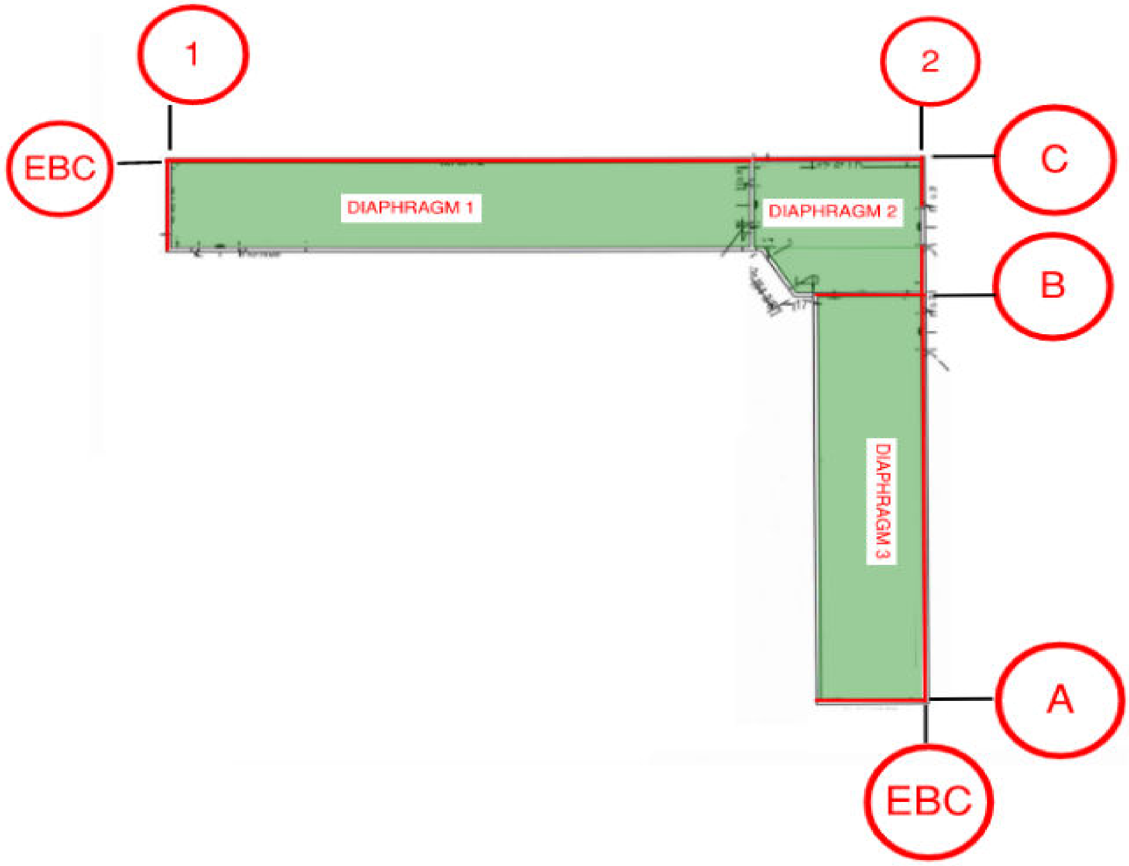
$P_{comb} := \frac{p_{design\_wall}}{P_{all\_axial}} + \frac{m_{max}}{M_{allow}} = 0.29$ — Interaction of Axial and Transverse Loads

CHECK $P_{comb} \le 1 = 1$ $H_{all\_axial} \ge H_w = 1$ $H_{all\_trans} \ge H_w = 1$

SUMMARY: : USE 4" THICK BASF HIGH DENSITY FOAM RAILS WALL PANELS.

LATERAL ANALYSIS

GRIDLINES:



EXECUTIVE SUMMARY:

PER ASCE 7 CHAPTER 15, SECTION 15.1.3, "STRUCTURAL ANALYSIS PROCEDURES FOR NONBUILDING STRUCTURES THAT ARE SIMILAR TO BUILDINGS SHALL BE SELECTED IN ACCORDANCE WITH SECTION 12.6.". THUS, PER ASCE 7 SECTION 12.8, THE EQUIVALENT LATERAL FORCE PROCEDURE WILL BE USED. PER ASCE 7 TABLE 12.2-1, THE SEISMIC FORCE-RESISTING SYSTEM SHALL BE "A. BEARING WALL SYSTEM, 17. LIGHT FRAME WALLS WITH SHEAR PANELS OF ALL OTHER MATERIALS."

$R_p := 1.5$ Response Modification Factor

$a_p := 1.0$ Amplification Factor

DESIGN DATA:

$I_e := 1.0$ Importance Factor

$S_s := 0.637$ Mapped Spectral Response Acceleration Parameter at Short Periods

$S_1 := 0.367$ Mapped Spectral Response Acceleration Parameter at a Period of 1 s

$S_{DS} := 0.548$ Design Spectral Response Acceleration Parameter at Short Periods

$F_v := 1.933$ Long-Period Site Coefficent (Table 11.4-2)

$S_{D1} := \frac{2}{3} \cdot S_1 \cdot F_v = 0.47$ — Design Spectral Acceleration Parameters at 1-s Period (11.4.5)

$T_L := 16$ — Long-Period Transition Period

$F_a := 1.290$ — Short-Period Site Coefficient

$T_s := \frac{S_{D1}}{S_{DS}} = 0.86$

$h_n := \frac{H}{ft} = 8$ — Height of Structure

$C_t := 0.02$ $x := 0.75$ — Approximate Period Parameter 1 & 2 (Table 12.8-2)

DIAPHRAGM 1:

$Width_1 := 7.67\ ft$ — Diaphragm Width

$Length_1 := 44.17\ ft$ — Diaphragm Length

DESIGN CRITERIA:

$A_{ceiling} := Width_1 \cdot Length_1 = 338.78\ ft^2$ — Total Area of Ceiling

$L_{wall} := 2 \cdot Length_1 + 2 \cdot Width_1 = 103.68\ ft$ — Total Length of Walls

$Wt_{units} := 2 \cdot 115\ lbf = 230\ lbf$ — Total Unit Weight

$Wt_{steel} := 0\ lbf$ — Total Steel Weight

LATERAL FORCE GENERATION - ASCE 7 13.3.1:

$S_{DS} = 0.548$ — Spectral Acceleration, Short Period

$I_p := 1.0$ — Importance Factor (ASCE 7 13.1.3)

$z := H = 8\ ft$ — Height of Attachment with Respect to the Base

$h := H = 8\ ft$ — Average Roof Height of Structure

$H_w = 7.67\ ft$ — Wall Panel Height

$W_P := \left(DL_{panel} \cdot \left(A_{ceiling}\right)\right) + \left(DL_{panel} \cdot H_w \cdot L_{wall}\right) + Wt_{units} + Wt_{steel} = 5900.05\ lbf$ — Operating Weight

$$F_p := \frac{0.4 \cdot a_p \cdot S_{DS} \cdot W_P}{\left(\frac{R_p}{I_p}\right)} \cdot \left(1 + 2 \cdot \frac{z}{h}\right) = 2586.58\ lbf$$ Seismic Force

$F_{p\_max} := 1.6 \cdot S_{DS} \cdot I_p \cdot W_P = 5173.16\ \mathit{lbf}$ Maximum Lateral Seismic Force

$F_{p\_min} := 0.3 \cdot S_{DS} \cdot I_p \cdot W_P = 969.97\ \mathit{lbf}$ Minimum Lateral Seismic Coefficient

$F_p := \mathit{min}\left(F_p, F_{p\_max}\right) = 2586.58\ \mathit{lbf}$

$F_p := \max\left(F_p, F_{p\_min}\right) = 2586.58\ \mathit{lbf}$ Seismic Design Force

$F_{p\_vert} := 0.2 \cdot S_{DS} \cdot W_P = 646.65\ \mathit{lbf}$ Vertical Seismic Design Force

$F_{p\_asd} := 0.7 \cdot F_p = 1810.61\ \mathit{lbf}$ ASD Seismic Design Force

$w_{design\_1} := \dfrac{F_{p\_asd}}{Length_1} = 40.99\ \mathit{plf}$ Design Load in 1-1

$w_{design\_2} := \dfrac{F_{p\_asd}}{Width_1} = 236.06\ \mathit{plf}$ Design Load in 2-2

DIAPHRAGM CHECK (2-2):

$Width_2 := Width_1 = 7.67\ \mathit{ft}$ Width of Diaphragm (2-2)

$Length_2 := Length_1 = 44.17\ \mathit{ft}$ Length of Diaphragm (2-2)

$R_2 := \dfrac{Width_2}{Length_2} = 0.17$ Aspect Ratio (2-2)

$F_{all\_2} := 130\ \mathit{plf}$ Allowable Diaphragm Capacity (Testing Report)

CHECK: $F_{all\_2} \ge \dfrac{w_{design\_2} \cdot Width_2}{Length_2} = 1$

<u>SUMMARY: : USE 4" THICK BASF HIGH DENSITY FOAM RAILS CEILING PANELS.</u>

CHORD FORCE:

$F_{chord} := \dfrac{\dfrac{w_{design\_2} \cdot {Width_2}^2}{2}}{Length_2} = 157.2\ \mathit{lbf}$ Chord Force

CAM-LOCK:

$N_{cam} := 2$ Number of Camlocks per Panel

$V_{all\_cam} := 244\ \mathit{lbf}$ Allowable Wall to Ceiling Panel Shear on Camlock (Testing Report)

CHECK: $N_{cam} \cdot V_{all\_cam} \ge F_{chord} = 1$

<u>SUMMARY: THUS, END CAMLOCKS ARE SUFFICIENT TO WITHSTAND THE CHORD FORCE.</u>

SHEAR WALL CALCULATIONS(1-1 Direction):

$d := \frac{Width_1}{2} = 3.84\ ft$ — Moment Arm

$M := F_{p\_asd} \cdot d = 6943.68\ ft \cdot lbf$ — Design Moment

$T := \frac{M}{Length_1} = 157.2\ lbf$ — Tension/ Comp. Force in chords

$L_1 := Width_1 = 7.67\ ft$ — Length of Gridline 1

$f_1 := \frac{T}{L_1} = 20.5\ plf$ — In-Plane Force along Gridline 1

$L_C := Length_1 = 44.17\ ft$ — Length of Gridline A

$T_{width\_A} := Width_1 = 7.67\ ft$ — Tributary Width

$f_C := \frac{w_{design\_2} \cdot T_{width\_A}}{L_C} = 40.99\ plf$ — In-Plane Force along Gridline A

$R := \frac{H_w}{min\left(L_1, L_C\right)} = 1.00$ — Worst Case Shape Ratio

$F_{all\_inplane} := 117\ plf$ — Allowable In-Plane Shear (Testing Report)

CHECK: $F_{all\_inplane} \geq \max\left(f_1, f_C\right) = 1$

SUMMARY: USE 4" THICK BASF HIGH DENSITY FOAM RAILS WALL PANELS FOR LATERAL RESISTANCE.

L-ANGLE CONNECTION TO EXISTING GROUTED CMU WALL

LOADS:

$W_{des} := w_{design\_1} = 40.99\ plf$ — Distributed Design Load

$t_{angle} := 0.036\ in$ — Thickness of Angle (20 GA)

$t_1 := t_{angle} = 0.036\ in$ — Thickness of Member in Contact with Screw Head or Washer

1/4" TITEN HD TO EXISTING GROUTED CMU WALL:

$S_{anchor} := 16\ in$ — Spacing of Anchor into Grouted CMU Wall

$\Omega_0 := 2.0$ — Overstrength Factor

$$T_{anchor} := \frac{\Omega_0 \cdot W_{des} \cdot S_{anchor}}{0.7} = 156.16\ lbf$$ Tension on Anchors From Distributed Design Load

$T_{all\_anchor} := 410\ lbf$ — Allowable Tension on Anchor (Simpson Catalog)

CHECK $T_{anchor} \le T_{all\_anchor} = 1$

#14 TEK SCREW TO THE CEILING:

$N_{screw} := 1$ — No. of Screws

$S_{screw} := 12\ in$ — Spacing of Screws

$$V_{all\_screw} := \frac{N_{screw} \cdot 60\ lbf}{S_{screw}} = 60\ plf$$ Allowable Shear on Screw (SSMA)

CHECK $V_{all\_screw} \ge W_{des} = 1$

ANGLE IN FLEXURE CHECK:
LOADS:

$t := t_{angle} = 0.036\ in$	Uncoated Sheet Thickness
$L_{leg} := 2\ in$	Leg Dimension
$d := 0.5\ in$	Moment-Arm

$I := \frac{S_{anchor} \cdot t^3}{12} = 0.0001\ in^4$ $\quad y := \frac{t}{2} = 0.02\ in$

$S_x := \frac{I}{y} = 0.003\ in^3$

$F_y := 40\ ksi$	Yield Stress of Steel
$M_{all} := \frac{F_y \cdot S_x}{1.67} = 6.9\ ft \cdot lbf$	Yield Moment about x-axis (AISI S100-16)
$w_{design\_all} := \frac{M_{all}}{d \cdot S_{anchor}} = 124.17\ plf$	Allowable Load in Angle

CHECK $w_{design\_all} \geq W_{des} = 1$

ANGLE TENSILE STRENGTH (AISI S100 SECTION D):

$s := 6\ in$	Spacing between holes	$F_u := 42.1\ ksi$	Tensile Strength of Angle
$t = 0.036\ in$	Thickness of Angle	$F_y := 30\ ksi$	Yield Stress of Angle
$\Omega_{t\_y} := 1.67$	ASD Factor for Yielding	$\Omega_{t\_r} := 2.00$	ASD Factor for Rupture
$d_{dia} := 0.25\ in$	Diameter of Screw		
$A_g := t \cdot s = 0.22\ in^2$	Gross Area of Cross-section	$A_n := \left(s - \left(d_{dia} + \frac{1}{16}\ in\right)\right) \cdot t = 0.2\ in^2$	Gross Area of Cross-section

$T_{n\_y} := \frac{A_g \cdot F_y}{\Omega_{t\_y}} = 3880.24\ lbf$	Allowable Tensile Strength due to Yielding
$T_{n\_r} := \frac{A_n \cdot F_u}{\Omega_{t\_r}} = 4309.99\ lbf$	Allowable Tensile Strength due to Rupture
$T_{n\_all} := \frac{min\left(T_{n\_y}, T_{n\_r}\right)}{s} = 7760.5\ plf$	Allowable Tensile Strength/ft

CHECK $T_{n\_all} \geq W_{des} = 1$

ANGLE COMPRESSIVE STRENGTH (AISI S100 SECTION E):

$L := 4\ in$ Length of Angle Leg — $E := 28000\ ksi$ Modulus of Elasticity of 316 SS

$t = 0.036\ in$ Thickness of Angle — $F_y := 30\ ksi$ Yield Stress of Angle

$\Omega_c := 1.80$ ASD Factor for Compression — $K := 1$ Effective length factor

$r := \frac{t}{\sqrt{12}} = 0.01\ in$ Radius of Gyration of full unreduced cross-section about axis of buckling

$S_l := \frac{K \cdot L}{r} = 384.9$ Slenderness Ratio

$F_{cre} := \frac{\pi^2 \cdot E}{{S_l}^2} = 1865.4\ psi$ Elastic Flexural Buckling Stress

$\lambda_c := \sqrt{\frac{F_y}{F_{cre}}} = 4.01$

$F_n := \text{if}\left(\lambda_c \le 1.5, \left(0.658^{{\lambda_c}^2}\right) \cdot F_y, \left(\frac{0.877}{{\lambda_c}^2}\right) \cdot F_y\right) = 1635.9\ psi$ Compressive Stress

$A_g := t = 0.04\ in$ Gross Area/ft

$P_{n\_all} := \frac{F_n \cdot A_g}{\Omega_c} = 392.6\ plf$ Allowable Axial Strength/ft

CHECK $P_{n\_all} \ge W_{des} = 1$

<u>SUMMARY: THERFORE, PROVIDE A 20GA. CONTINUOUS ANGLE WITH 1/4" SIMSON ANCHOR @ 16" O.C.INTO THE EXISTING BUILDING GROUTED CMU WALL AND #14 TEK SCREWS @12" O.C INTO CEILING PANELS</u>

CEILING PANEL TO WALL PANEL CONNECTION

$H_w = 7.67\ ft$ — Unit Height

LOADS:

$P_{internal} = 5\ psf$ — Transverse Load on Wall

$p_{trans} := P_{internal} \cdot \frac{H_w}{2} = 19.18\ plf$ — Transverse Shear Force on Wall-Ceiling Connection

$f_{inplane} := \max\left(f_1, f_c\right) = 40.99\ plf$ — In-Plane Shear Force on Wall-Ceiling Connection

$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 40.99\ plf$ — Governing Shear Force on Wall-Ceiling Connection

CAM-LOCK:

$S_{cam} := 23\ in$ — Spacing of Cam-lock

$V_{all\_cam} := \frac{244\ lbf}{S_{cam}} = 127.3\ plf$ — Allowable Shear on Camlock (Testing Report)

CHECK $f_{max} \le V_{all\_cam} = 1$

<u>SUMMARY: USE CAMLOCKS @23" O.C. FOR CEILING PANELS TO WALL PANEL CONNECTION.</u>

TYPICAL WALL TO FLOOR/CONCRETE CONNECTION

LOADS:

$P_{internal} = 5\ psf$	Transverse Load on Wall
$p_{trans} := P_{internal} \cdot \frac{H_w}{2} = 19.18\ plf$	Transverse Shear Force on Floor-Wall Connection
$f_{inplane} := \max\left(f_1, f_c\right) = 40.99\ plf$	In-Plane Shear Force on Floor-Wall Connection
$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 40.99\ plf$	Governing Shear Force on Floor-Wall Connection

#10 TEK SCREWS:

$S_{screw} := 16\ in$	Spacing of Screw
$V_{all\_screw} := \frac{370\ lbf}{S_{screw}} = 277.5\ plf$	Allowable Shear Load (SSMA)
$T_{all\_screw} := \frac{137\ lbf}{S_{screw}} = 102.75\ plf$	Allowable Tension Load (SSMA)

3/8" HILTI KH-EZ ANCHOR:

$\Omega_o := 2.0$	Overstrength Factor
$S_{anchor} := 23\ in$	Spacing of Anchors
$v_{anchor} := \frac{\Omega_o \cdot f_{max} \cdot S_{anchor}}{0.7} = 224.48\ lbf$	LRFD Maximum Shear Force on Anchors
$V_{all\_anchor} := 1500\ lbf$	Allowable Shear on Anchor (See Anchor Report)

CHECK $f_{inplane} \le V_{all\_cam} = 1$ $p_{trans} \le T_{all\_screw} = 1$ $v_{anchor} \le V_{all\_anchor} = 1$

SUMMARY: USE 10"x1" S/S TEK SCREWS @ 16" O.C. AND 3/8"x3" HILTI-KH-EZ @23" O.C. FOR WALL TO FLOOR/CONCRETE CONNECTION.

OVERTURNING CALCULATIONS

$DL_{panel} := 5\ psf$ — Panel Dead Load

$T_{width\_ceiling} := 3.92\ ft$ — Tributary Width of Ceiling

$H_w = 7.67\ ft$ — Height of Wall Panel

ASD LOADS:

$f := f_1 = 20.5\ plf$ — In-Plane Force on Wall

$L := L_1 = 7.67\ ft$ — Length of Wall

$S_{DS} = 0.548$ — Seismic Design Parameter

$Wt_{wall} := (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{panel} \cdot H_w \cdot L = 153.92\ lbf$ — Weight of Wall

$Wt_{ceiling} := (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 78.67\ lbf$ — Weight of Ceiling

$w_R := \dfrac{Wt_{wall} + Wt_{ceiling}}{L} = 30.32\ plf$ — Weight Resisting Overturning

$M_{wall} := f \cdot L \cdot H_w - w_R \cdot \dfrac{L^2}{2} = 313.78\ lbf \cdot ft$ — Overturning Moment Acting on Wall

$w := \dfrac{3 \cdot M_{wall}}{L^2} = 16\ plf$ — Maximum Value of Overturning Force at End ($M_{wall} := \dfrac{1}{2} \cdot w \cdot L \cdot \dfrac{2\ L}{3} = 313.78\ lbf \cdot ft$)

#10 TEK SCREWS:

$S_{screw} := 16\ in$ — Spacing of Screw

$V_{des\_screw} := 370\ lbf$ — Design Shear Load (SSMA)

$T_{des\_screw} := 137\ lbf$ — Design Tension Load (SSMA)

$V_{screw\_inplane} := f \cdot S_{screw} = 27.33\ lbf$ — Maximum Shear Force on End Screw due to Inplane Shear

$V_{screw\_uplift} := w \cdot S_{screw} = 21.34\ lbf$ — Maximum Shear Force on End Screw due to Uplift

$V_{screw} := \sqrt{V_{screw\_inplane}^{\ 2} + V_{screw\_uplift}^{\ 2}} = 34.67\ lbf$ — Maximum Resultant Shear Force on End Screw

$T_{screw} := p_{trans} \cdot S_{screw} = 25.57\ lbf$ — Maximum Tension Force on End Screw

CHECK $V_{des\_screw} \ge V_{screw} = 1$ $T_{des\_screw} \ge T_{screw} = 1$

LRFD LOADS:

$f := \frac{f}{0.7} = 29.28\ plf$ — In-Plane Force on Wall

$Wt_{wall} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot H_w \cdot L = 232.49\ lbf$ — Weight of Wall

$Wt_{ceiling} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 118.82\ lbf$ — Weight of Ceiling

$w_R := \frac{Wt_{wall} + Wt_{ceiling}}{L} = 45.8\ plf$ — Weight Resisting Overturning

$M_{wall} := \Omega_0 \cdot f \cdot L \cdot H_w - w_R \cdot \frac{L^2}{2} = 2097.71\ lbf \cdot ft$ — Overturning Moment Acting on Wall

$w := \frac{3 \cdot M_{wall}}{L^2} = 106.97\ plf$ — Maximum Value of Overturning Force at End $\left(M_{wall} := \frac{1}{2} \cdot w \cdot L \cdot \frac{2\ L}{3} = 2097.71\ lbf \cdot ft\right)$

3/8" HILTI KH-EZ ANCHOR:

$S_{anchor} := 23\ in$ — Spacing of Anchor

$\Omega_0 := 2.0$ — Overstrength Factor

$v_{anchor} := \Omega_0 \cdot f \cdot S_{anchor} = 112.24\ lbf$ — Ultimate Governing Shear on Anchor

$T_{anchor} := w \cdot S_{anchor} = 205.03\ lbf$ — Maximum Tension Force on End Anchor

NOTE: SEE APPENDIX FOR ANCHOR SOFTWARE PRINTOUTS FOR THE ANCHOR ANALYSIS.

SUMMARY: THUS, 10"x1" S/S TEK SCREWS @16" O.C. AND 3/8"x3" HILTI-KH-EZ @23" O.C. IS SUFFICIENT TO RESIST THE OVERTURNING FORCES.

DIAPHRAGM 2:

$Width_2 := 11.50\ ft$ Diaphragm Width

$Length_2 := 12.83\ ft$ Diaphragm Length

DESIGN CRITERIA:

$A_{ceiling} := Width_2 \cdot Length_2 = 147.55\ ft^2$ Total Area of Ceiling

$L_{wall} := 2 \cdot Length_1 + 2 \cdot Width_1 = 103.68\ ft$ Total Length of Walls

$Wt_{units} := 115\ lbf = 115\ lbf$ Total Unit Weight

$Wt_{steel} := 0\ lbf$ Total Steel Weight

LATERAL FORCE GENERATION:

$Wt := \left(A_{ceiling} \cdot DL_{panel}\right) + \left(\frac{H_w}{2} \cdot L_{wall} \cdot DL_{panel}\right) + Wt_{units} + Wt_{steel} = 2840.79\ lbf$ Effective Seismic Weight

$T_a := C_t \cdot h_n{}^x = 0.095$ Approximate Fundamental Period

NOTE: IF THE STRUCTURE IS 5 STORIES OR LESS ABOVE THE BASE, S_{DS} MAY BE RECALCULATED AS:

$S_{DS\_max} := \text{if}\left(T_a \le 0.5, \max\left(1.0, 0.7 \cdot S_{DS}\right), S_{DS}\right) = 1$ Seismic Coefficient (12.8.1.3)

$S_{DS} := \min\left(S_{DS\_max}, S_{DS}\right) = 0.55$ Design Spectral Response for Short Period, (g)

$C_s := \frac{S_{DS}}{\left(\frac{R_p}{I_e}\right)} = 0.37$ Seismic Response Coefficient (Sec. 12.8.1.1)

$\left(T_a \le 1.5\ T_s = 1\right)$

$C_{s\_max} := \text{if}\left(T_a \le T_L, 1.5 \cdot \frac{S_{D1}}{T_a \cdot \left(\frac{R_p}{I_e}\right)}, 1.5 \cdot \frac{S_{D1} \cdot T_L}{T_a{}^2 \cdot \left(\frac{R_p}{I_e}\right)}\right) = 4.97$ Maximum Coefficient

$C_{s\_min} := \max\left(0.044 \cdot S_{DS} \cdot I_e, 0.01\right) = 0.024$ Minimum Coefficient

$C_{s\_min} := \text{if}\left(S_1 \ge 0.6, \frac{0.5 \cdot S_1}{\left(\frac{R_p}{I_e}\right)}, C_{s\_min}\right) = 0.024$ Minimum Coefficient

$C_s := \max\left(C_s, C_{s\_min}\right) = 0.365$ Seismic Response Coefficient

$C_s := \min\left(C_s, C_{s\_max}\right) = 0.365$ Seismic Response Coefficient

$V_p := C_s \cdot Wt = 1037.83\ lbf$ Seismic Base Shear

$V_{p\_asd} := 0.7 \cdot V_p = 726.48\ lbf$ ASD Seismic Base Shear

$w_{design\_1} := \frac{V_{p\_asd}}{Length_2} = 56.62\ plf$ Distributed Design Load (1-1)

$w_{design\_2} := \frac{V_{p\_asd}}{Width_2} = 63.17\ plf$ Distributed Design Load (2-2)

DIAPHRAGM CHECK (1-1):

$Width_1 := Width_2 = 11.5\ ft$ Width of Diaphragm (1-1)

$Length_1 := Length_2 = 12.83\ ft$ Length of Diaphragm (1-1)

$R_1 := \frac{Length_1}{Width_1} = 1.12$ Aspect Ratio (1-1)

$F_{all\_1} := 114\ plf$ Allowable Diaphragm Capacity (Testing Report)

CHECK: $F_{all\_1} \geq \frac{w_{design\_1} \cdot Length_1}{2 \cdot Width_1} = 1$

<u>SUMMARY: : USE 4" THICK BASF HIGH DENSITY FOAM RAILS CEILING PANELS.</u>

CAMLOCK:

$V := w_{design\_1} \cdot Length_1 = 726.48\ lbf$ Max Shear on Diaphram

$N_{cam} := ceil\left(\frac{Length_1}{23\ in}\right) \cdot 2 = 14$ Number of Camlocks Connecting panels

$V_{all\_cam} := 244\ lbf = 244\ lbf$ Allowable Shear on Camlock (Testing Report)

$V_{all\_inplane} := N_{cam} \cdot V_{all\_cam} = 3416\ lbf$ Allowable In-Plane Shear on Camlock (Testing Report)

CHECK: $V_{all\_inplane} \geq V = 1$

<u>SUMMARY: THUS, CAMLOCKS @48" O.C. ARE SUFFICIENT TO WITHSTAND THE DIAPHRAGM SHEAR</u>

CHORD FORCE:

$F_{chord} := \frac{\frac{w_{design\_1} \cdot Length_1^{\,2}}{2}}{Width_1} = 405.25\ lbf$ Chord Force

CAM-LOCK:

$N_{cam} := 2$ Number of Camlocks per Panel

$V_{all\_cam} := 244\ lbf = 244\ lbf$ Allowable Wall to Ceiling Panel Shear on Camlock (Testing Report)

CHECK: $N_{cam} \cdot V_{all\_cam} \geq F_{chord} = 1$

<u>SUMMARY: THUS, END CAMLOCKS ARE SUFFICIENT TO WITHSTAND THE CHORD FORCE.</u>

DIAPHRAGM CHECK (2-2):

$Width_2 := Width_2 = 11.5\ ft$ — Width of Diaphragm (2-2)

$Length_2 := Length_2 = 12.83\ ft$ — Length of Diaphragm (2-2)

$R_2 := \dfrac{Width_2}{Length_2} = 0.90$ — Aspect Ratio (2-2)

$F_{all\_2} := 120\ plf$ — Allowable Diaphragm Capacity (Testing Report)

CHECK: $F_{all\_2} \geq \dfrac{w_{design\_2} \cdot Width_2}{2 \cdot Length_2} = 1$

SUMMARY: : USE 4" THICK BASF HIGH DENSITY FOAM RAILS CEILING PANELS.

CAMLOCK:

$V := \dfrac{w_{design\_2} \cdot Width_2}{2} = 363.24\ lbf$ — Max Shear on Diaphram

$N_{cam} := \mathrm{ceil}\left(\dfrac{Width_2 - 2\ ft}{23\ in}\right) + 1 = 6$ — Number of Camlocks Connecting panels

$V_{all\_cam} := 244\ lbf$ — Allowable Shear on Camlock (Testing Report)

$V_{all\_inplane} := N_{cam} \cdot V_{all\_cam} = 1464\ lbf$ — Allowable In-Plane Shear on Camlock (Testing Report)

CHECK: $V_{all\_inplane} \geq V = 1$

SUMMARY: THUS, CAMLOCKS @23" O.C. ARE SUFFICIENT TO WITHSTAND THE DIAPHRAGM SHEAR.

CHORD FORCE:

$F_{chord} := \dfrac{\dfrac{w_{design\_2} \cdot Width_2^{\ 2}}{8}}{Length_2} = 81.4\ lbf$ — Chord Force

CAM-LOCK:

$N_{cam} := 2$ — Number of Camlocks per Panel

$V_{all\_cam} := 244\ lbf$ — Allowable Wall to Ceiling Panel Shear on Camlock (Testing Report)

CHECK: $N_{cam} \cdot V_{all\_cam} \geq F_{chord} = 1$

SUMMARY: THUS, END CAMLOCKS ARE SUFFICIENT TO WITHSTAND THE CHORD FORCE.

SHEAR LOAD CALCULATIONS:

$L_2 := Width_2 - 3\ ft = 8.5\ ft$ Length of Gridline 1

$T_{width\_2} := Length_2 = 12.83\ ft$ Tributary Width

$f_2 := \dfrac{w_{design\_1} \cdot T_{width\_2}}{L_2} = 85.47\ plf$ In-Plane Force along Gridline 1

$L_B := 9.83\ ft$ Length of Gridline B

$T_{width\_B} := \dfrac{Width_2}{2} = 5.75\ ft$ Tributary Width

$f_{B2} := \dfrac{w_{design\_2} \cdot T_{width\_B}}{L_B} = 36.95\ plf$ In-Plane Force along Gridline B

$L_C := Length_2 = 12.83\ ft$ Length of Gridline C

$T_{width\_C} := \dfrac{Width_2}{2} = 5.75\ ft$ Tributary Width

$f_C := \dfrac{w_{design\_2} \cdot T_{width\_C}}{L_C} = 28.31\ plf$ In-Plane Force along Gridline A

OPEN FRONT DIAPHRAGM ANALYSIS:

$X := \dfrac{Length_2}{2} = 6.42\ ft$ Moment Arm

$F_p := w_{design\_1} \cdot Length_2 = 726.48\ lbf$

$M_F := F_p \cdot X = 4660.4\ ft \cdot lbf$ Max Moment

$R := \dfrac{M_F}{Width_2} = 405.25\ lbf$ Reaction Forces to Resist Imposed Moment

$w_B := \dfrac{R}{L_B} = 41.23\ plf$ Inplane Force Resisted on Wall Line B

$w_C := \dfrac{R}{L_C} = 31.59\ plf$ Inplane Force Resisted on Wall Line 3

$f_B := \max\left(f_{B2}, w_B\right) = 41.23\ plf$ $f_C := \max\left(f_C, w_C\right) = 31.59\ plf$

$R := \dfrac{H_w}{min\left(L_2, L_B, L_C\right)} = 0.90$ Worst Case Shape Ratio

$F_{all\_inplane} := 120\ plf$ Allowable In-Plane Shear (Testing Report)

CHECK: $F_{all\_inplane} \geq \max\left(f_1, f_{B2}, f_C\right) = 1$

SUMMARY: USE 4" THICK BASF HIGH DENSITY FOAM RAILS WALL PANELS FOR LATERAL RESISTANCE.

CEILING PANEL TO WALL PANEL CONNECTION

$H_w = 7.67\ ft$ — Unit Height

LOADS:

$P_{internal} = 5\ psf$ — Transverse Load on Wall

$p_{trans} := P_{internal} \cdot \frac{H_w}{2} = 19.18\ plf$ — Transverse Shear Force on Wall-Ceiling Connection

$f_{inplane} := \max\left(f_2, f_C, f_{B2}\right) = 85.47\ plf$ — In-Plane Shear Force on Wall-Ceiling Connection

$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 85.47\ plf$ — Governing Shear Force on Wall-Ceiling Connection

CAM-LOCK:

$S_{cam} := 23\ in$ — Spacing of Cam-lock

$V_{all\_cam} := \frac{244\ lbf}{S_{cam}} = 127.3\ plf$ — Allowable Shear on Camlock (Testing Report)

CHECK $f_{max} \le V_{all\_cam} = 1$

<u>SUMMARY: USE CAMLOCKS @23" O.C. FOR CEILING PANELS TO WALL PANEL CONNECTION.</u>

TYPICAL WALL TO FLOOR/CONCRETE CONNECTION

LOADS:

$P_{internal} = 5\ psf$	Transverse Load on Wall
$p_{trans} := P_{internal} \cdot \frac{H_w}{2} = 19.18\ plf$	Transverse Shear Force on Floor-Wall Connection
$f_{inplane} := \max\left(f_2, f_C, f_{B2}\right) = 85.47\ plf$	In-Plane Shear Force on Floor-Wall Connection
$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 85.47\ plf$	Governing Shear Force on Floor-Wall Connection

#10 TEK SCREWS:

$S_{screw} := 16\ in$	Spacing of Screw
$V_{all\_screw} := \frac{370\ lbf}{S_{screw}} = 277.5\ plf$	Allowable Shear Load (SSMA)
$T_{all\_screw} := \frac{137\ lbf}{S_{screw}} = 102.75\ plf$	Allowable Tension Load (SSMA)

3/8" HILTI KH-EZ ANCHOR:

$\Omega_o := 2.0$	Overstrength Factor
$S_{anchor} := 23\ in$	Spacing of Anchors
$v_{anchor} := \frac{\Omega_o \cdot f_{max} \cdot S_{anchor}}{0.7} = 468.04\ lbf$	LRFD Maximum Shear Force on Anchors
$V_{all\_anchor} := 1500\ lbf$	Allowable Shear on Anchor (See Anchor Report)

CHECK $f_{inplane} \le V_{all\_cam} = 1$ $p_{trans} \le T_{all\_screw} = 1$ $v_{anchor} \le V_{all\_anchor} = 1$

SUMMARY: USE 10"x1" S/S TEK SCREWS @ 16" O.C. AND 3/8"x3" HILTI-KH-EZ @23" O.C. FOR WALL TO FLOOR/CONCRETE CONNECTION.

TYPICAL WALL TO FLOOR/CONCRETE CONNECTION

LOADS:

$P_{internal} = 5\ psf$ Transverse Load on Wall

$p_{trans} := P_{internal} \cdot \frac{H_w}{2} = 19.18\ plf$ Transverse Shear Force on Floor-Wall Connection

$f_{inplane} := f_2 = 85.47\ plf$ In-Plane Shear Force on Floor-Wall Connection

$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 85.47\ plf$ Governing Shear Force on Floor-Wall Connection

#10 TEK SCREWS:

$S_{screw} := 16\ in$ Spacing of Screw

$V_{all\_screw} := \frac{370\ lbf}{S_{screw}} = 277.5\ plf$ Allowable Shear Load (SSMA)

$T_{all\_screw} := \frac{137\ lbf}{S_{screw}} = 102.75\ plf$ Allowable Tension Load (SSMA)

3/8" HILTI KH-EZ ANCHOR:

$\Omega_o := 2.0$ Overstrength Factor

$S_{anchor} := 16\ in$ Spacing of Anchors

$v_{anchor} := \frac{\Omega_o \cdot f_{max} \cdot S_{anchor}}{0.7} = 325.6\ lbf$ LRFD Maximum Shear Force on Anchors

$V_{all\_anchor} := 1500\ lbf$ Allowable Shear on Anchor (See Anchor Report)

CHECK $f_{inplane} \le V_{all\_cam} = 1$ $p_{trans} \le T_{all\_screw} = 1$ $v_{anchor} \le V_{all\_anchor} = 1$

SUMMARY: USE 10"x1" S/S TEK SCREWS @ 16" O.C. AND 3/8"x3" HILTI-KH-EZ @16" O.C. FOR WALL TO FLOOR/CONCRETE CONNECTION.

OVERTURNING CALCULATIONS (WORST CASE)

$DL_{panel} := 5\ psf$ — Panel Dead Load

$T_{width\_ceiling} := 3.92\ ft$ — Tributary Width of Ceiling

$H_w = 7.67\ ft$ — Height of Wall Panel

ASD LOADS:

$f := f_2 = 85.47\ plf$ — In-Plane Force on Wall

$L := L_2 = 8.5\ ft$ — Length of Wall

$S_{DS} = 0.548$ — Seismic Design Parameter

$Wt_{wall} := (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{panel} \cdot H_w \cdot L = 170.58\ lbf$ — Weight of Wall

$Wt_{ceiling} := (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 87.18\ lbf$ — Weight of Ceiling

$w_R := \frac{Wt_{wall} + Wt_{ceiling}}{L} = 30.32\ plf$ — Weight Resisting Overturning

$M_{wall} := f \cdot L \cdot H_w - w_R \cdot \frac{L^2}{2} = 4476.68\ lbf \cdot ft$ — Overturning Moment Acting on Wall

$w := \frac{3 \cdot M_{wall}}{L^2} = 185.88\ plf$ — Maximum Value of Overturning Force at End $\left(M_{wall} := \frac{1}{2} \cdot w \cdot L \cdot \frac{2\ L}{3} = 4476.68\ lbf \cdot ft\right)$

#10 TEK SCREWS:

$S_{screw} := 16\ in$ — Spacing of Screw

$V_{des\_screw} := 370\ lbf$ — Design Shear Load (SSMA)

$T_{des\_screw} := 137\ lbf$ — Design Tension Load (SSMA)

$V_{screw\_inplane} := f \cdot S_{screw} = 113.96\ lbf$ — Maximum Shear Force on End Screw due to Inplane Shear

$V_{screw\_uplift} := w \cdot S_{screw} = 247.84\ lbf$ — Maximum Shear Force on End Screw due to Uplift

$V_{screw} := \sqrt{V_{screw\_inplane}^2 + V_{screw\_uplift}^2} = 272.79\ lbf$ — Maximum Resultant Shear Force on End Screw

$T_{screw} := p_{trans} \cdot S_{screw} = 25.57\ lbf$ — Maximum Tension Force on End Screw

CHECK $V_{des\_screw} \ge V_{screw} = 1$ $T_{des\_screw} \ge T_{screw} = 1$

LRFD LOADS:

$f := \frac{f}{0.7} = 122.1\ plf$ — In-Plane Force on Wall

$Wt_{wall} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot H_w \cdot L = 257.65\ lbf$ — Weight of Wall

$Wt_{ceiling} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 131.68\ lbf$ — Weight of Ceiling

$Wt_{floor} := Wt_{ceiling} = 131.68\ lbf$ — Weight of Floor

$w_R := \frac{Wt_{wall} + Wt_{ceiling}}{L} = 45.8\ plf$ — Weight Resisting Overturning

$M_{wall} := \Omega_0 \cdot f \cdot L \cdot H_w - w_R \cdot \frac{L^2}{2} = 14265.73\ lbf \cdot ft$ — Overturning Moment Acting on Wall

$w := \frac{3 \cdot M_{wall}}{L^2} = 592.35\ plf$ — Maximum Value of Overturning Force at End $\left(M_{wall} := \frac{1}{2} \cdot w \cdot L \cdot \frac{2\ L}{3} = 14265.73\ lbf \cdot ft\right)$

3/8" HILTI KH-EZ ANCHOR:

$S_{anchor} := 16\ in$ — Spacing of Anchor

$\Omega_0 := 2.0$ — Overstrength Factor

$v_{anchor} := \Omega_0 \cdot f \cdot S_{anchor} = 325.6\ lbf$ — Ultimate Governing Shear on Anchor

$T_{anchor} := w \cdot S_{anchor} = 789.8\ lbf$ — Maximum Tension Force on End Anchor

NOTE: SEE APPENDIX FOR ANCHOR SOFTWARE PRINTOUTS FOR THE ANCHOR ANALYSIS.

SUMMARY: THUS, 10"x1" S/S TEK SCREWS @16" O.C. AND 3/8"x3" HILTI-KH-EZ @16" O.C. IS SUFFICIENT TO RESIST THE OVERTURNING FORCES.

DIAPHRAGM 3:

$Width_3 := 34.00\ ft$ — Diaphragm Width

$Length_3 := 8.50\ ft$ — Diaphragm Length

DESIGN CRITERIA:

$A_{ceiling} := Width_3 \cdot Length_3 = 289\ ft^2$ — Total Area of Ceiling

$L_{wall} := 2 \cdot Length_3 + 2 \cdot Width_3 = 85\ ft$ — Total Length of Walls

$Wt_{units} := 2 \cdot 115\ lbf = 230\ lbf$ — Total Unit Weight

$Wt_{steel} := 0\ lbf$ — Total Steel Weight

LATERAL FORCE GENERATION - ASCE 7 13.3.1:

$S_{DS} = 0.548$ — Spectral Acceleration, Short Period

$I_p := 1.0$ — Importance Factor (ASCE 7 13.1.3)

$z := H = 8\ ft$ — Height of Attachment with Respect to the Base

$h := H = 8\ ft$ — Average Roof Height of Structure

$H_w = 7.67\ ft$ — Wall Panel Height

$W_P := (DL_{panel} \cdot (A_{ceiling})) + (DL_{panel} \cdot H_w \cdot L_{wall}) + Wt_{units} + Wt_{steel} = 4934.75\ lbf$ — Operating Weight

$$F_p := \frac{0.4 \cdot a_p \cdot S_{DS} \cdot W_P}{\left(\frac{R_p}{I_p}\right)} \cdot \left(1 + 2 \cdot \frac{z}{h}\right) = 2163.39\ lbf$$ Seismic Force

$F_{p\_max} := 1.6 \cdot S_{DS} \cdot I_p \cdot W_P = 4326.79\ lbf$ — Maximum Lateral Seismic Force

$F_{p\_min} := 0.3 \cdot S_{DS} \cdot I_p \cdot W_P = 811.27\ lbf$ — Minimum Lateral Seismic Coefficient

$F_p := min(F_p, F_{p\_max}) = 2163.39\ lbf$

$F_p := \max(F_p, F_{p\_min}) = 2163.39\ lbf$ — Seismic Design Force

$F_{p\_vert} := 0.2 \cdot S_{DS} \cdot W_P = 540.85\ lbf$ — Vertical Seismic Design Force

$F_{p\_asd} := 0.7 \cdot F_p = 1514.38\ lbf$ — ASD Seismic Design Force

$$w_{design\_1} := \frac{F_{p\_asd}}{Length_3} = 178.16\ plf$$ Design Load in 1-1

$w_{design\_2} := \frac{F_{p\_asd}}{Width_3} = 44.54\ plf$ — Design Load in 2-2

DIAPHRAGM CHECK (1-1):

$Width_1 := Width_3 = 34\ ft$ — Width of Diaphragm (2-2)

$Length_1 := Length_3 = 8.5\ ft$ — Length of Diaphragm (2-2)

$R_2 := \frac{Length_1}{Width_1} = 0.25$ — Aspect Ratio (2-2)

$F_{all\_2} := 130\ plf$ — Allowable Diaphragm Capacity (Testing Report)

CHECK: $F_{all\_2} \geq \frac{w_{design\_2} \cdot Width_2}{Length_2} = 1$

SUMMARY: : USE 4" THICK BASF HIGH DENSITY FOAM RAILS CEILING PANELS.

CAMLOCK:

$V := w_{design\_2} \cdot Width_2 = 512.22\ lbf$ — Max Shear on Diaphram

$N_{cam} := ceil\left(\frac{Length_1}{23\ in}\right) \cdot 2 = 10$ — Number of Camlocks Connecting panels

$V_{all\_cam} := 244\ lbf$ — Allowable Shear on Camlock (Testing Report)

$V_{all\_inplane} := N_{cam} \cdot V_{all\_cam} = 2440\ lbf$ — Allowable In-Plane Shear on Camlock (Testing Report)

CHECK: $V_{all\_inplane} \geq V = 1$

SUMMARY: THUS, CAMLOCKS @23" O.C. ARE SUFFICIENT TO WITHSTAND THE DIAPHRAGM SHEAR.

CHORD FORCE:

$F_{chord} := \frac{\frac{w_{design\_2} \cdot Length_1^{\ 2}}{2}}{Width_1} = 47.32\ lbf$ — Chord Force

CAM-LOCK:

$N_{cam} := 2$ — Number of Camlocks per Panel

$V_{all\_cam} := 244\ lbf$ — Allowable Wall to Ceiling Panel Shear on Camlock (Testing Report)

CHECK: $N_{cam} \cdot V_{all\_cam} \geq F_{chord} = 1$

SUMMARY: THUS, END CAMLOCKS ARE SUFFICIENT TO WITHSTAND THE CHORD FORCE.

SHEAR WALL CALCULATIONS(2-2 Direction):

$d := \frac{Length_3}{2} = 4.25\ ft$ — Moment Arm

$M := F_{p\_asd} \cdot d = 6436.1\ ft \cdot lbf$ — Design Moment

$T := \frac{M}{Width_3} = 189.3\ lbf$ — Tension/ Comp. Force in chords

$L_1 := Width_3 = 34\ ft$ — Length of Gridline A

$T_{width\_1} := Length_3 = 8.5\ ft$ — Tributary Width

$f_1 := \frac{w_{design\_1} \cdot T_{width\_1}}{L_1} = 44.54\ plf$ — In-Plane Force along Gridline A

$L_A := Length_3 = 8.5\ ft$ — Length of Gridline A

$f_A := \frac{T}{2 \cdot L_A} = 11.14\ plf$ — In-Plane Force along Gridline A

$L_B := 9.83\ ft$ — Length of Gridline B

$f_{B3} := \frac{T}{2 \cdot L_B} = 9.63\ plf$ — In-Plane Force along Gridline B

$R := \frac{H_w}{min\left(L_1, L_A, L_B\right)} = 0.90$ — Worst Case Shape Ratio

$F_{all\_inplane} := 117\ plf$ — Allowable In-Plane Shear (Testing Report)

CHECK: $F_{all\_inplane} \geq max\left(f_1, f_A, f_{B3}\right) = 1$

SUMMARY: USE 4" THICK BASF HIGH DENSITY FOAM RAILS WALL PANELS FOR LATERAL RESISTANCE.

L-ANGLE CONNECTION TO EXISTING GROUTED CMU WALL

LOADS:

$W_{des} := w_{design\_2} = 44.54\ plf$ — Distributed Design Load

$t_{angle} := 0.036\ in$ — Thickness of Angle (20 GA)

$t_1 := t_{angle} = 0.036\ in$ — Thickness of Member in Contact with Screw Head or Washer

1/4" TITEN HD TO EXISTING GROUTED CMU WALL:

$S_{anchor} := 16\ in$ — Spacing of Anchor into Grouted CMU Wall

$\Omega_0 := 2.0$ — Overstrength Factor

$T_{anchor} := \dfrac{\Omega_0 \cdot W_{des} \cdot S_{anchor}}{0.7} = 169.68\ lbf$ — Tension on Anchors From Distributed Design Load

$T_{all\_anchor} := 410\ lbf$ — Allowable Tension on Anchor (Simpson Catalog)

CHECK $T_{anchor} \le T_{all\_anchor} = 1$

#14 TEK SCREW TO THE CEILING:

$N_{screw} := 1$ — No. of Screws

$S_{screw} := 12\ in$ — Spacing of Screws

$V_{all\_screw} := \dfrac{N_{screw} \cdot 60\ lbf}{S_{screw}} = 60\ plf$ — Allowable Shear on Screw (SSMA)

CHECK $V_{all\_screw} \ge W_{des} = 1$

ANGLE IN FLEXURE CHECK:
LOADS:

$t := t_{angle} = 0.036\ in$ — Uncoated Sheet Thickness

$L_{leg} := 2\ in$ — Leg Dimension

$d := 0.5\ in$ — Moment-Arm

$I := \frac{S_{anchor} \cdot t^3}{12} = 0.0001\ in^4$ $\quad y := \frac{t}{2} = 0.02\ in$

$S_x := \frac{I}{y} = 0.003\ in^3$

$F_y := 40\ ksi$ — Yield Stress of Steel

$M_{all} := \frac{F_y \cdot S_x}{1.67} = 6.9\ ft \cdot lbf$ — Yield Moment about x-axis (AISI S100-16)

$w_{design\_all} := \frac{M_{all}}{d \cdot S_{anchor}} = 124.17\ plf$ — Allowable Load in Angle

CHECK $\quad w_{design\_all} \geq W_{des} = 1$

ANGLE TENSILE STRENGTH (AISI S100 SECTION D):

$s := 12\ in$	Spacing between holes	$F_u := 42.1\ ksi$	Tensile Strength of Angle
$t = 0.036\ in$	Thickness of Angle	$F_y := 30\ ksi$	Yield Stress of Angle
$\Omega_{t\_y} := 1.67$	ASD Factor for Yielding	$\Omega_{t\_r} := 2.00$	ASD Factor for Rupture
$d_{dia} := 0.25\ in$	Diameter of Screw		
$A_g := t \cdot s = 0.43\ in^2$	Gross Area of Cross-section	$A_n := \left(s - \left(d_{dia} + \frac{1}{16}\ in\right)\right) \cdot t = 0.42\ in^2$	Gross Area of Cross-section

$T_{n\_y} := \frac{A_g \cdot F_y}{\Omega_{t\_y}} = 7760.48\ lbf$ — Allowable Tensile Strength due to Yielding

$T_{n\_r} := \frac{A_n \cdot F_u}{\Omega_{t\_r}} = 8856.79\ lbf$ — Allowable Tensile Strength due to Rupture

$T_{n\_all} := \frac{\min\left(T_{n\_y}, T_{n\_r}\right)}{s} = 7760.5\ plf$ — Allowable Tensile Strength/ft

CHECK $\quad T_{n\_all} \geq W_{des} = 1$

ANGLE COMPRESSIVE STRENGTH (AISI S100 SECTION E):

$L := 4\ in$	Length of Angle Leg	$E := 28000\ ksi$	Modulus of Elasticity of 316 SS
$t = 0.036\ in$	Thickness of Angle	$F_y := 30\ ksi$	Yield Stress of Angle
$\Omega_c := 1.80$	ASD Factor for Compression	$K := 1$	Effective length factor

$r := \dfrac{t}{\sqrt{12}} = 0.01\ in$ Radius of Gyration of full unreduced cross-section about axis of buckling

$S_l := \dfrac{K \cdot L}{r} = 384.9$ Slendreness Ratio

$F_{cre} := \dfrac{\pi^2 \cdot E}{S_l^{\ 2}} = 1865.4\ psi$ Elastic Flexural Buckling Stress

$\lambda_c := \sqrt{\dfrac{F_y}{F_{cre}}} = 4.01$

$F_n := \text{if}\left(\lambda_c \le 1.5, \left(0.658^{\lambda_c^{\ 2}}\right) \cdot F_y, \left(\dfrac{0.877}{\lambda_c^{\ 2}}\right) \cdot F_y\right) = 1635.9\ psi$ Compressive Stress

$A_g := t = 0.04\ in$ Gross Area/ft

$P_{n\_all} := \dfrac{F_n \cdot A_g}{\Omega_c} = 392.6\ plf$ Allowable Axial Strength/ft

CHECK $P_{n\_all} \ge W_{des} = 1$

<u>SUMMARY: THERFORE, PROVIDE A 20GA. CONTINUOUS ANGLE WITH 1/4" SIMSON ANCHOR @ 16" O.C.INTO THE EXISTING BUILDING GROUTED CMU WALL AND #14 TEK SCREWS @12" O.C INTO CEILING PANELS</u>

CEILING PANEL TO WALL PANEL CONNECTION

$H_w = 7.67\ ft$ — Unit Height

LOADS:

$P_{internal} = 5\ psf$ — Transverse Load on Wall

$p_{trans} := P_{internal} \cdot \frac{H_w}{2} = 19.18\ plf$ — Transverse Shear Force on Wall-Ceiling Connection

$f_{inplane} := \max\left(f_1, f_A, f_{B3}\right) = 44.54\ plf$ — In-Plane Shear Force on Wall-Ceiling Connection

$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 44.54\ plf$ — Governing Shear Force on Wall-Ceiling Connection

CAM-LOCK:

$S_{cam} := 23\ in$ — Spacing of Cam-lock

$V_{all\_cam} := \frac{244\ lbf}{S_{cam}} = 127.3\ plf$ — Allowable Shear on Camlock (Per LARR/Testing Report)

CHECK $f_{max} \le V_{all\_cam} = 1$

SUMMARY: USE CAMLOCKS @23" O.C. FOR CEILING PANELS TO WALL PANEL CONNECTION.

TYPICAL WALL TO FLOOR/CONCRETE CONNECTION

LOADS:

$P_{internal} = 5\ psf$ — Transverse Load on Wall

$p_{trans} := P_{internal} \cdot \frac{H_w}{2} = 19.18\ plf$ — Transverse Shear Force on Floor-Wall Connection

$f_{inplane} := \max\left(f_1, f_A, f_{B3}\right) = 44.54\ plf$ — In-Plane Shear Force on Floor-Wall Connection

$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 44.54\ plf$ — Governing Shear Force on Floor-Wall Connection

#10 TEK SCREWS:

$S_{screw} := 16\ in$ — Spacing of Screw

$V_{all\_screw} := \frac{370\ lbf}{S_{screw}} = 277.5\ plf$ — Allowable Shear Load (SSMA)

$T_{all\_screw} := \frac{137\ lbf}{S_{screw}} = 102.75\ plf$ — Allowable Tension Load (SSMA)

3/8" HILTI KH-EZ ANCHOR:

$\Omega_o := 2.0$ — Overstrength Factor

$S_{anchor} := 23\ in$ — Spacing of Anchors

$v_{anchor} := \frac{\Omega_o \cdot f_{max} \cdot S_{anchor}}{0.7} = 243.91\ lbf$ — LRFD Maximum Shear Force on Anchors

$V_{all\_anchor} := 1500\ lbf$ — Allowable Shear on Anchor (See Anchor Report)

CHECK $f_{inplane} \le V_{all\_cam} = 1$ $p_{trans} \le T_{all\_screw} = 1$ $v_{anchor} \le V_{all\_anchor} = 1$

SUMMARY: USE 10"x1" S/S TEK SCREWS @ 16" O.C. AND 3/8"x3" HILTI-KH-EZ @23" O.C. FOR WALL TO FLOOR/CONCRETE CONNECTION.

TYPICAL WALL TO FLOOR/CONCRETE CONNECTION

LOADS:

$P_{internal} = 5\ psf$ — Transverse Load on Wall

$p_{trans} := P_{internal} \cdot \dfrac{H_w}{2} = 19.18\ plf$ — Transverse Shear Force on Floor-Wall Connection

$f_{inplane} := f_{B2} + f_{B3} = 46.58\ plf$ — In-Plane Shear Force on Floor-Wall Connection

$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 46.58\ plf$ — Governing Shear Force on Floor-Wall Connection

#10 TEK SCREWS:

$S_{screw} := 16\ in$ — Spacing of Screw

$V_{all\_screw} := \dfrac{370\ lbf}{S_{screw}} = 277.5\ plf$ — Allowable Shear Load (SSMA)

$T_{all\_screw} := \dfrac{137\ lbf}{S_{screw}} = 102.75\ plf$ — Allowable Tension Load (SSMA)

3/8" HILTI KH-EZ ANCHOR:

$\Omega_o := 2.0$ — Overstrength Factor

$S_{anchor} := 23\ in$ — Spacing of Anchors

$v_{anchor} := \dfrac{\Omega_o \cdot f_{max} \cdot S_{anchor}}{0.7} = 255.09\ lbf$ — LRFD Maximum Shear Force on Anchors

$V_{all\_anchor} := 1500\ lbf$ — Allowable Shear on Anchor (See Anchor Report)

CHECK $f_{inplane} \le V_{all\_cam} = 1$ $p_{trans} \le T_{all\_screw} = 1$ $v_{anchor} \le V_{all\_anchor} = 1$

SUMMARY: USE 10"x1" S/S TEK SCREWS @ 16" O.C. AND 3/8"x3" HILTI-KH-EZ @23" O.C. FOR WALL TO FLOOR/CONCRETE CONNECTION.

OVERTURNING CALCULATIONS

$DL_{panel} := 5\ psf$ — Panel Dead Load

$T_{width\_ceiling} := 3.92\ ft$ — Tributary Width of Ceiling

$H_w = 7.67\ ft$ — Height of Wall Panel

ASD LOADS:

$f := f_{B2} + f_{B3} = 46.58\ plf$ — In-Plane Force on Wall

$L := L_B = 9.83\ ft$ — Length of Wall

$S_{DS} := 0.458$ — Seismic Design Parameter

$Wt_{wall} := (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{panel} \cdot H_w \cdot L = 202.02\ lbf$ — Weight of Wall

$Wt_{ceiling} := (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 103.25\ lbf$ — Weight of Ceiling

$w_R := \dfrac{Wt_{wall} + Wt_{ceiling}}{L} = 31.05\ plf$ — Weight Resisting Overturning

$M_{wall} := f \cdot L \cdot H_w - w_R \cdot \dfrac{L^2}{2} = 2011.65\ lbf \cdot ft$ — Overturning Moment Acting on Wall

$w := \dfrac{3 \cdot M_{wall}}{L^2} = 62.46\ plf$ — Maximum Value of Overturning Force at End
$\left(M_{wall} := \dfrac{1}{2} \cdot w \cdot L \cdot \dfrac{2\ L}{3} = 2011.65\ lbf \cdot ft \right)$

#10 TEK SCREWS:

$S_{screw} := 16\ in$ — Spacing of Screw

$V_{des\_screw} := 370\ lbf$ — Design Shear Load (SSMA)

$T_{des\_screw} := 137\ lbf$ — Design Tension Load (SSMA)

$V_{screw\_inplane} := f \cdot S_{screw} = 62.11\ lbf$ — Maximum Shear Force on End Screw due to Inplane Shear

$V_{screw\_uplift} := w \cdot S_{screw} = 83.27\ lbf$ — Maximum Shear Force on End Screw due to Uplift

$V_{screw} := \sqrt{V_{screw\_inplane}^2 + V_{screw\_uplift}^2} = 103.88\ lbf$ — Maximum Resultant Shear Force on End Screw

$T_{screw} := p_{trans} \cdot S_{screw} = 25.57\ lbf$ — Maximum Tension Force on End Screw

CHECK $V_{des\_screw} \geq V_{screw} = 1$ $T_{des\_screw} \geq T_{screw} = 1$

LRFD LOADS:

$f := \frac{f}{0.7} = 66.54\ plf$ — In-Plane Force on Wall

$Wt_{wall} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot H_w \cdot L = 304.75\ lbf$ — Weight of Wall

$Wt_{ceiling} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 155.75\ lbf$ — Weight of Ceiling

$Wt_{floor} := Wt_{ceiling} = 155.75\ lbf$ — Weight of Floor

$w_R := \frac{Wt_{wall} + Wt_{ceiling} + Wt_{floor}}{L} = 62.69\ plf$ — Weight Resisting Overturning

$M_{wall} := \Omega_0 \cdot f \cdot L \cdot H_w - w_R \cdot \frac{L^2}{2} = 7005.45\ lbf \cdot ft$ — Overturning Moment Acting on Wall

$w := \frac{3 \cdot M_{wall}}{L^2} = 217.5\ plf$ — Maximum Value of Overturning Force at End $\left(M_{wall} := \frac{1}{2} \cdot w \cdot L \cdot \frac{2\ L}{3} = 7005.45\ lbf \cdot ft \right)$

3/8" HILTI KH-EZ ANCHOR:

$S_{anchor} := 23\ in$ — Spacing of Anchor

$\Omega_0 := 2.0$ — Overstrength Factor

$v_{anchor} := \Omega_0 \cdot f \cdot S_{anchor} = 255.09\ lbf$ — Ultimate Governing Shear on Anchor

$T_{anchor} := w \cdot S_{anchor} = 416.87\ lbf$ — Maximum Tension Force on End Anchor

NOTE: SEE APPENDIX FOR ANCHOR SOFTWARE PRINTOUTS FOR THE ANCHOR ANALYSIS.

SUMMARY: THUS, 10"x1" S/S TEK SCREWS @16" O.C. AND 3/8"x3" HILTI-KH-EZ @23" O.C. IS SUFFICIENT TO RESIST THE OVERTURNING FORCES.

EVAPORATOR COIL LATERAL ANALYSIS

NOTE: WHETHER THE UNIT IS SUSPENDED FROM STEEL BEAMS OR SUSPENDED DIRECTLY FROM CEILING PANELS, THE TOP OF THE UNIT WILL BE FLUSH WITH THE BOTTOM OF THE CEILING PANELS. IN EITHER CASE, THE ALL-THREAD RODS WILL BEAR DIRECTLY ON THE STEEL SKIN OF THE PANELS. IF THE SKIN BEARING CAPACITY IS ADEQUATE TO CARRY THE REQUIRED SHEAR FORCE, THE LATERAL LOAD OF THE UNIT WILL TRANSFER INTO THE CEILING DIAPHRAGM WHICH IS TAKEN INTO ACCOUNT IN THE LATERAL ANALYSIS

$R_{p\_unit} := 1.5$	Mech. Unit Response Modification Factor
$a_{p\_unit} := 1.0$	Mech. Unit Amplification Factor
$Wt := 58\ lbf$	Unit Weight
$S_{DS} = 0.458$	Seismic Coefficient
$I_e := 1.5$	Importance Factor
$z := H = 8\ ft$	Height of Attachment
$h := H = 8\ ft$	Height of Diaphragm
$H_{unit} := 16.44\ in$	Height of Unit
$D_{unit} := 12\ in$	Depth of Unit
$f_p := \dfrac{0.4 \cdot a_{p\_unit} \cdot S_{DS} \cdot Wt}{\dfrac{R_{p\_unit}}{I_e}} \cdot \left(1 + 2\ \dfrac{z}{h}\right) = 31.88\ lbf$	Horizontal Seismic Force
$f_{max} := 1.6 \cdot S_{DS} \cdot I_e \cdot Wt = 63.75\ lbf$	Maximum Horizontal Force
$f_{min} := 0.3 \cdot S_{DS} \cdot I_e \cdot Wt = 11.95\ lbf$	Minimum Horizontal Force
$F_p := \max\left(f_{min}, \min\left(f_p, f_{max}\right)\right) = 31.88\ lbf$	Deisigned Horizontal Seismic Force
$M_{OT} := F_p \cdot \dfrac{H_{unit}}{2} = 21.84\ ft \cdot lbf$	Overturning Moment
$T_{OT} := \dfrac{M_{OT}}{D_{unit}} = 21.84\ lbf$	Tension due to Overturning Moment
$F_{p\_vert} := 0.2 \cdot S_{DS} \cdot Wt = 5.31\ lbf$	Concurrent Veritical Force
$n := 4$	Number of Bolt Connections on Coil
$t_{bolt} := \left\lvert \dfrac{Wt + F_{p\_vert}}{n} \right\rvert + \dfrac{T_{OT}}{\dfrac{n}{2}} = 26.75\ lbf$	Tension Load on Single Bolt

$v_{bolt} := \frac{F_p}{n} = 7.97\ lbf$ Shear Load on Single Bolt

$\Omega_{ASD} := 2.00$ ASD Safety Factor

$D_{bolt} := 0.375\ in$ Diameter of Bolt

$f_{nt} := 45\ ksi = 45000\ psi$ Tensile Strength of All-Thread

$f_{nv} := 27\ ksi = 27000\ psi$ Shear Strength of All-Thread

$R_{nt} := \left(\frac{D_{bolt}^2 \cdot \pi}{4}\right) \cdot \frac{f_{nt}}{\Omega_{ASD}} = 2485.05\ lbf$ Allowable Tensile Strength of All-Thread

$R_{nv} := \left(\frac{D_{bolt}^2 \cdot \pi}{4}\right) \cdot \frac{f_{nv}}{\Omega_{ASD}} = 1491.03\ lbf$ Allowable Shear Strength of All-Thread

PANEL SKIN BEARING STRENGTH:

$d := 0.375\ in$	Diameter of Bolt	$l_c := 1\ in$	Clear Edge Distance
$t := 0.0217\ in$	Thickness of Panel Skin	$F_u := 35\ ksi$	Tensile Strength of Panel Skin
$n_{skin} := 2$	(2) Skins Resisting	$\Omega_{ASD} := 2$	ASD Factor

$R_{n\_1} := 1.2\ l_c \cdot t \cdot F_u = 911.4\ lbf$

$R_{n\_2} := 2.4\ d \cdot t \cdot F_u = 683.55\ lbf$

$R_{n\_skin} := n_{skin} \cdot \frac{\min\left(R_{n\_1}, R_{n\_2}\right)}{\Omega_{ASD}} = 683.55\ lbf$ Bearing Capacity

CHECK: $R_{nt} \ge t_{bolt} = 1$ $R_{nv} \ge v_{bolt} = 1$ $R_{n\_skin} \ge v_{bolt} = 1$

SUMMARY: USE (4) 3/8" A307 BOLTS TO CARRY THE COILS W/ 4" THICK BASF HIGH DENSITY FOAM RAILS CEILING PANELS.

SOFTWARE PRINTOUTS (ENERCALC)

General Beam Analysis

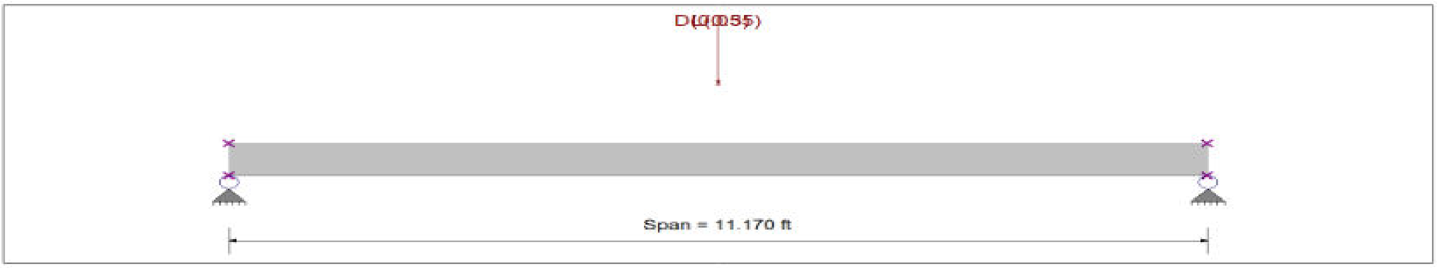
Project File: 24-23100.ec6

LIC# : KW-06013705, Build:20.23.2.14 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2022

DESCRIPTION: Ceiling Panel Analysis-(Cooler)

General Beam Properties

Elastic Modulus	29,000.0 ksi						
Span #1	Span Length =	11.170 ft	Area =	10.0 in^2	Moment of Inertia =	100.0	in^4



Applied Loads

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

Load(s) for Span Number 1

Point Load : L = 0.30 k @ 5.585 ft, (Maintenance Worker LL)

Point Load : D = 0.0550 k @ 5.585 ft, (Evap. Coil Load)

DESIGN SUMMARY

Maximum Bending =	0.991 k-ft	Maximum Shear =	0.1775 k
Load Combination	+D+L	Load Combination	+D+L
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	5.585 ft	Location of maximum on span	0.000 ft

Maximum Deflection

Max Downward Transient Deflection	0.005 in	25633
Max Upward Transient Deflection	0.000 in	0
Max Downward Total Deflection	0.006 in	21662
Max Upward Total Deflection	0.000 in	9391099

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0062	5.585		0.0000	0.000

Vertical Reactions

Support notation : Far left is # Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.178	0.178
Overall MINimum		
D Only	0.028	0.028
+D+L	0.178	0.178
+D+0.750L	0.140	0.140
+0.60D	0.017	0.017
L Only	0.150	0.150

SOFTWARE PRINTOUTS (ANCHOR)

HILTI

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Wall to Floor	Date:	1/11/2024
Fastening point:			

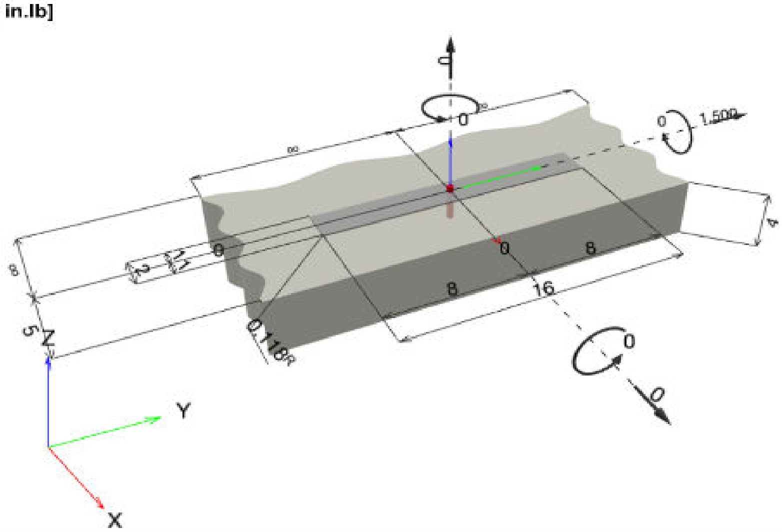
Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 3/8 (2 1/2)**
Item number:	418057 KH-EZ 3/8"x3"
Effective embedment depth:	$h_{ef,act}$ = 1.860 in., h_{nom} = 2.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	4/1/2022 | 12/1/2023
Proof:	Design Method ACI 318-19 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.118 in.
Anchor plate[R] :	l_x x l_y x t = 2.000 in. x 16.000 in. x 0.118 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 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.

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



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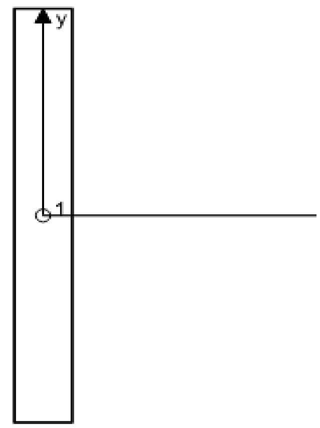
Phone I Fax:	|	E-Mail:	
Design:	Wall to Floor	Date:	1/11/2024
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 0$; $V_x = 0$; $V_y = 1{,}500$; $M_x = 0$; $M_y = 0$; $M_z = 0$;	yes	100

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	0	1,500	0	1,500

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 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*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A

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

HILTI

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Address:		Specifier:	
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Design:	Wall to Floor	Date:	1/11/2024
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*	1,500	1,866	81	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1,500	1,509	100	OK
Concrete edge failure in direction x+**	1,500	3,375	45	OK

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

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-3027

ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.09	120,300	0.600

Calculations

$V_{sa,eq}$ [lb]
3,110

Results

$V_{sa,eq}$ [lb]	ϕ_{steel}	$\phi_{nonductile}$	ϕ $V_{sa,eq}$ [lb]	V_{ua} [lb]
3,110	0.600	1.000	1,866	1,500

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4.2 Pryout Strength

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

$\phi\ V_{cp} \geq V_{ua}$ 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)

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

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ 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.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
1	1.860	5.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
2.920	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
31.14	31.14	1.000	1.000	2,156

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
2,156	0.700	1.000	1.000	1,509	1,500

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4.3 Concrete edge failure in direction x+

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ — ACI 318-19 Eq. (17.7.2.1a)

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

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\ c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
5.000	-	1.000	4.000	1.860

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.375	2,500	2.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
60.00	112.50	1.000	1.369	3,301

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
4,821	0.700	1.000	1.000	3,375	1,500

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5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 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 CBFEM 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!
- 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://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- "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.

Fastening meets the design criteria!

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6 Installation data

Profile: no profile

Hole diameter in the fixture: d_f = 0.500 in.

Plate thickness (input): 0.118 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) 3/8 (2 1/2)

Item number: 418057 KH-EZ 3/8"x3"

Maximum installation torque: 480 in.lb

Hole diameter in the base material: 0.375 in.

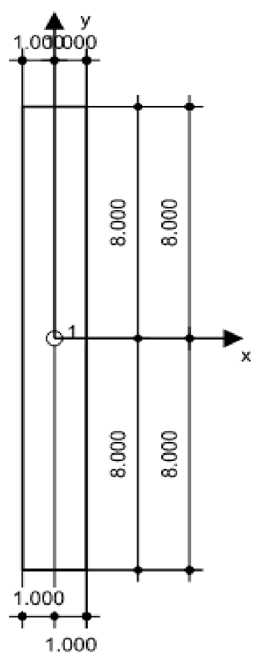
Hole depth in the base material: 2.750 in.

Minimum thickness of the base material: 4.000 in.

Hilti KH-EZ screw anchor with 2.5 in embedment, 3/8 (2 1/2), Carbon steel, installation per ESR-3027

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque wrench



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	-	5.000	-	-

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7 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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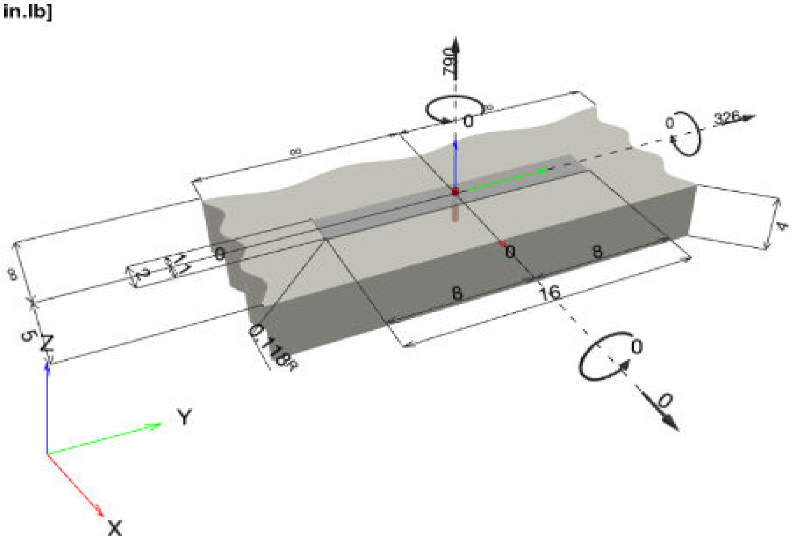
Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 3/8 (2 1/2)**
Item number:	418057 KH-EZ 3/8"x3"
Effective embedment depth:	$h_{ef,act}$ = 1.860 in., h_{nom} = 2.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	4/1/2022 | 12/1/2023
Proof:	Design Method ACI 318-19 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.118 in.
Anchor plate[R] :	l_x x l_y x t = 2.000 in. x 16.000 in. x 0.118 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 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.

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



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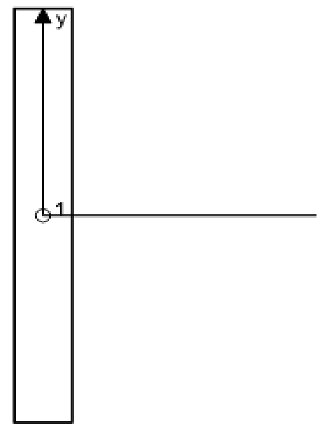
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 790$; $V_x = 0$; $V_y = 326$; $M_x = 0$; $M_y = 0$; $M_z = 0$;	yes	76

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	790	326	0	326

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 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*	790	6,718	12	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	790	1,051	76	OK

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

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3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3027

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

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.09	120,300

Calculations

N_{sa} [lb]
10,335

Results

N_{sa} [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
10,335	0.650	1.000	6,718	790

3.2 Concrete Breakout Failure

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

$\phi\ N_{cb} \geq N_{ua}$ 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)

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

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ 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.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.860	5.000	1.000	2.920	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
31.14	31.14	1.000	1.000	2,156

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
2,156	0.650	0.750	1.000	1,051	790

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4 Shear load

	Load V_{ua} [lb]	Capacity $\phi\ V_n$ [lb]	Utilization $\beta_V = V_{ua}/\phi\ V_n$	Status
Steel Strength*	326	1,866	18	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	326	1,509	22	OK
Concrete edge failure in direction x+**	326	3,375	10	OK

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

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-3027

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

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.09	120,300	0.600

Calculations

$V_{sa,eq}$ [lb]
3,110

Results

$V_{sa,eq}$ [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ V_{sa,eq}$ [lb]	V_{ua} [lb]
3,110	0.600	1.000	1,866	326

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4.2 Pryout Strength

$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ed,N} \, \psi_{c,N} \, \psi_{cp,N} \, N_b \right]$	ACI 318-19 Eq. (17.7.3.1a)
$\phi \; V_{cp} \geq V_{ua}$	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)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.4.1b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$	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.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
1	1.860	5.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
2.920	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
31.14	31.14	1.000	1.000	2,156

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi \; V_{cp}$ [lb]	V_{ua} [lb]
2,156	0.700	1.000	1.000	1,509	326

4.3 Concrete edge failure in direction x+

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b$ ACI 318-19 Eq. (17.7.2.1a)

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

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vc0} = 4.5\, c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\, c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
5.000	-	1.000	4.000	1.860

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.375	2,500	2.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
60.00	112.50	1.000	1.369	3,301

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
4,821	0.700	1.000	1.000	3,375	326

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

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.752	0.216	5/3	70	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 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 CBFEM 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!
- 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://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- "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.

Fastening meets the design criteria!

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7 Installation data

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) 3/8 (2 1/2)

Item number: 418057 KH-EZ 3/8"x3"

Maximum installation torque: 480 in.lb

Hole diameter in the base material: 0.375 in.

Hole depth in the base material: 2.750 in.

Minimum thickness of the base material: 4.000 in.

Profile: no profile

Hole diameter in the fixture: d_f = 0.500 in.

Plate thickness (input): 0.118 in.

Recommended plate thickness: not calculated

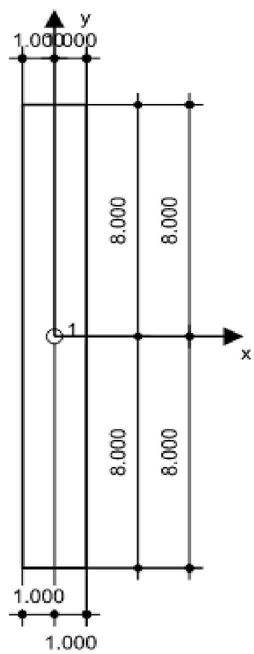
Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti KH-EZ screw anchor with 2.5 in embedment, 3/8 (2 1/2), Carbon steel, installation per ESR-3027

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque wrench



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	-	5.000	-	-

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8 Remarks; Your Cooperation Duties

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

TESTING REPORT OR LARR

TAMARACK GROVE ENGINEERING

PANEL CONSTRUCTION AND DETAILS

Prefabricated panels are manufactured up to 46 inches wide and in a minimum thickness of 4 inches with metal facings consisting of a minimum 26-gauge (0.019-inch base metal thickness) galvanized steel complying with ASTM A653 CS Type A that are textured to a stucco embossed galvanized finish. The facing can also be of minimum 0.032" aluminum complying with ASTM B209, or min. 26-gauge stainless steel complying with ASTM A240.

The 2.25-pcf nominal density core material consists of Class 1 two-part component urethane foam. The foam core insulation itself has a smoke density and flame spread ratings of less than or equal to 450 and less than or equal to 25, respectively when tested in accordance with ASTM E84.

The panels are framed around the perimeter with high-density foam rails made of BASF two-part component urethane with a minimum density of 8 pcf. The panels have tongue and groove edges and are connected using cam-locking devices.

The panels are approved as structural wall and ceiling panels for use in interior and exterior, non-fire rated walk-in cooler and freezers as load bearing walls, roof panels, shear walls, and diaphragms.

The data and tables presented here are subject to the following conditions:

1. The cooler and freezers shall be limited to locations where combustible construction is permitted by the governing 2021 International Building Code (IBC) and any local code amendments.

2. Materials for the panel construction shall be as specified above. Test data by certified testing agency shall be submitted upon request.

3. Complete plans and design calculations bearing the signature of a registered civil or structural engineer shall be submitted to the structural plan check for their approval for each job. Wall panels shall be connected to the supporting structure with fasteners complying with the building code and shall be detailed on the approved plans.

4. A thermal barrier is required per Section 2603.4 of the IBC unless exempted per Section 2603.4.1.

5. An approved fire-retardant roof covering (Class "A" or "B") shall be placed over the panels when used as exterior roof panels.

6. A separate approval from the Electrical Testing Laboratory shall be required for electrical installations within the panels.

7. Design of a building utilizing the panels shall be in accordance with the requirements of the IBC and the design data specified in the tables below: