DOUGLAS D. HARDIN, P.E.
STRUCTURAL ENGINEERING CALCULATIONS
FOR
KPS GLOBAL

COSTCO 660

WALK-INS
PUYALLUP, WASHINGTON
TGE PROJECT NUMBER: 21-18278

DOUGLAS DEAN HARDIN
STATE OF WASHINGTON
20105231
REGISTERED
PROFESSIONAL ENGINEER
expires 6-09-23
12/03/2021

812 LA CASSIA DR. BOISE, IDAHO 83705 (208) 345-8941 PHONE (208) 345-8946 FAX

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PROJECT INFORMATION

Tamarack Grove Engineering:

Address:	812 La Cassia Dr Boise, Idaho 83705
Date:	12/3/2021
Firm Registration Number:	603490470
TGE Engineer of Record:	Douglas D. Hardin, P.E.
Project Manager:	Ruchin Khadka
Direct Phone:	(208) 779-4321
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Project Client Information:

Company:	KPS Global
Project Number:	FT31124
Contact:	Glenn Shuping
Address:	4201 N. Beach St. Fort Worth, TX 76137
Phone:	(682) 317-5357
Email:	Glenn.Shuping@kpsglobal.com
Client Logo:	KPS GLOBAL

Project Site Information:

Name:	Costco 660
Address:	1201 39th Ave SW Puyallup, Washington 98373
Client Reference Number:	FT31124

Local Jurisdiction Information:

Jurisdiction:	Pierce County
Enforced Code Used:	2018 International Building Code
Contact Info:	https://www.piercecountywa.gov/

Project Scope of Work:

Tamarack Grove Engineering is providing structural engineering calculations for the walk-in manufactured by KPS Global 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 STRUTCTURAL 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 engineer of record is not responsible for any deviations from these plans unless such changes are authorized in writing to the engineer of record.
   - D. 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.
   - E. Notching and/or cutting of any structural member in the field is prohibited, unless prior consent is given by the engineer of record.
   - F. 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.
   - G. The assumed thickness of existing concrete will be 6” with an f’c of 3,000 psi, unless otherwise noted in calculations.

2. Structural Steel
   - A. All steel construction shall conform to requirements set forth in the latest editions of AISC, “American Institute of Steel Construction”, AISC 341-16, “Seismic Provisions for Structural Steel Buildings, Including Supplement No. 1, dated 2016” and AISC 360-16, “Specifications for Structural Steel Buildings”.
   - B. Steel Designations:
     1. Wide Flange Shapes (Beams & Columns).. ASTM A992 (GRADE 50)
     2. Other Rolled Shapes & Plate... ASTM A36 (U.N.O.)
     3. Pipe Columns... ASTM A53, GRADE 'B'
     4. Structural HSS Tubing.. ASTM A500, GRADE 'B' 46 KSI
   - C. All anchor bolts, bolts and lags in wood shall conform to ASTM A307 steel U.N.O. and shall have steel washers beneath all nuts and bolt heads. If a certain situation is not detailed use a similar detail. All structural bolts shall conform to ASTM A307. Connections shall generally follow the types shown in AISC manual of steel construction. joints are ‘snug tightened’ unless otherwise detailed per AISC.
   - D. Steel fabricator shall also include and coordinate all structural steel shown on architectural sheets with that of the structural sheets. Coordinate any steel not shown on structural drawings, contractor to verify.
   - E. All bearing elevations for joists, beams, and column heights shall be coordinated and verified by the contractor. All elevations must be approved by engineer and architect of record in the shop drawing review process.
   - F. All steel welding shall conform to AWS D1.1 with E70XX electrodes.
   - G. Provide high strength non-shrink grout under all steel base plates, f’c = 5,000 psi, min.

3. 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 2018 International Building Code (2018 IBC), 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 2018 IBC.
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: KPS Global

Analysis Method / Report Used: LARR 24921

Dead/ Live Load Information Per ASCE 7:

DL_{panel} =	5.00 psf	Steel Facing (ASTM-A-646) Weight =	1.80 psf
LL_{panel} =	10.00 psf	Insulation Weight =	0.75 psf
LL_{panel_acc} =	20.00 psf	Rail Weight =	0.45 psf
$L_{internal}$ =	5.00 psf	Miscellaneous =	2.00 psf
P_{LL} =	300 lbf		

(Dead Load Calculation)

Seismic Load Information Per ASCE 7:

BSC =	D - Default		F_a =	1.200	Table 11.4-1
I_E =	1.0	Table 1.5-2	F_v =	1.863	Table 11.4-2
SDC =	D	Sec. 11.6	S_{MS} =	1.521	$S_{MS} = F_a * S_S$, Equation 11.4-1
S_S =	1.267	Per ATC Hazards	S_{M1} =	0.814	$S_{M1} = F_v * S_1$, Equation 11.4-2
S_1 =	0.437	Per ATC Hazards	S_{DS} =	1.014	$S_{DS} = 2/3 * S_{MS}$, Equation 11.4-3
T_L =	6	Per ATC Hazards	S_{D1} =	0.543	$S_{D1} = 2/3 * S_{M1}$, Equation 11.4-4

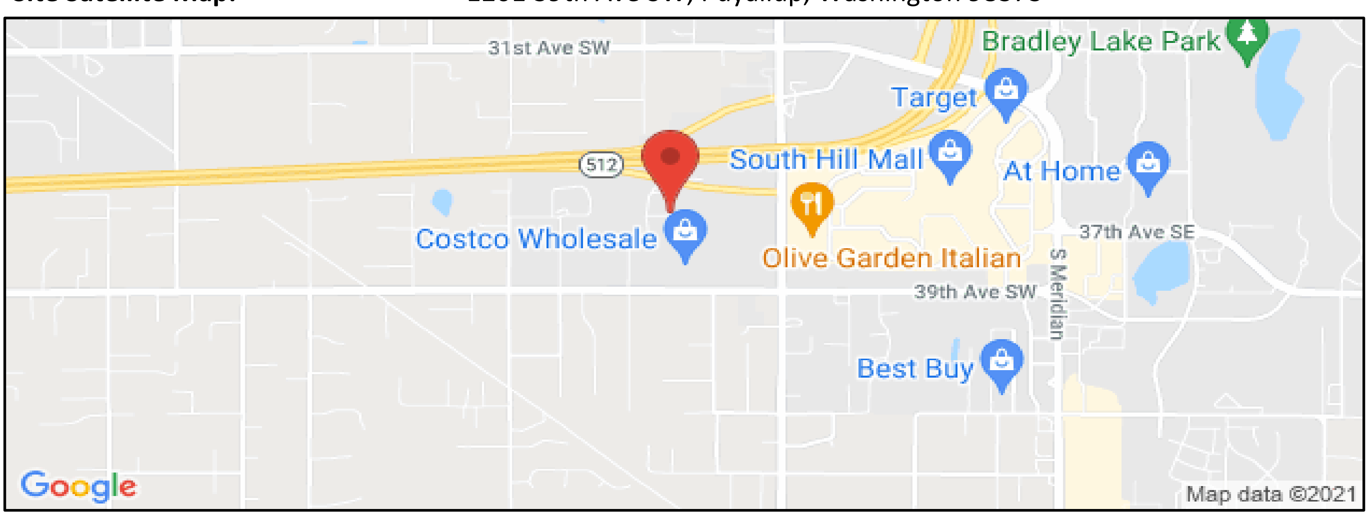
Snow Design Information Per ASCE 7:

Not Applicable

Wind Design Information:

Not Applicable

Site Satellite Map: 1201 39th Ave SW, Puyallup, Washington 98373



STRUCTURAL CALCULATIONS CBX (COMBO BOX)

JURISDICTION INFORMATION

JURISDICTION: PUYALLUP, WASHINGTON
STRUCTURAL CODE: 2018 INTERNATIONAL BUILDING CODE

PASS = 1.0 FAILURE = 0

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 7-10/16)

$LC_3 := DL_{panel} + LL_{panel} = 15\ psf$	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 <UNIT 1>,<UNIT 2> & <UNIT 5>

$Width := 45.25\ ft$	Unit Width
$Length := 23.79\ ft$	Unit Length
$H_{1f} := 10.48\ ft$	Unit Height
$H_{1w} := 10.06\ ft$	Wall Height
$H_{2f} := 20.00\ ft$	Unit Height
$H_{2w} := 19.58\ ft$	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

$L := 15.42\ ft$	Ceiling Panel Span
$L_{all} := 16.42\ ft$	Allowable Span (Per LARR/Testing Report)
$T_{width\_panel} := 47\ in$	Tributary Width of Panel

LOADS:

$LL_{panel} := 10\ psf$	Panel Live Load - Not Accessible
$w_{design\_ceiling} := LL_{panel} \cdot T_{width\_panel} = 39.17\ plf$	Distributed Live Load
$P_{LL} := 300\ lbf$	Maintenance Worker Live Load
$m_{max} := \max\left(\frac{w_{design\_ceiling} \cdot L^2}{8}, \frac{P_{LL} \cdot L}{4}\right) = 1164.11\ ft \cdot lbf$	Maximum Moment
$w_{all} := 20\ psf \cdot T_{width\_panel} = 78.33\ plf$	Allowable Panel Load (Per LARR/Testing Report)
$M_{allow} := \frac{w_{all} \cdot L_{all}^{\ 2}}{8} = 2639.99\ ft \cdot lbf$	Allowable Moment

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

<u>SUMMARY: USE SPECIFIED CEILING PANELS PER PLANS.</u>

CEILING SUPPORT BEAMS AND COLUMNS

<UNIT 1> W12X40 ANALYSIS:

$L := 22.83\ ft$	Design Length
$T_{width} := \frac{30.34}{2}\ ft = 15.17\ ft$	Tributary Width of Ceiling on Beam

LOADS:

$w_{DL} := DL_{panel} \cdot T_{width} = 75.85\ plf$	Distributed Dead Load on Beam
$w_{LL} := LL_{panel} \cdot T_{width} = 151.7\ plf$	Distributed Live Load on Beam

REACTIONS (ENERCALC):

$R_{DL\_1} := 1322\ lbf$	Dead Load Reaction on Beam
$R_{LL\_1} := 1732\ lbf$	Live Load Reaction on Beam
$\delta_{DL} := 0.185\ in - 0.105\ in = 0.08\ in$	Deflection due to Dead Load

BEAM STRAPS ANALYSIS:

$d := 12\ in$	Beam Depth
$M_{max} := 17.431\ kip \cdot ft$	Maximum Moment (ENERCALC)

LOADS:

$P_{axial} := \frac{M_{max}}{d} = 17431\ lbf$	Maximum Axial Force
$P_{design\_brace} := 0.02 \cdot P_{axial} = 348.62\ lbf$	Design Tensile Force on Brace

BRACING ELEMENT:

$\Omega := 1.67$	ASD Factor (Tension)
$F_y := 33\ ksi$	Nominal Yield Strength of Brace
$E := 29000\ ksi$	Modulus of Elasticity for Steel
$w_{brace} := 3\ in$	Width of Brace
$t_{brace} := 0.048\ in$	Thickness of Brace
$A_g := w_{brace} \cdot t_{brace} = 0.14\ in^2$	Gross Cross-Sectional Area of Brace

TENSILE YIELDING:

$T_n := F_y \cdot A_g = 4752\ \mathit{lbf}$ Nominal Yield Strength of Brace

$T_{all} := \frac{T_n}{\Omega} = 2845.509\ \mathit{lbf}$ Allowable Yield Strength of Brace

TENSILE RUPTURE:

$F_u := 45\ \mathit{ksi}$ Tensile Strength

$b_{edge} := 0.75\ \mathit{in}$ Actual Edge Distance

$b_e := min\left(2 \cdot t_{brace} + 0.63\ \mathit{in}, b_{edge}\right) = 0.73\ \mathit{in}$ Design Edge Distance

$\Omega := 2.00$ ASD Adjustment Factor

$A_n := 2 \cdot t_{brace} \cdot b_e = 0.07\ \mathit{in}^2$ Net Area of Plate Resisting Shear

$R_{nt} := \frac{F_u \cdot A_n}{\Omega} = 1.57\ \mathit{kip}$ Allowable Force on Plate

BEARING STRENGTH:

$A_{pb} := 2 \cdot b_e \cdot t_{brace} = 0.07\ \mathit{in}^2$ Area of Plate Resisting Bearing Force

$R_{nb} := \frac{1.8 \cdot F_y \cdot A_{pb}}{\Omega} = 2.07\ \mathit{kip}$ Allowable Bearing Strength

BEAM FASTENER CAPACITY:

$n_{screws\_beam} := 1$ Number of Screws

$V_{all\_screw\_beam} := n_{screws\_beam} \cdot 486\ \mathit{lbf} = 486\ \mathit{lbf}$ Allowable Shear of Tek Screw (ESR-1976)

CEILING FASTENER CAPACITY:

$n_{screws\_ceil} := 1$ Number of Screws

$V_{all\_screw\_ceil} := n_{screws\_ceil} \cdot 76\ \mathit{lbf} = 76\ \mathit{lbf}$ Allowable Shear of Tek Screw (ESR-1976)

REQUIRED NUMBER OF BRACES:

$P_{all} := min\left(T_{all}, R_{nt}, R_{nb}, V_{all\_screw\_beam}, V_{all\_screw\_ceil}\right) = 76\ \mathit{lbf}$ Governing Allowable Load

$n_{reqd} := ceil\left(\frac{P_{design\_brace}}{P_{all}}\right) = 5$ Minimum Number of Braces Required

NOTE: PROVIDE BRACES AT EACH END AND EVENLY SPACED.

<UNIT 1> W12X22 ANALYSIS:

$L := 18.00\ ft$ — Design Length

LOADS:

$R_{DL\_1} = 1322\ lbf$ — Dead Load Reaction on Beam

$R_{LL\_1} = 1732\ lbf$ — Live Load Reaction on Beam

$a := 1.25\ ft$ $\quad b := 16.75\ ft$ — Locations of W12x35 Reactions

REACTIONS (ENERCALC):

$R_{DL\_2} := 1520\ lbf$ — Dead Load Reaction on Column

$R_{LL\_2} := 1732\ lbf$ — Live Load Reaction on Column

$\delta_{DL} := 0.071\ in - 0.033\ in = 0.038\ in$ — Deflection due to Dead Load

W12X22 STIFFENER CHECK (BRIDGE BEAM):
MEMBER PROPERTIES

$k_1 := 0.625\ in$ $\quad t_w := 0.260\ in$ $\quad b_f := 4.03\ in$ $\quad T := 10.375\ in$ $\quad F_y := 50\ ksi$

$k := 0.725\ in$ $\quad d := 12.3\ in$ $\quad t_f := 0.425\ in$ $\quad l_b := 2 \cdot k_1 = 1.25\ in$ $\quad E := 29000\ ksi$

$Q_f := 4.76$ $\quad h := 41.8 \cdot t_w = 10.87\ in$ $\quad S_x := 25.4\ in^3$

LOADS

$R_{DL\_1} = 1322\ lbf$ — Dead Load Acting on Beam

$R_{LL\_1} = 1732\ lbf$ — Live Load Acting on Beam

$LC_3 := R_{DL\_1} + R_{DL\_1} = 2644\ lbf$ — Load Combination 3: D+(Lr, S, or R)

WEB LOCAL YIELDING

$\Omega_y := 1.50$ — ASD Factor

$d = 12.3\ in$ — Depth of Member

$a := 1.25\ ft$ — Location of Beam on Bridge Beam

$f_{a\_y} := \textbf{if}\left(d \ge a, 2.5, 5\right) = 5$ — Factor for Location of Beam Loading (Yielding)

$$R_{n\_y} := \frac{\left(f_{a\_y} \cdot k + l_b\right)\ F_y \cdot t_w}{\Omega_y} = 42.25\ kip$$ — Allowable Yielding of Beam

$\textbf{if}\left(LC_3 \ge R_{n\_y}, \text{“Stiffener Required”}, \text{“Stiffener Not Required”}\right) = \text{“Stiffener Not Required”}$

WEB LOCAL CRIPPLING

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

$d = 12.3\ in$ Depth of Member

$a = 1.25\ ft$ Location of Beam on Bridge Beam

$f_{a\_c} := \mathbf{if}\left(\frac{d}{2} \geq a, 0.4, 0.8\right) = 0.8$ Factor for Location of Beam Loading

$f_{lb\_d} := \mathbf{if}\left(\frac{l_b}{d} \leq 0.2, 3 \cdot \left(\frac{l_b}{d}\right), \frac{4 \cdot l_b}{d} - 0.2\right) = 0.3$ Factor for Ratio of Bearing Length to Beam Depth

$$R_{n\_c} := \frac{\left(f_{a\_c} \cdot t_w^{\ 2} \cdot \left(1 + f_{lb\_d} \cdot \left(\frac{t_w}{t_f}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_y \cdot t_f}{t_w}}\right)}{\Omega_{cr}} \cdot Q_f = 227062.53\ lbf$$

Allowable Beam Crippling

$\mathbf{if}\left(LC_3 \geq R_{n\_c}, \text{“Stiffener Required”}, \text{“Stiffener Not Required”}\right) = \text{“Stiffener Not Required”}$

WEB SIDEWAYS BUCKLING:

$\Omega_{wsb} := 1.76$ ASD Factor

$L_b := b - a = 15.5\ ft$ Unbraced Length

$V_b := 3252\ lbf$ Shear Force at Support(Enercalc)

$X := 1.25\ ft$ Distance From Support to Load Location

$M_a := V_b \cdot X = 4065\ lbf \cdot ft$ Moment at Location of Load (Enercalc)

$M_y := F_y \cdot S_x = 105833.33\ ft \cdot lbf$ Yield Moment About the Axis of Bending

$C_r := \mathbf{if}\left(1.5 \cdot M_a < M_y, 960000\ ksi, 480000\ ksi\right) = 960000\ ksi$

$$R_n := \frac{C_r \cdot t_w^{\ 3} \cdot t_f}{h^2} \cdot \left(1 + 0.4 \left(\frac{\frac{h}{t_w}}{\frac{L_b}{b_f}}\right)^3\right) = 78.75\ kip$$

Nominal Sideways Buckling Capacity

$$R_a := \mathbf{if}\left(\frac{\frac{h}{t_w}}{\frac{L_b}{b_f}} > 2.3, \text{“No Sideways Buckling”}, \frac{R_n}{\Omega_{wsb}}\right) = 44746.19\ lbf$$

$\mathbf{if}\left(LC_3 \geq R_a, \text{“Stiffener Required”}, \text{“Stiffener Not Required”}\right) = \text{“Stiffener Not Required”}$

W12X40 STIFFENER CHECK (BEAM FRAMING INTO BRIDGE BEAM):

MEMBER PROPERTIES

$k_1 := 0.875\ in$ $t_w := 0.295\ in$ $b_f := 8.01\ in$ $T := 9.25\ in$ $F_y := 50\ ksi$

$k := 1.02\ in$ $d := 11.9\ in$ $t_f := 0.515\ in$ $l_b := 2 \cdot k_1 = 1.75\ in$ $E := 29000\ ksi$

$Q_f := 11.3$ $h := 36.2 \cdot t_w = 10.68\ in$

LOADS

$R_{DL\_2} = 1520\ lbf$ Dead Load Acting on Beam

$R_{LL\_2} = 1732\ lbf$ Live Load Acting on Beam

$LC_3 := R_{DL\_1} + R_{LL\_1} = 3054\ lbf$ Load Combination 3: D+(Lr, S, or R)

WEB LOCAL YIELDING

$\Omega_y := 1.50$ ASD Factor

$d = 11.9\ in$ Depth of Member

$a := 0\ ft$ Location of Beam on Bridge Beam

$f_{a\_y} := \textbf{if}\left(d \ge a, 2.5, 5\right) = 2.5$ Factor for Location of Beam Loading (Yielding)

$R_{n\_y} := \dfrac{\left(f_{a\_y} \cdot k + l_b\right) F_y \cdot t_w}{\Omega_y} = 42283.33\ lbf$ Allowable Yielding of Beam

$\textbf{if}\left(LC_3 \ge R_{n\_y}, \text{“Stiffener Required”}, \text{“Stiffener Not Required”}\right) = \text{“Stiffener Not Required”}$

WEB LOCAL CRIPPLING

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

$d = 11.9\ in$ Depth of Member

$a = 0\ ft$ Location of Bridge Beam on Gravity Beam

$f_{a\_c} := \textbf{if}\left(\dfrac{d}{2} \ge a, 0.4, 0.8\right) = 0.4$ Factor for Location of Beam Loading (Crippling)

$f_{lb\_d} := \textbf{if}\left(\dfrac{l_b}{d} \le 0.2, 3 \cdot \left(\dfrac{l_b}{d}\right), \dfrac{4 \cdot l_b}{d} - 0.2\right) = 0.44$ Factor for Ratio of Bearing Length to Beam Depth

$R_{n\_c} := \dfrac{\left(f_{a\_c} \cdot {t_w}^2 \cdot \left(1 + f_{lb\_d} \cdot \left(\dfrac{t_w}{t_f}\right)^{1.5}\right) \cdot \sqrt{\dfrac{E \cdot F_y \cdot t_f}{t_w}}\right)}{\Omega_{cr}} \cdot Q_f = 372766.87\ lbf$ Allowable Beam Crippling

$\textbf{if}\left(LC_3 \ge R_{n\_c}, \text{“Stiffener Required”}, \text{“Stiffener Not Required”}\right) = \text{“Stiffener Not Required”}$

WEB SIDEWAYS BUCKLING:

$\Omega_{wsb} := 1.76$ — ASD Factor

$L_b := 48\ in$ — Unbraced Length

$M_a := 17431\ lbf \cdot ft$ — Moment at Location of Load (Enercalc)

$M_y := F_y \cdot S_x = 105833.33\ ft \cdot lbf$ — Yield Moment About the Axis of Bending

$C_r := \mathbf{if}\left(1.5 \cdot M_a < M_y, 960000\ ksi, 480000\ ksi\right) = 960000\ ksi$

$$R_n := \frac{C_r \cdot {t_w}^3 \cdot t_f}{h^2} \cdot \left(1 + 0.4 \left(\frac{\frac{h}{t_w}}{\frac{L_b}{b_f}}\right)^3\right) = 9925.23\ kip$$

Nominal Sideways Buckling Capacity

$$R_a := \mathbf{if}\left(\frac{\frac{h}{t_w}}{\frac{L_b}{b_f}} > 2.3, \text{“No Sideways Buckling”}, \frac{R_n}{\Omega_{wsb}}\right) = \text{“No Sideways Buckling”}$$

<u>SUMMARY: CONNECT SUPPORT BEAM TO BRIDGE BEAM PER PLANS.</u>

Material Specification

HSS 5x5x3/16"<UNIT 1> — A500 GRADE B

$H_{column} := H_{2w} - 12\ in = 18.58\ ft$ — Height of a Column

LOADS:

$p_{column\_DL} := \max\left(R_{DL\_1}, R_{DL\_2}\right) = 1520\ lbf$ — Dead Load on Column

$p_{column\_LL} := \max\left(R_{LL\_1}, R_{LL\_2}\right) = 1732\ lbf$ — Live Load on Column

NOTE: SEE THE ENERCALC SOFTWARE PRINTOUTS IN DESIGN AIDS FOR MEMBER ANALYSIS.

COLUMN REACTIONS (ENERCALC):

$R_{DL\_base} := 1742\ lbf$ — Dead Load at Base Plate

$R_{LL\_base} := 1732\ lbf$ — Live Load at Base Plate

BASE PLATE THICKNESS:

$b_f := 5\ in$ $d_f := 5\ in$ Column Dimensions

$\lambda := 1$ Normal Weight Concrete Factor

$B := 6\ in$ $N := 10\ in$ Base Plate Dimensions

$n := (B - 0.8 \cdot b_f) \cdot 0.5 = 1\ in$

$m := (N - .95 \cdot d_f) \cdot 0.5 = 2.63\ in$

$n' := \frac{\sqrt{b_f \cdot d_f}}{4} = 1.25\ in$ Yield-Line Theory Cantilever Distance From Column Web or Column Flange

$l := \max(m, n, \lambda \cdot n') = 2.63\ in$

$F_y := 38\ ksi$ Base Plate Yield Strength

$\Omega := 1.67$ ASD Factor

$t_{min} := l \cdot \sqrt{\frac{2 \cdot \Omega \cdot (R_{DL\_base} + R_{LL\_base})}{F_y \cdot B \cdot N}} = 0.19\ in$ Minimum Thickness of Plate

$t_{actual} := 0.50\ in$ Actual Thickness of Base Plate

CHECK $t_{actual} \geq t_{min} = 1$

PUNCHING SHEAR CAPACITY:

$f'_c := 3000$ Compressive Strength of Concrete (psi)

$\lambda = 1$ Normal Weight Concrete Factor

$t_{slab} := 6\ in$ Thickness of Slab

$B = 6\ in$ Base Plate Length $N = 10\ in$ Base Plate Width

$b_f = 5\ in$ Column Dimension along B $d_f = 5\ in$ Column Dimension along N

$b := b_f + \frac{B - b_f}{2} = 5.5\ in$ Equivalent Loaded Length $c := d_f + \frac{N - d_f}{2} = 7.5\ in$ Equivalent Loaded Width

$\beta := \frac{\max(b, c)}{\min(b, c)} = 1.36$ Ratio of Long Side to Short Side

$d := \frac{t_{slab}}{2} = 3\ in$ Assumed Distance to Steel Reinforcement

$\alpha_s := 20$ Assumed Position on Slab Factor

$b_0 := 2 \cdot (b + d) + 2 \cdot (c + d) = 38\ in$ Effective Perimeter around Baseplate

$\varphi := 0.75$ LRFD Shear factor

$v_1 := 4 \cdot \lambda \cdot \sqrt{f'_c}\ psi = 219.09\ psi$

$v_2 := \left(2 + \frac{4}{\beta}\right) \cdot \lambda \cdot \sqrt{f'_c}\ psi = 270.21\ psi$

$v_3 := \left(2 + \frac{\alpha_s \cdot d}{b_0}\right) \cdot \lambda \cdot \sqrt{f'_c}\ psi = 196.03\ psi$

$v_n := \min(v_1, v_2, v_3) \cdot \varphi \cdot b_0 \cdot d = 16760.31\ lbf$ Two-way Shear Strength of Slab

$P_u := 1.2 \cdot R_{DL\_base} + 1.6 \cdot R_{LL\_base} = 4861.6\ lbf$ Factored Load on Slab

CHECK $v_n \geq P_u = 1$

SUMMARY: USE SPECIFIED STRUCTURAL SUPPORT PER PLANS.

<UNIT 5> W12X14 ANALYSIS:

$L := 27.33\ ft$ — Design Length

$T_{width} := \frac{Length}{2} = 11.9\ ft$ — Tributary Width of Ceiling on Beam

LOADS:

$w_{DL} := DL_{panel} \cdot T_{width} = 59.48\ plf$ — Distributed Dead Load on Beam

$w_{LL} := LL_{panel} \cdot T_{width} = 118.95\ plf$ — Distributed Live Load on Beam

REACTIONS (ENERCALC):

$R_{DL\_1} := 1004\ lbf$ — Dead Load Reaction on Beam

$R_{LL\_1} := 1626\ lbf$ — Live Load Reaction on Beam

$\delta_{DL} := 0.945\ in - 0.584\ in = 0.361\ in$ — Deflection due to Dead Load

Material Specification

HSS 4x4x3/16"<UNIT 5> — A500 GRADE B

$H_{column} := H_{2w} = 19.58\ ft$ — Height of a Column

LOADS:

$p_{column\_DL} := R_{DL\_1} = 1004\ lbf$ — Dead Load on Column

$p_{column\_LL} := R_{LL\_1} = 1626\ lbf$ — Live Load on Column

NOTE: SEE THE ENERCALC SOFTWARE PRINTOUTS IN DESIGN AIDS FOR MEMBER ANALYSIS.

COLUMN REACTIONS (ENERCALC):

$R_{DL\_base} := 1188\ lbf$ — Dead Load at Base Plate

$R_{LL\_base} := 1626\ lbf$ — Live Load at Base Plate

BASE PLATE THICKNESS:

$b_f := 4\ in$ $\quad d_f := 4\ in$ — Column Dimensions

$\lambda := 1$ — Normal Weight Concrete Factor

$B := 5.5\ in$ $\quad N := 7\ in$ — Base Plate Dimensions

$n := (B - 0.8 \cdot b_f) \cdot 0.5 = 1.15\ in$

$m := (N - .95 \cdot d_f) \cdot 0.5 = 1.6\ in$

$n' := \frac{\sqrt{b_f \cdot d_f}}{4} = 1\ in$ — Yield-Line Theory Cantilever Distance From Column Web or Column Flange

$l := \max(m, n, \lambda \cdot n') = 1.6\ in$

$F_y := 38\ ksi$ — Base Plate Yield Strength

$\Omega := 1.67$ — ASD Factor

$t_{min} := l \cdot \sqrt{\frac{2 \cdot \Omega \cdot (R_{DL\_base} + R_{LL\_base})}{F_y \cdot B \cdot N}} = 0.13\ in$ — Minimum Thickness of Plate

$t_{actual} := 0.375\ in$ — Actual Thickness of Base Plate

CHECK $t_{actual} \geq t_{min} = 1$

WORST CASE WALL PANEL ANALYSIS

$H_{2w} = 19.58\ ft$ — Design Height

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

$T_{width\_wall} := \frac{15.5\ ft + 3.92\ ft}{2} = 9.71\ ft$ — Tributary Width of Ceiling Panel Acting on Wall

AXIAL LOADS:

$w_{design\_ceiling} = 39.17\ plf$ — Distributed Live Load

$v_{max} := \max\left(w_{design\_ceiling} \cdot T_{width\_wall}, P_{LL}\right) = 380.31\ lbf$ — Governing Live Load

$p_{design\_wall} := \frac{v_{max}}{T_{width\_panel}} + DL_{panel} \cdot T_{width\_wall} = 145.65\ plf$ — Ceiling Panel Total Axial Load

$H_{all\_axial} := 21\ ft$ — Allowable Height for Axial Load (Per LARR/Testing Report)

$P_{all\_axial} := 1582\ plf$ — Allowable Axial Load (Per LARR/Testing Report)

TRANSVERSE LOADS:

$w_{wall} := P_{internal} \cdot T_{width\_panel} = 19.58\ ft \cdot psf$ — Transverse Load on Wall

$m_{max} := \frac{w_{wall} \cdot H_{2w}^{2}}{8} = 938.47\ ft \cdot lbf$ — Maximum Moment

$H_{all\_trans} := 26.00\ ft$ — Allowable Height for Transverse Load (Per LARR/Testing Report)

$P_{all\_trans} := 5\ psf \cdot T_{width\_panel} = 19.58\ plf$ — Allowable Transverse Load (Per LARR/Testing Report)

$M_{allow} := \frac{P_{all\_trans} \cdot H_{all\_trans}^{2}}{8} = 1654.79\ ft \cdot lbf$ — Allowable Moment

$P_{comb} := \frac{p_{design\_wall}}{P_{all\_axial}} + \frac{m_{max}}{M_{allow}} = 0.66$ — Interaction of Axial and Transverse Loads

CHECK $P_{comb} \le 1 = 1$ $H_{all\_axial} \ge H_{2w} = 1$ $H_{all\_trans} \ge H_{2w} = 1$

SUMMARY: USE SPECIFIED WALL PANELS PER PLANS.

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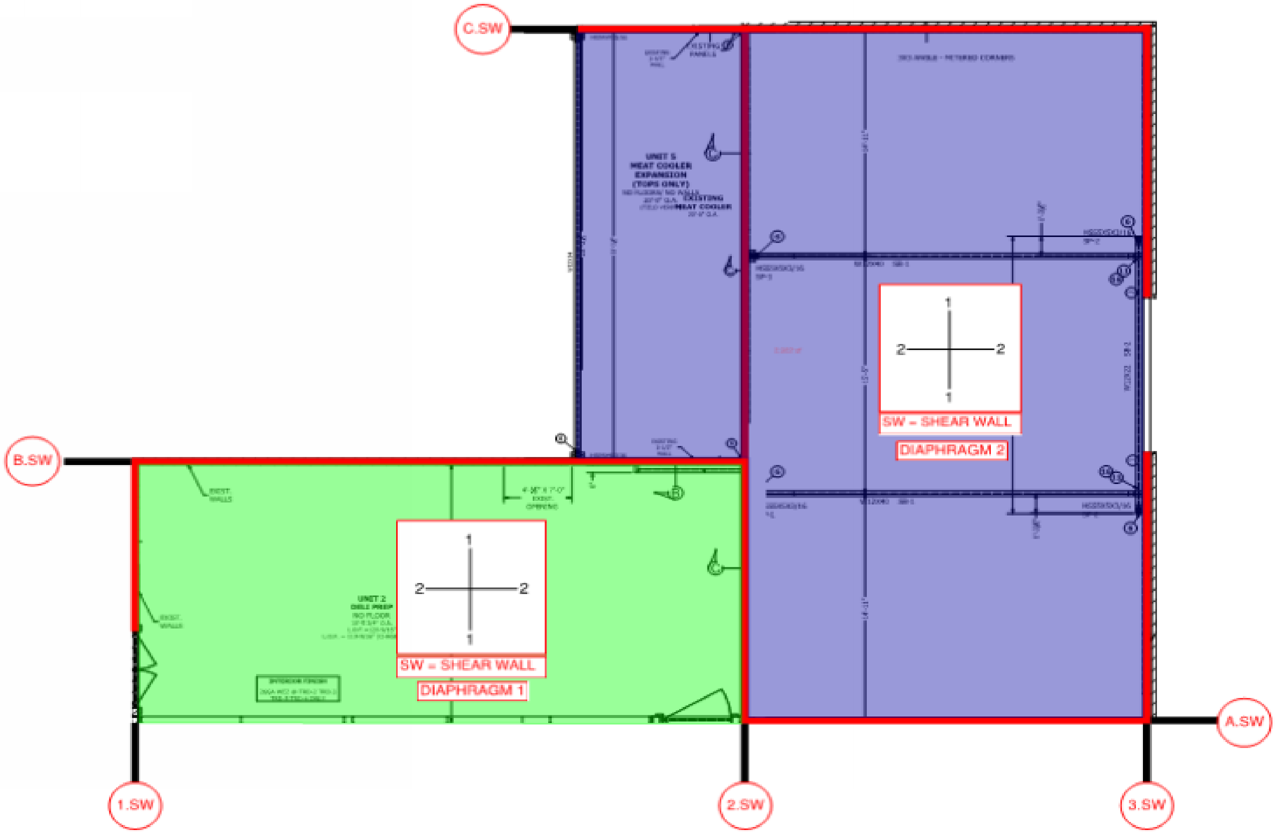
LATERAL ANALYSIS

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."

$R_p := 2.0$ Response Modification Factor

$\Omega_0 := 2.0$ Overstrength Factor

DESIGN DATA :

$I_e := 1.0$ Importance Factor

$S_s := 1.267$ Mapped Spectral Response Acceleration Parameter at Short Periods

$S_1 := 0.437$ Mapped Spectral Response Acceleration Parameter at a Period of 1 s

$S_{DS} := 1.014$ Design Spectral Response Acceleration Parameter at Short Periods

$S_{D1} := 0.543$ Design Spectral Response Acceleration Parameter at a Period of 1 s

$T_L := 6$ Long-Period Transition Period

$F_a := 1.2$ Short-Period Site Coefficient

$T_s := \frac{S_{D1}}{S_{DS}} = 0.54$ Period Define by the Ratio, Sd1/Sds

$C_t := 0.02$ Approximate Period Parameter 1 (Table 12.8-2)

$x := 0.75$ Approximate Period Parameter 2 (Table 12.8-2)

DESIGN DATA DIAPHRAGM-1 :

$h_n := \frac{H_{1f}}{ft} = 10.48$ Height of Structure

$Width := 17.29\ ft$ Unit Width

$Length := 35.96\ ft$ Unit Length

$A_{ceiling} := Length \cdot Width = 621.75\ ft^2$ Area of Ceiling

$L_{wall} := Length + 2 \cdot Width = 70.54\ ft$ Total Wall Length

LATERAL FORCE GENERATION:

$$Wt := \left(A_{ceiling} \cdot DL_{panel}\right) + \left(\frac{H_{1w}}{2} \cdot L_{wall} \cdot DL_{panel}\right) = 4882.82\ lbf$$

$T_a := C_t \cdot h_n^{\,x} = 0.116$ Approximate Fundamental Period

$S_{DS} := \frac{2}{3} \cdot F_a \cdot S_s = 1.01$ Seismic Coefficient (12.8.1.3)

$C_s := \frac{S_{DS}}{\left(\frac{R_p}{I_e}\right)} = 0.51$ Seismic Response Coefficient (Sec. 12.8.1.1)

$T_a \le 1.5\ T_s = 1$

$C_{s\_max} := \mathbf{if}\left(T_a \le T_L, 1.5\ \frac{S_{D1}}{T_a \cdot \left(\frac{R_p}{I_e}\right)}, 1.5\ \frac{S_{D1} \cdot T_L}{T_a^{\,2} \cdot \left(\frac{R_p}{I_e}\right)}\right) = 3.5$ Maximum Coefficient

$C_{s\_min} := \max\left(0.044 \cdot S_{DS} \cdot I_e, 0.01\right) = 0.045$ Minimum Coefficient

$C_{s\_min} := \mathbf{if}\left(S_1 \ge 0.6, \frac{0.5 \cdot S_1}{\left(\frac{R_p}{I_e}\right)}, C_{s\_min}\right) = 0.045$ Minimum Coefficient

$C_s := \max\left(C_s, C_{s\_min}\right) = 0.507$ Seismic Response Coefficient

$C_s := \min\left(C_s, C_{s\_max}\right) = 0.507$ Seismic Response Coefficient

$V_p := C_s \cdot Wt = 2474.61\ lbf$ — Seismic Base Shear

$V_{p\_asd} := 0.7 \cdot V_p = 1732.23\ lbf$ — ASD Seismic Base Shear

$w_{design\_1} := \dfrac{V_{p\_asd}}{Length} = 48.17\ plf$ — Distributed Design Load (1-1)

$w_{design\_2} := \dfrac{V_{p\_asd}}{Width} = 100.19\ plf$ — Distributed Design Load (2-2)

DIAPHRAGM CHECK (1-1):

$Width_1 := Width = 17.29\ ft$ — Width of Diaphragm (1-1)

$Length_1 := Length = 35.96\ ft$ — Length of Diaphragm (1-1)

$R_1 := \dfrac{Length_1}{Width_1} = 2.08$ — Aspect Ratio (1-1)

$F_{all\_1} := 178\ plf$ — Allowable Diaphragm Capacity (Per LARR/Testing Report)

CHECK: $F_{all\_1} \ge \dfrac{w_{design\_1} \cdot Length_1}{2.\ Width_1} = 1$

LAG-BOLT:

$V := \dfrac{w_{design\_1} \cdot Length_1}{2} = 866.12\ lbf$ — Max Shear at Diaphram Edge

$D := 0.375\ in$ $p := 1.5\ in$ $C_D := 1.6$ — Lag Bolt Parameters

$N_{lag} := ceil\left(\dfrac{Width_1}{24\ in}\right) = 9$ — Number of Lag-Bolts Connecting panels

$V_{all\_bolt} := C_D \cdot \dfrac{p}{8 \cdot D} \cdot 180\ lbf = 144\ lbf$ — Allowable Shear on Bolt (NDS)

$V_{all\_inplane} := N_{lag} \cdot V_{all\_bolt} = 1296\ lbf$ — Total In-Plane Shear on Lag-Bolts (Per LARR/Testing Report)

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

CHORD FORCE:

$F_{chord\_1} := \dfrac{\dfrac{w_{design\_1} \cdot Length_1^{\ 2}}{8}}{Width_1} = 450.34\ lbf$ — Chord Force 1-1

$f_{chord\_1} := \dfrac{F_{chord\_1}}{0.5 \cdot Length_1} = 25.05\ plf$ — Chord Force on 2-2 Shear Walls

DIAPHRAGM CHECK (2-2):

$Width_2 := Width = 17.29\ ft$ — Width of Diaphragm (2-2)

$Length_2 := Length = 35.96\ ft$ — Length of Diaphragm (2-2)

$R_2 := \dfrac{Width_2}{Length_2} = 0.48$ — Aspect Ratio (2-2)

$F_{all\_2} := 646\ plf$ — Allowable Diaphragm Capacity (Per LARR/Testing Report)

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

LAG-BOLT:

$V := w_{design\_2} \cdot Width_2 = 1732.23\ lbf$ — Max Shear at Diaphram Edge

$D := 0.375\ in$ $\quad p := 1.5\ in$ $\quad C_D := 1.6$ — Lag Bolt Parameters

$N_{lag} := ceil\left(\dfrac{Length_2}{24\ in}\right) = 18$ — Number of Lag-Bolts Connecting panels

$V_{all\_bolt} := C_D \cdot \dfrac{p}{8 \cdot D} \cdot 180\ lbf = 144\ lbf$ — Allowable Shear on Bolt (NDS)

$V_{all\_inplane} := N_{lag} \cdot V_{all\_bolt} = 2592\ lbf$ — Total In-Plane Shear on Lag-Bolts (Per LARR/Testing Report)

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

CHORD FORCE:

$F_{chord\_2} := \dfrac{\dfrac{w_{design\_2} \cdot {Width_2}^2}{2}}{Length_2} = 416.44\ lbf$ — Chord Force 2-2

$f_{chord\_2} := \dfrac{F_{chord\_2}}{0.5 \cdot Width_2} = 48.17\ plf$ — Chord Force on 1-1 Shear Walls

SHEAR LOAD CALCULATIONS:

<u>CHORD FORCE TRANSFER TO SHEAR WALLS:</u>

$L_{chord\_1\_1} := 0.5 \cdot Width_2 = 8.65\ ft$ — Length of (1-1) Wall Taking "f_chord_2"

$L_{chord\_2\_2} := 0.5 \cdot Length_1 = 17.98\ ft$ — Length of (2-2) Wall Taking "f_chord_1"

$f_{chord\_2} = 48.17\ plf$ — Chord Force on 1-1 Shear Walls

$f_{chord\_1} = 25.05\ plf$ — Chord Force on 2-2 Shear Walls

$f_{chord} := \max\left(f_{chord\_1}, f_{chord\_2}\right) = 48.17\ plf$ — Maximum Chord Force on Shear Walls

SHEAR WALL CALCULATIONS - DIAPHRAGM 1:

$L_{1\_1} := 11.33\ ft$ Length of Wall 1

$T_{width\_1} := \frac{Length}{2} = 17.98\ ft$ Tributary Width

$f_{1\_1} := \frac{w_{design\_1} \cdot T_{width\_1}}{L_{1\_1}} = 76.44\ plf$ In-Plane Force on Gridline 1

$L_{2\_1} := Width = 17.29\ ft$ Length of Wall 2

$T_{width\_2} := \frac{Length}{2} = 17.98\ ft$ Tributary Width

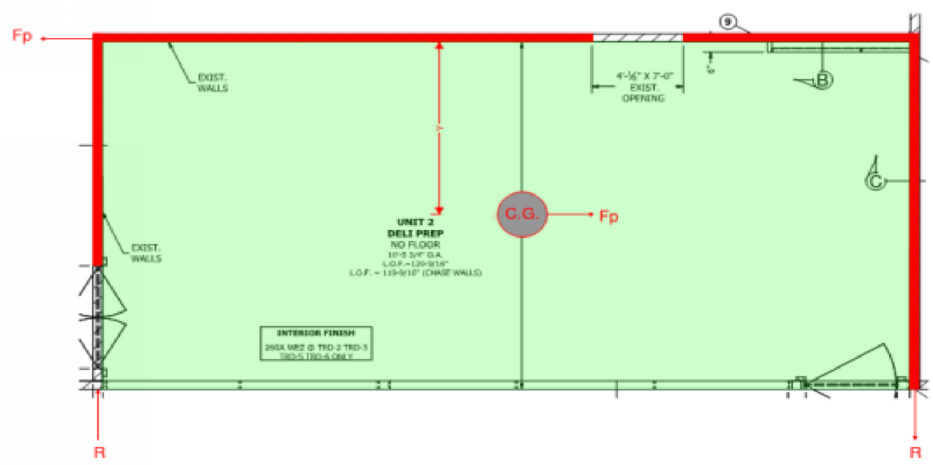
$f_{2\_1} := \frac{w_{design\_1} \cdot T_{width\_2}}{L_{2\_1}} = 50.09\ plf$ In-Plane Force on Gridline 2

NOTE: GRIDLINE 2 OF DIAPHRAGM *1<UNIT2> SHARES A WALL WITH* DIAPHRAGM *2<UNIT1 & UNIT5>. THE RESULTING FORCES ON THIS WALL WILL BE ANALYZED IN* DIAPHRAGM *2<UNIT1 & UNIT5> SHEAR WALL CALCULATIONS.*

$L_{B\_1} := Length - 4.01\ ft = 31.95\ ft$ Length of Wall B

$T_{width\_B} := Width = 17.29\ ft$ Tributary Width

$f_{B\_1} := \frac{w_{design\_2} \cdot T_{width\_B}}{L_{B\_1}} = 54.22\ plf$ In-Plane Force on Gridline B



RESULTANT FORCES FROM ECCENTRICITY (DIAPHRAGM 1):

$Y := \frac{Width}{2} = 8.65\ ft$ Moment Arm

$F_{p\_design2\_2} := w_{design\_2} \cdot Width = 1732.23\ lbf$

$M_F := F_{p\_design2\_2} \cdot Y = 14975.13\ ft \cdot lbf$ Resultant Moment from Eccentricity

$R := \frac{M_F}{Length} = 416.44\ lbf$ Reaction Forces to Resist Imposed Moment

$w_{1\_1} := \frac{R}{L_{1\_1}} = 36.76\ plf$ Inplane Force Resisted on Wall Line 1

$w_{2\_1} := \frac{R}{L_{2\_1}} = 24.09\ plf$ Inplane Force Resisted on Wall Line 2

$f_{1\_1} := \max\left(f_{1\_1}, w_{1\_1}\right) = 76.44\ plf$ Inplane Force in Wall Line 1 due to Diaphragm 1

$f_{2\_1} := \max\left(f_{2\_1}, w_{2\_1}\right) = 50.09\ plf$ Inplane Force in Wall Line 2 due to Diaphragm 1

$R := \frac{H_{1w}}{\min\left(L_{1\_1}, L_{2\_1}, L_{B\_1}, 10.42\ ft, L_{chord\_1\_1}, L_{chord\_2\_2}\right)} = 1.16$ Diaphragm 1 Shape Factor Ratio

$F_{all\_inplane} := 305\ plf$ Allowable In-Plane Shear

CHECK: $F_{all\_inplane} \geq \max\left(f_{1\_1}, f_{2\_1}, f_{B\_1}, f_{chord}\right) = 1$

<u>SUMMARY: THE ALLOWABLE FORCES ARE GREATER THAN THE APPLIED FORCES. THEREFORE, USE WALL PANELS PER PLAN.</u>

DESIGN DATA DIAPHRAGM-2 :

$h_n := \frac{H_{2f}}{ft} = 20$ Height of Structure

$Width := 45.25\ ft$ Unit Width

$Length := 33.33\ ft$ Unit Length

$A_{ceiling} := Length \cdot Width = 1508.18\ ft^2$ Area of Ceiling

$L_{wall} := 2 \cdot Length + 2 \cdot Width = 157.16\ ft$ Total Wall Length

$Wt_{steel\_beam} := 2 \cdot 22.83\ ft \cdot 40\ plf + 18.00\ ft \cdot 22\ plf + 27.33\ ft \cdot 14\ plf = 2605.02\ lbf$ Total Weight of Steel Beam

$Wt_{steel\_column} := 2 \cdot \frac{H_{2w}}{2} \cdot 9.42\ plf + 4 \cdot \frac{H_{2w}}{2} \cdot 11.97\ plf = 653.19\ lbf$ Total Weight of Steel Column

$Wt_{steel} := Wt_{steel\_beam} + Wt_{steel\_column} = 3258.21\ lbf$ Total Weight of Steel

LATERAL FORCE GENERATION:

$Wt := \left(A_{ceiling} \cdot DL_{panel}\right) + \left(\frac{H_{2w}}{2} \cdot L_{wall} \cdot DL_{panel}\right) + Wt_{steel} = 18492.1\ lbf$

$T_a := C_t \cdot h_n^{\ x} = 0.189$ Approximate Fundamental Period

NOTE: IF THE STRUCTURE IS 5 STORIES OR LESS ABOVE THE BASE, Ss MAY BE RECALCULATED AS:

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

$S_{DS} := min\left(S_{DS\_max}, S_{DS}\right) = 1$ Design Spectral Response for Short Period, (g)

$C_s := \frac{S_{DS}}{\left(\frac{R_p}{I_e}\right)} = 0.5$ Seismic Response Coefficient (Sec. 12.8.1.1)

$T_a \le 1.5\ T_s = 1$

$C_{s\_max} := \text{if}\left(T_a \le T_L, 1.5\ \frac{S_{D1}}{T_a \cdot \left(\frac{R_p}{I_e}\right)}, 1.5\ \frac{S_{D1} \cdot T_L}{T_a^{\ 2} \cdot \left(\frac{R_p}{I_e}\right)}\right) = 2.15$ Maximum Coefficient

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

$C_{s\_min} := \text{if}\left(S_1 \ge 0.6, \frac{0.5 \cdot S_1}{\left(\frac{R_p}{I_e}\right)}, C_{s\_min}\right) = 0.044$ Minimum Coefficient

$C_s := \max\left(C_s, C_{s\_min}\right) = 0.5$ Seismic Response Coefficient

$C_s := min\left(C_s, C_{s\_max}\right) = 0.5$ Seismic Response Coefficient

$V_p := C_s \cdot Wt = 9246.05\ lbf$ Seismic Base Shear

$V_{p\_asd} := 0.7 \cdot V_p = 6472.24\ lbf$ ASD Seismic Base Shear

$w_{design\_1} := \dfrac{V_{p\_asd}}{Length} = 194.19\ plf$ Distributed Design Load (1-1)

$w_{design\_2} := \dfrac{V_{p\_asd}}{Width} = 143.03\ plf$ Distributed Design Load (2-2)

DIAPHRAGM CHECK (1-1):

$Width_1 := Width = 45.25\ ft$ Width of Diaphragm (1-1)

$Length_1 := Length = 33.33\ ft$ Length of Diaphragm (1-1)

$R_1 := \dfrac{Length_1}{Width_1} = 0.74$ Aspect Ratio (1-1)

$F_{all\_1} := 496\ plf$ Allowable Diaphragm Capacity (Per LARR/Testing Report)

CHECK: $F_{all\_1} \ge \dfrac{w_{design\_1} \cdot Length_1}{2.\ Width_1} = 1$

LAG-BOLT:

$V := \dfrac{w_{design\_1} \cdot Length_1}{2} = 3236.12\ lbf$ Max Shear at Diaphram Edge

$D := 0.375\ in$ $p := 1.5\ in$ $C_D := 1.6$ Lag Bolt Parameters

$N_{lag} := ceil\left(\dfrac{Width_1}{24\ in}\right) = 23$ Number of Lag-Bolts Connecting panels

$V_{all\_bolt} := C_D \cdot \dfrac{p}{8 \cdot D} \cdot 180\ lbf = 144\ lbf$ Allowable Shear on Bolt (NDS)

$V_{all\_inplane} := N_{lag} \cdot V_{all\_bolt} = 3312\ lbf$ Total In-Plane Shear on Lag-Bolts (Per LARR/Testing Report)

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

CHORD FORCE:

$F_{chord\_1} := \dfrac{\dfrac{w_{design\_1} \cdot Length_1^{\,2}}{8}}{Width_1} = 595.91\ lbf$ Chord Force 1-1

$f_{chord\_1} := \dfrac{F_{chord\_1}}{0.5 \cdot Length_1} = 35.76\ plf$ Chord Force on 2-2 Shear Walls

DIAPHRAGM CHECK (2-2):

$Width_2 := Width = 45.25\ ft$ — Width of Diaphragm (2-2)

$Length_2 := 23.83\ ft$ — Length of Diaphragm (2-2)

$R_2 := \dfrac{Width_2}{Length_2} = 1.90$ — Aspect Ratio (2-2)

$F_{all\_2} := 192\ plf$ — Allowable Diaphragm Capacity (Per LARR/Testing Report)

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

LAG-BOLT:

$V := \dfrac{w_{design\_2} \cdot Width_2}{2} = 3236.12\ lbf$ — Max Shear at Diaphram Edge

$D := 0.375\ in$ $\quad p := 1.5\ in$ $\quad C_D := 1.6$ — Lag Bolt Parameters

$N_{lag} := ceil\left(\dfrac{Width_1}{24\ in}\right) = 23$ — Number of Lag-Bolts Connecting panels

$V_{all\_bolt} := C_D \cdot \dfrac{p}{8 \cdot D} \cdot 180\ lbf = 144\ lbf$ — Allowable Shear on Bolt (NDS)

$V_{all\_inplane} := N_{lag} \cdot V_{all\_bolt} = 3312\ lbf$ — Total In-Plane Shear on Lag-Bolts (Per LARR/Testing Report)

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

CHORD FORCE:

$F_{chord\_2} := \dfrac{\dfrac{w_{design\_2} \cdot Width_2^{\ 2}}{8}}{Length_2} = 1536.24\ lbf$ — Chord Force 2-2

$f_{chord\_2} := \dfrac{F_{chord\_2}}{0.5 \cdot Width_2} = 67.9\ plf$ — Chord Force on 1-1 Shear Walls

SHEAR LOAD CALCULATIONS:

<u>CHORD FORCE TRANSFER TO SHEAR WALLS:</u>

$L_{chord\_1\_1} := 0.5 \cdot Width_2 = 22.63\ ft$ — Length of (1-1) Wall Taking "f_chord_2"

$L_{chord\_2\_2} := 0.5 \cdot Length_1 = 16.67\ ft$ — Length of (2-2) Wall Taking "f_chord_1"

$f_{chord\_2} = 67.9\ plf$ — Chord Force on 1-1 Shear Walls

$f_{chord\_1} = 35.76\ plf$ — Chord Force on 2-2 Shear Walls

$f_{chord} := \max\left(f_{chord\_1}, f_{chord\_2}\right) = 67.9\ plf$ — Maximum Chord Force on Shear Walls

SHEAR WALL CALCULATIONS - DIAPHRAGM 2:

$L_{2\_2} := Width = 45.25\ ft$ Length of Wall 2

$T_{width\_2} := \dfrac{23.83\ ft}{2} + 9.5\ ft = 21.42\ ft$ Tributary Width

$f_{2\_2} := \dfrac{w_{design\_1} \cdot T_{width\_2}}{L_{2\_2}} = 91.9\ plf$ In-Plane Force on Gridline 2

$f_{2\_total} := f_{2\_2} + f_{2\_1} = 141.99\ plf$ Total In-Plane Force on Gridline 2

$L_{3\_2} := Width - 10\ ft = 35.25\ ft$ Length of Wall 3

$T_{width\_3} := \dfrac{23.83\ ft}{2} = 11.92\ ft$ Tributary Width

$f_{3\_2} := \dfrac{w_{design\_1} \cdot T_{width\_3}}{L_{3\_2}} = 65.64\ plf$ In-Plane Force on Gridline 3

$L_{A\_2} := 23.83\ ft$ Length of Wall A

$T_{width\_A} := \dfrac{Width}{2} = 22.63\ ft$ Tributary Width

$f_{A\_2} := \dfrac{w_{design\_2} \cdot T_{width\_A}}{L_{A\_2}} = 135.8\ plf$ In-Plane Force on Gridline A

$L_{C\_2} := Length = 33.33\ ft$ Length of Wall C

$T_{width\_C} := \dfrac{Width}{2} = 22.63\ ft$ Tributary Width

$f_{C\_2} := \dfrac{w_{design\_2} \cdot T_{width\_C}}{L_{C\_2}} = 97.09\ plf$ In-Plane Force on Gridline C

$R := \dfrac{H_{2w}}{min\left(L_{2\_2}, L_{3\_2}, L_{A\_2}, 17.63\ ft, L_{chord\_1\_1}, L_{chord\_2\_2}\right)} = 1.17$ Diaphragm 1 Shape Factor Ratio

$F_{all\_inplane} := 303\ plf$ Allowable In-Plane Shear

CHECK: $F_{all\_inplane} \geq max\left(f_{2\_total}, f_{3\_2}, f_{A\_2}, f_{C\_2}, f_{chord}\right) = 1$

SUMMARY: THE ALLOWABLE FORCES ARE GREATER THAN THE APPLIED FORCES. THEREFORE, USE WALL PANELS PER PLAN.

CEILING PANEL TO WALL PANEL CONNECTION (DETAIL 1)

LOADS:

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

$p_{trans} := P_{internal} \cdot \frac{H_{2w}}{2} = 48.95\ plf$ Transverse Shear Force on Wall-Ceiling Connection

$f_{inplane} := \max\left(f_{1\_1}, f_{2\_total}, f_{A\_2}, f_{B\_1}, f_{C\_2}, f_{chord}\right) = 141.99\ plf$ In-Plane Shear Force on Wall-Ceiling Connection

$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 141.99\ plf$ Governing Shear Force on Wall-Ceiling Connection

3/8" LAG BOLT:

$D := 0.375\ in$ Lag Bolt Diameter

$S_{bolt} := 24\ in$ Spacing of Lag Bolt

$C_D := 1.6$ Load Duration Factor (NDS)

$p := 1.5\ in$ Penetration Depth

$V_{all\_bolt\_par} := \frac{C_D \cdot \frac{p}{8 \cdot D} \cdot 180\ lbf}{S_{bolt}} = 72\ plf$ Allowable Shear of Lag Bolt Parallel to Grain (NDS)

$V_{all\_bolt\_perp} := \frac{C_D \cdot \frac{p}{8 \cdot D} \cdot 110\ lbf}{S_{bolt}} = 44\ plf$ Allowable Shear of Lag Bolt Perpendicular to Grain (NDS)

#14 TEK SCREW:

$S_{screw} := 6\ in$ Spacing of Screws

$V_{all\_screw} := \frac{76\ lbf}{S_{screw}} = 152\ plf$ Allowable Shear on Screw (ESR-1976)

$T_{all\_screw} := \frac{57\ lbf}{S_{screw}} = 114\ plf$ Allowable Tension on Screw (ESR-1976)

CHECK $\frac{f_{inplane}}{\left(V_{all\_bolt\_par} + V_{all\_screw}\right)} + \frac{p_{trans}}{\left(V_{all\_bolt\_perp} + T_{all\_screw}\right)} \le 1 = 1$

SUMMARY: THE IMPOSED FORCE ON THE CONNECTION IS LESS THAN THE ALLOWABLE FORCE. THEREFORE, THE CEILING TO WALL PANEL CONNECTION IS ACCEPTABLE .

CEILING PANEL TO WALL PANEL CONNECTION AT PARTITION WALL (DETAIL 13/9)

$H_{1w} = 10.06\ ft$ — Design Height Of Diaphragm 2

$T_{width\_lower} := 3.92\ ft$ — Tributary Width of Ceiling on Lower Connection

LOADS:

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

$LC_3 = 15\ psf$ — Gravity Load on Lower Wall- Ceiling Connection

$p_{grav} := LC_3 \cdot T_{width\_lower} = 58.8\ plf$ — Force on Lower Wall-Ceiling Connection

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

$f_{inplane} := \max\left(f_{2\_1}, f_{chord}\right) = 67.9\ plf$ — In-Plane Shear Force on Wall-Ceiling Connection

$p_{trans} := P_{internal} \cdot \dfrac{H_{1w}}{2} = 25.15\ plf$ — Transverse Shear Force on Lower Wall-Ceiling Connection

$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 67.9\ plf$ — Governing Shear Force on Lower Wall-Ceiling Connection

#14 TEK SCREW:

$S_{screw} := 6\ in$ — Spacing of Screw

$n_{screw} := 1$ — Number of Screws into Wall Panel

$V_{all\_screw} := \dfrac{n_{screw} \cdot 76\ lbf}{S_{screw}} = 152\ plf$ — Allowable Shear of Connection (ESR 1976)

$T_{all\_screw} := \dfrac{n_{screw} \cdot 57\ lbf}{S_{screw}} = 114\ plf$ — Allowable Tension of Connection (ESR 1976)

$R_{combined} := \dfrac{p_{trans}}{T_{all\_screw}} + \dfrac{\sqrt{f_{max}^{\ 2} + p_{grav}^{\ 2}}}{V_{all\_screw}} = 0.81$ — Combined Stress Ratio for Connection

CHECK: $R_{combined} \le 1.0 = 1$ $\quad f_{inplane} \le V_{all\_screw} = 1$

SUMMARY: THE IMPOSED FORCE ON THE CONNECTION IS LESS THAN THE ALLOWABLE FORCE. THEREFORE, THE WALL-CEILING CONNECTION IS ACCEPTABLE FOR RESISTING IMPOSED LOADS.

WALL TO FLOOR CONNECTION (DETAIL 2)

$H_{2w} = 19.58\ ft$ — Design Height

LOADS:

$p_{trans} := P_{internal} \cdot \frac{H_{2w}}{2} = 48.95\ plf$ — Transverse Shear Force on Floor-Wall Connection

$f_{inplane} := \max\left(f_{1\_1}, f_{A\_2}, f_{B\_1}, f_{C\_2}, f_{2\_total}, f_{chord}\right) = 141.99\ plf$ — In-Plane Shear Force on Floor-Wall Connection

$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 141.99\ plf$ — Governing Shear Force on Floor-Wall Connection

#14 TEK SCREW:

$L_{screw} := 1.5\ in$ — Length of Screws

$S_{screw} := 9\ in$ — Spacing of Screws

$C_D := 1.6$ — Load Duration Factor

$V_{all\_screw} := \frac{C_D \cdot 104.7\ lbf}{S_{screw}} = 223.36\ plf$ — Allowable Shear of #14 TEK (Dowel Bearing Strength - NDS)

$T_{all\_screw} := \frac{C_D \cdot 0.95\ in \cdot 121\ \frac{lbf}{in}}{S_{screw}} = 245.23\ plf$ — Allowable Tension of #14 TEK (NDS)

3/8" Simpson Titen HD:

$S_{anchor} := 24\ in$ — Spacing of Anchor Through Angle

$\Omega_0 := 2.0$ — Overstrength Factor

$v_{anchor} := \frac{\Omega_0 \cdot f_{max} \cdot S_{anchor}}{0.7} = 811.39\ lbf$ — LRFD Shear Force on Anchor

$V_{all\_anchor} := 1650\ lbf$ — Allowable Shear on Anchor

NOTE: SEE SIMPSON REPORT FOR THE ANCHOR ANALYSIS.

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

SUMMARY: THE MAXIMUM FORCE ON THE CONNECTION IS LESS THAN THE ALLOWABLE FORCE; THEREFORE, THE WALL-FLOOR CONNECTION IS ACCEPTABLE FOR RESISTING IMPOSED LOADS.

OVERTURNING CALCULATIONS FOR CONTINUOUS ANGLE (WORST CASE)

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

$T_{width\_ceiling} := \dfrac{2 \cdot 3.92\ ft}{2} = 3.92\ ft$ — Tributary Width of Roof

$H_{2w} = 19.58\ ft$ — Height of Wall Panel

ASD LOADS:

$f := f_{2\_total} = 141.99\ plf$ — In-Plane Force on Wall In ASD

$L := L_{2\_2} = 45.25\ ft$ — Length of Wall

$S_{DS} := 1.014$ — Seismic Design Value

$Wt_{wall} := \left(0.6 - 0.14 \cdot S_{DS}\right) \cdot DL_{panel} \cdot H_{2w} \cdot L = 2029.11\ lbf$ — Weight of Wall

$Wt_{ceiling} := \left(0.6 - 0.14 \cdot S_{DS}\right) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 406.24\ lbf$ — Weight of Ceiling

$w_R := \dfrac{Wt_{wall} + Wt_{ceiling}}{L} = 53.82\ plf$ — Weight Resisting Overturning

$M_{wall} := f \cdot L \cdot H_{2w} - w_R \cdot \dfrac{L^2}{2} = 70706.42\ lbf \cdot ft$ — Overturning Moment Acting on Wall

$w := \dfrac{3 \cdot M_{wall}}{L^2} = 103.6\ plf$ — Maximum Value of Overturning Force at End

$\left(M_{wall} := \dfrac{1}{2} \cdot w \cdot L \cdot \dfrac{2\ L}{3} = 70706.42\ lbf \cdot ft \right)$

#14 TEK SCREWS:

$S_{screw} := 9\ in$ — Spacing of Screw

$n_{screw} := 1$ — Number of screw within the Spacing Considered

$V_{des\_screw} := n_{screw} \cdot C_D \cdot 104.7\ lbf = 167.52\ lbf$ — Allowable Shear of #14 TEK (Dowel Bearing Strength - NDS)

$T_{des\_screw} := n_{screw} \cdot C_D \cdot 0.95\ in \cdot 121\ \dfrac{lbf}{in} = 183.92\ lbf$ — Allowable Tension of #14 TEK (NDS)

$V_{screw\_inplane} := f \cdot S_{screw} = 106.5\ lbf$ — Maximum Shear Force on End Screw due to Inplane Shear

$V_{screw\_uplift} := \dfrac{\dfrac{w \cdot \left(L - S_{screw}\right)}{L} + w}{2} \cdot S_{screw} = 77.05\ lbf$ — Maximum Shear Force on End Screw due to Uplift

$V_{screw} := \sqrt{V_{screw\_inplane}^{\ 2} + V_{screw\_uplift}^{\ 2}} = 131.45\ lbf$ — Maximum Resultant Shear Force on End Screw

$T_{screw} := p_{trans} \cdot S_{screw} = 36.71\ lbf$ — Maximum Tension Force on End Screw

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

LRFD LOADS:

$f := \frac{f}{0.7} = 202.85\ plf$ In-Plane Force on Wall In LRFD

$L := L = 45.25\ ft$ Length of Wall

$Wt_{wall} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot H_{2w} \cdot L = 3088.58\ lbf$ Weight of Wall

$Wt_{ceiling} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 618.35\ lbf$ Weight of Ceiling

$w_R := \frac{Wt_{wall} + Wt_{ceiling}}{L} = 81.92\ plf$ Weight Resisting Overturning

$M_{wall} := f \cdot L \cdot H_{2w} - w_R \cdot \frac{L^2}{2} = 95853.71\ lbf \cdot ft$ Overturning Moment Acting on Wall

$w := \frac{3 \cdot M_{wall}}{L^2} = 140.44\ plf$ Maximum Value of Overturning Force at End

$\left(M_{wall} := \frac{1}{2} \cdot w \cdot L \cdot \frac{2\ L}{3} = 95853.71\ lbf \cdot ft \right)$

3/8" SIMPSON TITEN HD:

$S_{anchor} := 24\ in$ Spacing of Anchor

$v_{anchor} := \Omega \cdot f \cdot S_{anchor} = 677.51\ lbf$ Ultimate Governing Shear on Anchor

$T_{anchor} := \Omega \cdot \frac{\frac{w \cdot (L - S_{anchor})}{L} + w}{2} \cdot S_{anchor} = 458.71\ lbf$ Maximum Tension Force on End Anchor

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

SUMMARY: USE SPECIFIED CONNECTION PER PLANS.

STRUCTURAL CALCULATIONS CBX (UNIT 3, UNIT 6 & UNIT 7)

JURISDICTION INFORMATION

JURISDICTION: PUYALLUP, WASHINGTON
STRUCTURAL CODE: 2018 INTERNATIONAL BUILDING CODE

PASS = 1.0 FAILURE = 0

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 7-10/16)

$LC_3 := DL_{panel} + LL_{panel} = 15\ psf$ 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 <UNIT 3>

$Width := 8.92\ ft$ Unit Width

$Length := 26.00\ ft$ Unit Length

$H := 10.48\ ft$ Unit Height

$H_w := 10.01\ ft$ 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

$L := 8.92\ ft$ Ceiling Panel Span

$L_{all} := 9.08\ ft$ Allowable Span (Per LARR/Testing Report)

$T_{width\_panel} := 47\ in$ Tributary Width of Panel

LOADS:

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

$w_{design\_ceiling} := LL_{panel} \cdot T_{width\_panel} = 39.17\ plf$ Distributed Live Load

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

$m_{max} := \max\left(\frac{w_{design\_ceiling} \cdot L^2}{8}, \frac{P_{LL} \cdot L}{4}\right) = 669\ ft \cdot lbf$ Maximum Moment

$w_{all} := 90\ psf \cdot T_{width\_panel} = 352.5\ plf$ Allowable Panel Load (Per LARR/Testing Report)

$M_{allow} := \frac{w_{all} \cdot L_{all}^{\ 2}}{8} = 3632.79\ ft \cdot lbf$ Allowable Moment

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

SUMMARY: USE SPECIFIED CEILING PANELS PER PLANS.

WALL PANEL ANALYSIS

$H_w = 10.01\ ft$ — Design Height

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

$T_{width\_wall} := \dfrac{Width + 3.92\ ft}{2} = 6.42\ ft$ — Tributary Width of Ceiling Panel Acting on Wall

AXIAL LOADS:

$w_{design\_ceiling} = 39.17\ plf$ — Distributed Live Load

$v_{max} := \max\left(w_{design\_ceiling} \cdot T_{width\_wall}, P_{LL}\right) = 300\ lbf$ — Governing Live Load

$p_{design\_wall} := \dfrac{v_{max}}{T_{width\_panel}} + DL_{panel} \cdot T_{width\_wall} = 108.7\ plf$ — Ceiling Panel Total Axial Load

$H_{all\_axial} := 16\ ft$ — Allowable Height for Axial Load (Per LARR/Testing Report)

$P_{all\_axial} := 2779\ plf$ — Allowable Axial Load (Per LARR/Testing Report)

TRANSVERSE LOADS:

$w_{wall} := P_{internal} \cdot T_{width\_panel} = 19.58\ ft \cdot psf$ — Transverse Load on Wall

$m_{max} := \dfrac{w_{wall} \cdot H_w^{\ 2}}{8} = 245.28\ ft \cdot lbf$ — Maximum Moment

$H_{all\_trans} := 12.5\ ft$ — Allowable Height for Transverse Load (Per LARR/Testing Report)

$P_{all\_trans} := 46.3\ psf \cdot T_{width\_panel} = 181.34\ plf$ — Allowable Transverse Load (Per LARR/Testing Report)

$M_{allow} := \dfrac{P_{all\_trans} \cdot H_{all\_trans}^{\ 2}}{8} = 3541.83\ ft \cdot lbf$ — Allowable Moment

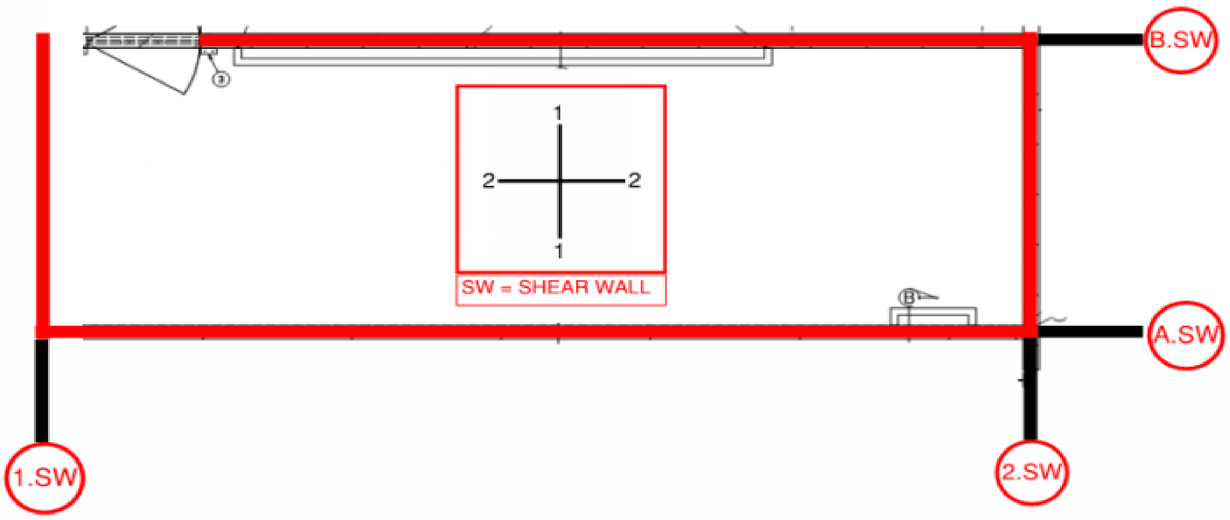
$P_{comb} := \dfrac{p_{design\_wall}}{P_{all\_axial}} + \dfrac{m_{max}}{M_{allow}} = 0.11$ — Interaction of Axial and Transverse Loads

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

SUMMARY: USE SPECIFIED WALL PANELS PER PLANS.

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."

$R_p := 2.0$ Response Modification Factor

$\Omega_0 := 2.0$ Overstrength Factor

DESIGN DATA:

$I_e := 1.0$ Importance Factor

$S_s := 1.267$ Mapped Spectral Response Acceleration Parameter at Short Periods

$S_1 := 0.437$ Mapped Spectral Response Acceleration Parameter at a Period of 1 s

$S_{DS} := 1.014$ Design Spectral Response Acceleration Parameter at Short Periods

$S_{D1} := 0.543$ Design Spectral Response Acceleration Parameter at a Period of 1 s

$T_L := 6$ Long-Period Transition Period

$F_a := 1.2$ Short-Period Site Coefficient

$T_s := \dfrac{S_{D1}}{S_{DS}} = 0.54$ Period Define by the Ratio, Sd1/Sds

$h_n := \frac{H}{ft} = 10.48$ Height of Structure

$C_t := 0.02$ Approximate Period Parameter 1 (Table 12.8-2)

$x := 0.75$ Approximate Period Parameter 2 (Table 12.8-2)

DESIGN CRITERIA:

$A_{ceiling} := Length \cdot Width = 231.92\ ft^2$ Total Area of Ceiling

$L_{wall} := 2 \cdot Length + 2 \cdot Width = 69.84\ ft$ Total Length of Walls

LATERAL FORCE GENERATION:

$Wt := \left(A_{ceiling} \cdot DL_{panel}\right) + \left(\frac{H_w}{2} \cdot L_{wall} \cdot DL_{panel}\right) = 2907.35\ lbf$ Effective Seismic Weight

$T_a := C_t \cdot h_n^{\ x} = 0.116$ Approximate Fundamental Period

NOTE: IF THE STRUCTURE IS 5 STORIES OR LESS ABOVE THE BASE, Ss MAY BE RECALCULATED AS:

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

$S_{DS} := \min\left(S_{DS\_max}, S_{DS}\right) = 1$ Design Spectral Response for Short Period, (g)

$C_s := \frac{S_{DS}}{\left(\frac{R_p}{I_e}\right)} = 0.5$ Seismic Response Coefficient (Sec. 12.8.1.1)

$T_a \le 1.5\ T_s = 1$

$C_{s\_max} := \text{if}\left(T_a \le T_L, 1.5\frac{S_{D1}}{T_a \cdot \left(\frac{R_p}{I_e}\right)}, 1.5\frac{S_{D1} \cdot T_L}{T_a^{\ 2} \cdot \left(\frac{R_p}{I_e}\right)}\right) = 3.5$ Maximum Coefficient

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

$C_{s\_min} := \text{if}\left(S_1 \ge 0.6, \frac{0.5 \cdot S_1}{\left(\frac{R_p}{I_e}\right)}, C_{s\_min}\right) = 0.044$ Minimum Coefficient

$C_s := \max\left(C_s, C_{s\_min}\right) = 0.5$ Seismic Response Coefficient

$C_s := \min\left(C_s, C_{s\_max}\right) = 0.5$ Seismic Response Coefficient

$V_p := C_s \cdot Wt = 1453.67\ lbf$ Seismic Base Shear

$V_{p\_asd} := 0.7 \cdot V_p = 1017.57\ lbf$ ASD Seismic Base Shear

$w_{design\_1} := \frac{V_{p\_asd}}{Length} = 39.14\ plf$ Distributed Design Load (1-1)

$w_{design\_2} := \frac{V_{p\_asd}}{Width} = 114.08\ plf$ Distributed Design Load (2-2)

DIAPHRAGM CHECK (1-1):

$Width_1 := Width = 8.92\ ft$ — Width of Diaphragm (1-1)

$Length_1 := Length = 26\ ft$ — Length of Diaphragm (1-1)

$R_1 := \frac{Length_1}{Width_1} = 2.91$ — Aspect Ratio (1-1)

$F_{all\_1} := 163\ plf$ — Allowable Diaphragm Capacity (Per LARR/Testing Report)

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

#14 TEK SCREW:

$V := \frac{w_{design\_1} \cdot Length_1}{2} = 508.79\ lbf$ — Max Shear at Diaphram Edge

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

$V_{all\_screw} := 76\ lbf$ — Allowable Shear on Screw (ESR-1976)

$N_{screw} := ceil\left(\frac{Width_1}{S_{screw}}\right) = 9$ — Number of Lag Bolts Connecting panels

$V_{all\_inplane} := N_{screw} \cdot V_{all\_screw} = 684\ lbf$ — Total In-Plane Shear on Screw (Per LARR/Testing Report)

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

CHORD FORCE:

$F_{chord\_1} := \frac{\frac{w_{design\_1} \cdot Length_1^2}{8}}{Width_1} = 370.75\ lbf$ — Chord Force 1-1

$f_{chord\_1} := \frac{F_{chord\_1}}{0.5 \cdot Length_1} = 28.52\ plf$ — Chord Force on 2-2 Shear Walls

DIAPHRAGM CHECK (2-2):

$Width_2 := Width = 8.92\ ft$ — Width of Diaphragm (2-2)

$Length_2 := Length = 26\ ft$ — Length of Diaphragm (2-2)

$R_2 := \frac{Width_2}{Length_2} = 0.34$ — Aspect Ratio (2-2)

$F_{all\_2} := 646\ plf$ — Allowable Diaphragm Capacity (Per LARR/Testing Report)

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

#14 TEK SCREW:

$V := \frac{w_{design\_2} \cdot Width_2}{2} = 508.79\ lbf$ — Max Shear at Diaphram Edge

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

$V_{all\_screw} := 76\ lbf$ — Allowable Shear on Screw (ESR-1976)

$N_{screw} := \text{ceil}\left(\frac{Length_2}{S_{screw}}\right) = 27$ — Number of Tek Screws Connecting panels

$V_{all\_inplane} := N_{screw} \cdot V_{all\_screw} = 2052\ lbf$ — Total In-Plane Shear on Screw (Per LARR/Testing Report)

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

CHORD FORCE:

$F_{chord\_2} := \frac{\frac{w_{design\_2} \cdot Width_2^{\,2}}{8}}{Length_2} = 43.64\ lbf$ — Chord Force 2-2

$f_{chord\_2} := \frac{F_{chord\_2}}{0.5 \cdot Width_2} = 9.78\ plf$ — Chord Force on 1-1 Shear Walls

SHEAR LOAD CALCULATIONS:
<u>CHORD FORCE TRANSFER TO SHEAR WALLS:</u>

$L_{chord\_1\_1} := 0.5 \cdot Width_2 = 4.46\ ft$ — Length of (1-1) Wall Taking "f_chord_2"

$L_{chord\_2\_2} := 0.5 \cdot Length_1 = 13\ ft$ — Length of (2-2) Wall Taking "f_chord_1"

$f_{chord\_2} = 9.78\ plf$ — Chord Force on 1-1 Shear Walls

$f_{chord\_1} = 28.52\ plf$ — Chord Force on 2-2 Shear Walls

$f_{chord} := \max\left(f_{chord\_1}, f_{chord\_2}\right) = 28.52\ plf$ — Maximum Chord Force on Shear Walls

SHEAR LOAD CALCULATIONS:

$L_1 := Width = 8.92\ ft$ Length of Gridline 1 $T_{width\_1} := \frac{Length}{2} = 13\ ft$ Tributary Width

$f_1 := \frac{w_{design\_1} \cdot T_{width\_1}}{L_1} = 57.04\ plf$ In-Plane Force along Gridline 1

$L_2 := Width = 8.92\ ft$ Length of Gridline 2 $T_{width\_2} := \frac{Length}{2} = 13\ ft$ Tributary Width

$f_2 := \frac{w_{design\_1} \cdot T_{width\_2}}{L_2} = 57.04\ plf$ In-Plane Force along Gridline 2

$L_A := Length = 26\ ft$ Length of Gridline A $T_{width\_A} := \frac{Width}{2} = 4.46\ ft$ Tributary Width

$f_A := \frac{w_{design\_2} \cdot T_{width\_A}}{L_A} = 19.57\ plf$ In-Plane Force along Gridline A

$L_B := 22.07\ ft$ Length of Gridline B $T_{width\_B} := \frac{Width}{2} = 4.46\ ft$ Tributary Width

$f_B := \frac{w_{design\_2} \cdot T_{width\_B}}{L_B} = 23.05\ plf$ In-Plane Force along Gridline B

$R := \frac{H_w}{min\left(L_1, L_2, L_A, L_B, L_{chord\_1\_1}, L_{chord\_2\_2}\right)} = 2.24$ Worst Case Shape Ratio

$F_{all\_inplane} := 175\ plf$ Allowable In-Plane Shear (Per LARR/Testing Report)

CHECK: $F_{all\_inplane} \geq max\left(f_1, f_2, f_A, f_B, f_{chord}\right) = 1$

<u>SUMMARY: USE SPECIFIED WALL PANELS PER PLANS.</u>

CEILING PANEL TO WALL PANEL CONNECTION (DETAIL 1)

LOADS:

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

$p_{trans} := P_{internal} \cdot \frac{H_w}{2} = 25.03\ plf$ — Transverse Shear Force on Wall-Ceiling Connection

$f_{inplane} := \max\left(f_1, f_2, f_A, f_B, f_{chord}\right) = 57.04\ plf$ — In-Plane Shear Force on Wall-Ceiling Connection

$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 57.04\ plf$ — Governing Shear Force on Wall-Ceiling Connection

#14 TEK SCREW:

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

$V_{all\_screw} := \frac{76\ lbf}{S_{screw}} = 76\ plf$ — Allowable Shear on Screw (ESR-1976)

$T_{all\_screw} := \frac{57\ lbf}{S_{screw}} = 57\ plf$ — Allowable Tension on Screw (ESR-1976)

CHECK $\frac{V_{all\_screw}}{f_{inplane}} + \frac{T_{all\_screw}}{p_{trans}} \geq 1 = 1$

SUMMARY: THE IMPOSED FORCE ON THE CONNECTION IS LESS THAN THE ALLOWABLE FORCE. THEREFORE, THE CEILING TO WALL PANEL CONNECTION IS ACCEPTABLE .

WALL TO FLOOR CONNECTION (DETAIL 2)

LOADS:

$P_{internal} = 5\ psf$	Transverse Load on Wall
$p_{trans} := P_{internal} \cdot \frac{H_w}{2} = 25.03\ plf$	Transverse Shear Force on Floor-Wall Connection
$f_{inplane} := \max\left(f_1, f_2, f_A, f_B, f_{chord}\right) = 57.04\ plf$	In-Plane Shear Force on Floor-Wall Connection
$f_{max} := \max\left(p_{trans}, f_{inplane}\right) = 57.04\ plf$	Governing Shear Force on Floor-Wall Connection

#14 TEK SCREW:

$L_{screw} := 1.5\ in$	Length of Screws
$S_{screw} := 12\ in$	Spacing of Screws
$C_D := 1.6$	Load Duration Factor
$V_{all\_screw} := \frac{C_D \cdot 104.7\ lbf}{S_{screw}} = 167.52\ plf$	Allowable Shear of #14 TEK (Dowel Bearing Strength - NDS)
$T_{all\_screw} := \frac{C_D \cdot 0.95\ in \cdot 121\ \frac{lbf}{in}}{S_{screw}} = 183.92\ plf$	Allowable Tension of #14 TEK (NDS)

3/8" SIMPSON TITEN HD:

$S_{anchor} := 24\ in$	Spacing of Anchor Through Angle
$\Omega := 2.0$	Overstrength Factor
$V_{anchor} := \frac{\Omega \cdot f_{max} \cdot S_{anchor}}{0.7} = 325.94\ lbf$	LRFD Shear Force on Anchor
$V_{all\_anchor} := 1650\ lbf$	Allowable Shear on Anchor

NOTE: SEE SIMPSON REPORT FOR THE ANCHOR ANALYSIS.

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

SUMMARY:THE MAXIMUM FORCE ON THE CONNECTION IS LESS THAN THE ALLOWABLE FORCE. THEREFORE, THE WALL-FLOOR CONNECTION IS ACCEPTABLE FOR RESISTING DESIGN LOADS.

OVERTURNING CALCULATIONS FOR CONTINUOUS ANGLE (WORST CASE)

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

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

$H_w = 10.01\ ft$ — Height of Wall Panel

ASD LOADS:

$f := f_1 = 57.04\ plf$ — In-Plane Force on Wall In ASD

$L := L_1 = 8.92\ ft$ — Length of Wall

$S_{DS} := 1.014$ — Seismic Design Value

$Wt_{wall} := (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{panel} \cdot H_w \cdot L = 204.49\ lbf$ — Weight of Wall

$Wt_{ceiling} := (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 80.08\ lbf$ — Weight of Ceiling

$$w_R := \frac{Wt_{wall} + Wt_{ceiling}}{L} = 31.9\ plf$$
Weight Resisting Overturning

$$M_{wall} := f \cdot L \cdot H_w - w_R \cdot \frac{L^2}{2} = 3823.76\ lbf \cdot ft$$
Overturning Moment Acting on Wall

$$w := \frac{3 \cdot M_{wall}}{L^2} = 144.17\ plf$$
Maximum Value of Overturning Force at End
$\left(M_{wall} := \frac{1}{2} \cdot w \cdot L \cdot \frac{2\ L}{3} = 3823.76\ lbf \cdot ft \right)$

#14 TEK SCREWS:

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

$n_{screw} := 1$ — Number of screw within the Spacing Considered

$V_{des\_screw} := n_{screw} \cdot C_D \cdot 104.7\ lbf = 167.52\ lbf$ — Allowable Shear of #14 TEK (Dowel Bearing Strength - NDS)

$$T_{des\_screw} := n_{screw} \cdot C_D \cdot 0.95\ in \cdot 121\ \frac{lbf}{in} = 183.92\ lbf$$
Allowable Tension of #14 TEK (NDS)

$V_{screw\_inplane} := f \cdot S_{screw} = 57.04\ lbf$ — Maximum Shear Force on End Screw due to Inplane Shear

$$V_{screw\_uplift} := \frac{\frac{w \cdot (L - S_{screw})}{L} + w}{2} \cdot S_{screw} = 136.09\ lbf$$
Maximum Shear Force on End Screw due to Uplift

$$V_{screw} := \sqrt{V_{screw\_inplane}^2 + V_{screw\_uplift}^2} = 147.56\ lbf$$
Maximum Resultant Shear Force on End Screw

$T_{screw} := p_{trans} \cdot S_{screw} = 25.03\ lbf$ — Maximum Tension Force on End Screw

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

LRFD LOADS:

$f := \frac{f}{0.7} = 81.48\ plf$ In-Plane Force on Wall In LRFD

$L := L = 8.92\ ft$ Length of Wall

$Wt_{wall} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot H_w \cdot L = 311.26\ lbf$ Weight of Wall

$Wt_{ceiling} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 121.89\ lbf$ Weight of Ceiling

$w_R := \frac{Wt_{wall} + Wt_{ceiling}}{L} = 48.56\ plf$ Weight Resisting Overturning

$M_{wall} := f \cdot L \cdot H_w - w_R \cdot \frac{L^2}{2} = 5343.76\ lbf \cdot ft$ Overturning Moment Acting on Wall

$w := \frac{3 \cdot M_{wall}}{L^2} = 201.48\ plf$ Maximum Value of Overturning Force at End $\left(M_{wall} := \frac{1}{2} \cdot w \cdot L \cdot \frac{2\ L}{3} = 5343.76\ lbf \cdot ft\right)$

3/8" SIMPSON TITEN HD:

$S_{anchor} := 24\ in$ Spacing of Anchor

$v_{anchor} := \Omega \cdot f \cdot S_{anchor} = 325.94\ lbf$ Ultimate Governing Shear on Anchor

$T_{anchor} := \Omega \cdot \frac{\frac{w \cdot (L - S_{anchor})}{L} + w}{2} \cdot S_{anchor} = 715.58\ lbf$ Maximum Tension Force on End Anchor

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

SUMMARY: USE SPECIFIED CONNECTION PER PLANS.

WALK-IN DESIGN CRITERIA <UNIT 6>

$Length := 147.58\ ft$ — Unit Length

$H := 14\ ft$ — Unit Height

$H_w := 10.06\ ft$ — Wall Height

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

SUSPENDED PARAPET WALL ALL THREAD ANALYSIS (DETAIL 4)

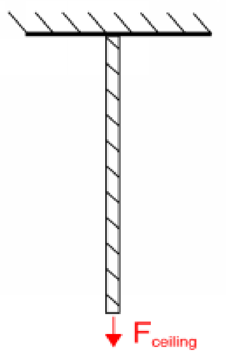
$H_p := 3.92\ ft$ — Height of Parapet Wall

$S_{all\_thread} := 48\ in$ — Spacing of All-Thread

LOADS:

$P_{DL} := DL_{panel} \cdot H_p \cdot S_{all\_thread} = 78.4\ lbf$ — Total Dead Load on Rod

ALL-THREAD ROD:



$\Omega := 1.67$ — ASD Factor

$K := 1.0$

$F_y := 36\ ksi$ — Nominal Yield Strength of All-Thread

$E := 29000\ ksi$ — Modulus of Elasticity for Steel

$d_{rod} := 0.375\ in$ — Diameter of All-Thread Rod

$$A_{gross} := \pi \cdot \left(\frac{d_{rod}}{2}\right)^2 = 0.11\ in^2$$ — Gross Cross-Sectional Area of Rod

ASTM STEEL CONSTRUCTION MANUAL CHAP. D - MEMBERS IN TENSION:

$T_n := F_y \cdot A_{gross} = 3976.078\ lbf$ — Nominal Yield Strength of Rod

$$T_{all} := \frac{T_n}{\Omega} = 2380.885\ lbf$$ — Allowable Yield Strength of Rod

THREADED ROD CHECK

$P_{DL} = 78.4\ lbf$ — Total Dead Load on All Threaded Rod

CHECK $P_{DL} \leq T_{all} = 1$

SUMMARY: THE ALLOWABLE LOADS ARE GREATER THAN THE ACTUAL LOADS ON THE ROD. THEREFORE, USE 3/8" THREADED ROD TO SUSPEND CEILING PANELS FROM EXISTING STRUCTURE PER PLAN. CONNECTION TO EXISTING STRUCTURES BY OTHERS.

TYPICAL PARAPET TO CEILING CONNECTION (DETAIL 3)

$H_p = 3.92\ ft$ — Design Height

LOADS:

$p_{trans} := P_{internal} \cdot \frac{H_p}{2} = 9.8\ plf$ — Transverse Shear Force on Floor-Wall Connection

#8 TEK SCREW:

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

$T_{all\_screw} := \frac{29\ lbf}{S_{screw}} = 29\ plf$ — Allowable Shear Load (SSMA)

CHECK $p_{trans} \le T_{all\_screw} = 1$

SUMMARY: THE MAXIMUM FORCE ON THE CONNECTION IS LESS THAN THE ALLOWABLE FORCE; THEREFORE, THE WALL-FLOOR CONNECTION IS ACCEPTABLE FOR RESISTING IMPOSED LOADS.

SUSPENDED CEILING ALL THREAD ANALYSIS <ROTISSERIE>

$T_{width} := 3.92\ ft$ — Tributary Width of Panel on Suspension Line

$S_{all\_thread} := 29.75\ in$ — Spacing of All-Thread

$S_{DS} := 1.014$ — Seismic Design Value at 0.2s SA

LOADS:

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

$w_{design} := LC_3 \cdot T_{width} = 58.8\ plf$ — Governing Load Combination for Ceiling

$P_{DL} := DL_{panel} \cdot T_{width} \cdot S_{all\_thread} = 48.59\ lbf$ — Total Dead Load on Rod

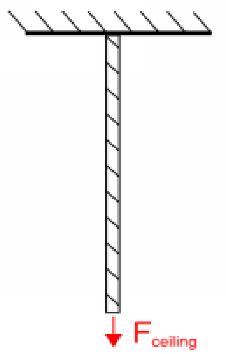
$P_{LL\_thread} := \max\left(LL_{panel} \cdot T_{width} \cdot S_{all\_thread}, P_{LL}\right) = 300\ lbf$ — Total Live Load on Rod

$F_{p\_vert} := 0.2 \cdot S_{DS} \cdot P_{DL} = 9.9\ lbf$ — Generated Seismic Uplift Force per Rod (12.4.2.2)

$C_{rod} := 0.7 \cdot F_{p\_vert} - 0.6 \cdot w_{design} \cdot S_{all\_thread} = -80.6\ lbf$ — Compression Load per Rod

NOTE: NEGATIVE VALUE INDICATES NO COMPRESSION IS PRESENT

ALL-THREAD ROD:



$\Omega := 1.67$ — ASD Factor

$K := 1.0$

$F_y := 36\ ksi$ — Nominal Yield Strength of All-Thread

$E := 29000\ ksi$ — Modulus of Elasticity for Steel

$d_{rod} := 0.375\ in$ — Diameter of All-Thread Rod

$A_{gross} := \pi \cdot \left(\frac{d_{rod}}{2}\right)^2 = 0.11\ in^2$ — Gross Cross-Sectional Area of Rod

ASTM STEEL CONSTRUCTION MANUAL CHAP. D - MEMBERS IN TENSION:

$T_n := F_y \cdot A_{gross} = 3976.078\ lbf$ — Nominal Yield Strength of Rod

$T_{all} := \frac{T_n}{\Omega} = 2380.885\ lbf$ — Allowable Yield Strength of Rod

THREADED ROD CHECK

$P_{DL} + P_{LL\_thread} = 348.59\ lbf$ — Total Load on All Threaded Rod

CHECK $P_{DL} + P_{LL\_thread} \le T_{all} = 1$

<u>SUMMARY: THE ALLOWABLE LOADS ARE GREATER THAN THE ACTUAL LOADS ON THE ROD. THEREFORE, USE 3/8" THREADED ROD TO SUSPEND CEILING PANELS FROM EXISTING STRUCTURE PER PLAN. CONNECTION TO EXISTING STRUCTURES BY OTHERS.</u>

SOFTWARE PRINTOUTS (ENERCALC)

Steel Beam

Project File: Daily Cooler.ec6

LIC# : KW-06013705, Build:20.21.10.30 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: < UNIT 1> W12x40 Beam

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

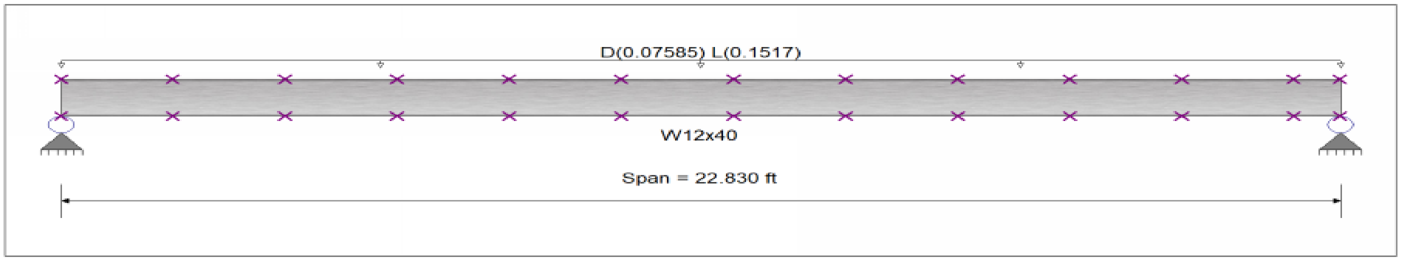
Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at ft from Left-Most support
Regular spacing of lateral supports on length of beam = 2.0 ft



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.0050, L = 0.010 ksf, Tributary Width = 15.170 ft, (Panel Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.123** : 1	Maximum Shear Stress Ratio =	**0.043** : 1
Section used for this span	**W12x40**	Section used for this span	**W12x40**
Ma : Applied	17.431 k-ft	Va : Applied	3.054 k
Mn / Omega : Allowable	142.216 k-ft	Vn/Omega : Allowable	70.210 k
Load Combination	+D+L	Load Combination	+D+L
Location of maximum on span	11.415ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.105 in	Ratio =	2,618	>=240.	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<240.0	Span: 1 : L Only
Max Downward Total Deflection	0.185 in	Ratio =	1485	>=180	Span: 1 : +D+L
Max Upward Total Deflection	0.000 in	Ratio =	0	<180	

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 1.96 ft	1	0.017	0.019	2.37		2.37	237.50	142.22	1.65	1.00	1.32	105.32	70.21
Dsgn. L = 2.02 ft	1	0.031	0.016	4.34	2.37	4.34	237.50	142.22	1.22	1.00	1.10	105.32	70.21
Dsgn. L = 1.96 ft	1	0.041	0.012	5.81	4.34	5.81	237.50	142.22	1.11	1.00	0.86	105.32	70.21
Dsgn. L = 2.02 ft	1	0.048	0.009	6.86	5.81	6.86	237.50	142.22	1.06	1.00	0.63	105.32	70.21
Dsgn. L = 2.02 ft	1	0.052	0.006	7.43	6.86	7.43	237.50	142.22	1.03	1.00	0.40	105.32	70.21
Dsgn. L = 1.96 ft	1	0.053	0.002	7.55	7.43	7.55	237.50	142.22	1.00	1.00	0.17	105.32	70.21
Dsgn. L = 2.02 ft	1	0.053	0.004	7.53	7.17	7.53	237.50	142.22	1.01	1.00	0.29	105.32	70.21
Dsgn. L = 2.02 ft	1	0.050	0.008	7.17	6.34	7.17	237.50	142.22	1.04	1.00	0.53	105.32	70.21
Dsgn. L = 1.96 ft	1	0.045	0.011	6.34	5.08	6.34	237.50	142.22	1.08	1.00	0.76	105.32	70.21
Dsgn. L = 2.02 ft	1	0.036	0.014	5.08	3.32	5.08	237.50	142.22	1.15	1.00	0.99	105.32	70.21
Dsgn. L = 2.02 ft	1	0.023	0.017	3.32	1.08	3.32	237.50	142.22	1.33	1.00	1.22	105.32	70.21
Dsgn. L = 0.85 ft	1	0.008	0.019	1.08		1.08	237.50	142.22	1.57	1.00	1.32	105.32	70.21
+D+L													
Dsgn. L = 1.96 ft	1	0.038	0.043	5.46		5.46	237.50	142.22	1.65	1.00	3.05	105.32	70.21
Dsgn. L = 2.02 ft	1	0.071	0.036	10.03	5.46	10.03	237.50	142.22	1.22	1.00	2.53	105.32	70.21
Dsgn. L = 1.96 ft	1	0.094	0.028	13.42	10.03	13.42	237.50	142.22	1.11	1.00	1.99	105.32	70.21
Dsgn. L = 2.02 ft	1	0.111	0.021	15.83	13.42	15.83	237.50	142.22	1.06	1.00	1.47	105.32	70.21
Dsgn. L = 2.02 ft	1	0.121	0.013	17.16	15.83	17.16	237.50	142.22	1.03	1.00	0.92	105.32	70.21

Steel Beam

Project File: Daily Cooler.ec6

LIC# : KW-06013705, Build:20.21.10.30 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: < UNIT 1> W12x40 Beam

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
Dsgn. L = 1.96 ft	1	0.123	0.005	17.43	17.16	17.43	237.50	142.22	1.00	1.00	0.38	105.32	70.21
Dsgn. L = 2.02 ft	1	0.122	0.010	17.39	16.57	17.39	237.50	142.22	1.01	1.00	0.68	105.32	70.21
Dsgn. L = 2.02 ft	1	0.116	0.017	16.57	14.64	16.57	237.50	142.22	1.04	1.00	1.22	105.32	70.21
Dsgn. L = 1.96 ft	1	0.103	0.025	14.64	11.74	14.64	237.50	142.22	1.08	1.00	1.75	105.32	70.21
Dsgn. L = 2.02 ft	1	0.083	0.033	11.74	7.66	11.74	237.50	142.22	1.15	1.00	2.29	105.32	70.21
Dsgn. L = 2.02 ft	1	0.054	0.040	7.66	2.49	7.66	237.50	142.22	1.33	1.00	2.83	105.32	70.21
Dsgn. L = 0.85 ft	1	0.018	0.043	2.49		2.49	237.50	142.22	1.57	1.00	3.05	105.32	70.21
+D+0.750L													
Dsgn. L = 1.96 ft	1	0.033	0.037	4.69		4.69	237.50	142.22	1.65	1.00	2.62	105.32	70.21
Dsgn. L = 2.02 ft	1	0.061	0.031	8.61	4.69	8.61	237.50	142.22	1.22	1.00	2.17	105.32	70.21
Dsgn. L = 1.96 ft	1	0.081	0.024	11.51	8.61	11.51	237.50	142.22	1.11	1.00	1.71	105.32	70.21
Dsgn. L = 2.02 ft	1	0.096	0.018	13.59	11.51	13.59	237.50	142.22	1.06	1.00	1.26	105.32	70.21
Dsgn. L = 2.02 ft	1	0.104	0.011	14.72	13.59	14.72	237.50	142.22	1.03	1.00	0.79	105.32	70.21
Dsgn. L = 1.96 ft	1	0.105	0.005	14.96	14.72	14.96	237.50	142.22	1.00	1.00	0.33	105.32	70.21
Dsgn. L = 2.02 ft	1	0.105	0.008	14.93	14.22	14.93	237.50	142.22	1.01	1.00	0.58	105.32	70.21
Dsgn. L = 2.02 ft	1	0.100	0.015	14.22	12.57	14.22	237.50	142.22	1.04	1.00	1.05	105.32	70.21
Dsgn. L = 1.96 ft	1	0.088	0.021	12.57	10.08	12.57	237.50	142.22	1.08	1.00	1.50	105.32	70.21
Dsgn. L = 2.02 ft	1	0.071	0.028	10.08	6.58	10.08	237.50	142.22	1.15	1.00	1.96	105.32	70.21
Dsgn. L = 2.02 ft	1	0.046	0.035	6.58	2.14	6.58	237.50	142.22	1.33	1.00	2.43	105.32	70.21
Dsgn. L = 0.85 ft	1	0.015	0.037	2.14		2.14	237.50	142.22	1.57	1.00	2.62	105.32	70.21
+0.60D													
Dsgn. L = 1.96 ft	1	0.010	0.011	1.42		1.42	237.50	142.22	1.65	1.00	0.79	105.32	70.21
Dsgn. L = 2.02 ft	1	0.018	0.009	2.61	1.42	2.61	237.50	142.22	1.22	1.00	0.66	105.32	70.21
Dsgn. L = 1.96 ft	1	0.025	0.007	3.49	2.61	3.49	237.50	142.22	1.11	1.00	0.52	105.32	70.21
Dsgn. L = 2.02 ft	1	0.029	0.005	4.11	3.49	4.11	237.50	142.22	1.06	1.00	0.38	105.32	70.21
Dsgn. L = 2.02 ft	1	0.031	0.003	4.46	4.11	4.46	237.50	142.22	1.03	1.00	0.24	105.32	70.21
Dsgn. L = 1.96 ft	1	0.032	0.001	4.53	4.46	4.53	237.50	142.22	1.00	1.00	0.10	105.32	70.21
Dsgn. L = 2.02 ft	1	0.032	0.003	4.52	4.30	4.52	237.50	142.22	1.01	1.00	0.18	105.32	70.21
Dsgn. L = 2.02 ft	1	0.030	0.005	4.30	3.80	4.30	237.50	142.22	1.04	1.00	0.32	105.32	70.21
Dsgn. L = 1.96 ft	1	0.027	0.006	3.80	3.05	3.80	237.50	142.22	1.08	1.00	0.45	105.32	70.21
Dsgn. L = 2.02 ft	1	0.021	0.008	3.05	1.99	3.05	237.50	142.22	1.15	1.00	0.59	105.32	70.21
Dsgn. L = 2.02 ft	1	0.014	0.010	1.99	0.65	1.99	237.50	142.22	1.33	1.00	0.73	105.32	70.21
Dsgn. L = 0.85 ft	1	0.005	0.011	0.65		0.65	237.50	142.22	1.57	1.00	0.79	105.32	70.21

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.1845	11.480		0.0000	0.000

Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.054	3.054
Overall MINimum	-0.793	-0.793
D Only	1.322	1.322
+D+L	3.054	3.054
+D+0.750L	2.621	2.621
+0.60D	0.793	0.793
L Only	1.732	1.732

Steel Beam

Project File: Daily Cooler.ec6

LIC# : KW-06013705, Build:20.21.10.30   TAMARACK GROVE ENGINEERING   (c) ENERCALC INC 1983-2021

DESCRIPTION: <UNIT 1> W12x22 Beam

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

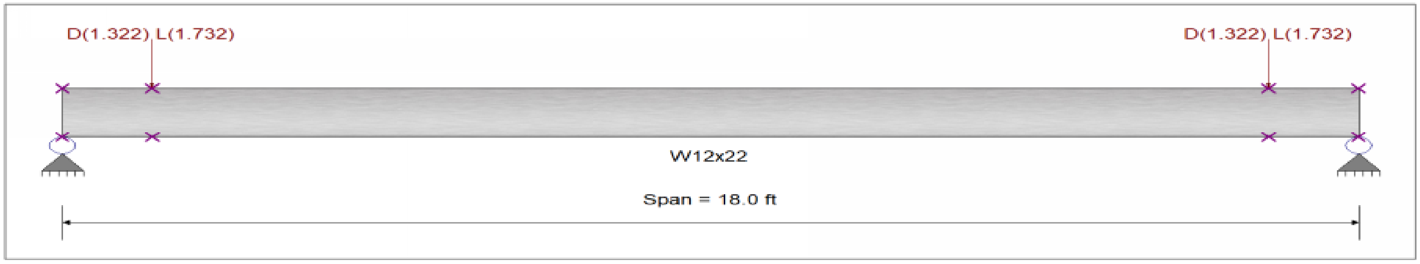
Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam bracing is defined Beam-by-Beam
Bending Axis : Major Axis Bending

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

Span # 1, Defined Brace Locations, First Brace at ft, Second Brace at 1.250 ft, Third Brace at 16.750 ft



Applied Loads

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

Beam self weight calculated and added to loading
Load(s) for Span Number 1
Point Load : D = 1.322, L = 1.732 k @ 1.250 ft, (Reaction from W12x35 beam)

Point Load : D = 1.322, L = 1.732 k @ 16.750 ft, (Reaction from W12x35 beam)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.219 : 1	Maximum Shear Stress Ratio =	0.051 : 1
Section used for this span	W12x22	Section used for this span	W12x22
Ma : Applied	4.709 k-ft	Va : Applied	3.252 k
Mn / Omega : Allowable	21.464 k-ft	Vn/Omega : Allowable	63.960 k
Load Combination	+D+L	Load Combination	+D+L
Location of maximum on span	9.000 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.033 in	Ratio =	6,456	>=240.	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<240.0	Span: 1 : L Only
Max Downward Total Deflection	0.071 in	Ratio =	3063	>=180	Span: 1 : +D+L
Max Upward Total Deflection	0.000 in	Ratio =	0	<180	

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 1.23 ft	1	0.025	0.024	1.86		1.86	122.08	73.10	1.66	1.00	1.52	95.94	63.96
Dsgn. L = 15.48 ft	1	0.117	0.023	2.54	1.86	2.54	36.37	21.78	1.03	1.00	1.49	95.94	63.96
Dsgn. L = 1.29 ft	1	0.026	0.024	1.89		1.89	122.08	73.10	1.59	1.00	1.52	95.94	63.96
+D+L													
Dsgn. L = 1.23 ft	1	0.055	0.051	4.00		4.00	122.08	73.10	1.67	1.00	3.25	95.94	63.96
Dsgn. L = 15.48 ft	1	0.219	0.050	4.71	4.00	4.71	35.85	21.46	1.02	1.00	3.22	95.94	63.96
Dsgn. L = 1.29 ft	1	0.055	0.051	4.05		4.05	122.08	73.10	1.59	1.00	3.25	95.94	63.96
+D+0.750L													
Dsgn. L = 1.23 ft	1	0.047	0.044	3.46		3.46	122.08	73.10	1.66	1.00	2.82	95.94	63.96
Dsgn. L = 15.48 ft	1	0.194	0.044	4.17	3.46	4.17	35.92	21.51	1.02	1.00	2.79	95.94	63.96
Dsgn. L = 1.29 ft	1	0.048	0.044	3.51		3.51	122.08	73.10	1.59	1.00	2.82	95.94	63.96
+0.60D													
Dsgn. L = 1.23 ft	1	0.015	0.014	1.12		1.12	122.08	73.10	1.66	1.00	0.91	95.94	63.96
Dsgn. L = 15.48 ft	1	0.070	0.014	1.53	1.12	1.53	36.37	21.78	1.03	1.00	0.90	95.94	63.96
Dsgn. L = 1.29 ft	1	0.016	0.014	1.13		1.13	122.08	73.10	1.59	1.00	0.91	95.94	63.96

Steel Beam

Project File: Daily Cooler.ec6

LIC# : KW-06013705, Build:20.21.10.30 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: <UNIT 1> W12x22 Beam

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0705	9.051		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.252	3.252
Overall MINimum	-0.912	-0.912
D Only	1.520	1.520
+D+L	3.252	3.252
+D+0.750L	2.819	2.819
+0.60D	0.912	0.912
L Only	1.732	1.732

Steel Column

Project File: Daily Cooler.ec6

LIC# : KW-06013705, Build:20.21.10.30 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: <UNIT 1> > HSS5x5x3/16 Column

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name : **HSS5x5x3/16**

Analysis Method :	Allowable Strength
Steel Stress Grade	, A500, Grade B, Fy = 46 ksi, Carbon Steel
Fy : Steel Yield	46.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height 18.580 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for buckling ABOUT Y-Y Axis = 18.580 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 18.580 ft, K = 1.0

Applied Loads

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

Column self weight included : 222.403 lbs * Dead Load Factor
AXIAL LOADS . . .
Reaction from W12x22 Beam: Axial Load at 18.580 ft, D = 1.520, L = 1.732 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.09183**	: 1
Load Combination	+D+L	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pa : Axial	3.474	k
Pn / Omega : Allowabl	37.834	k
Ma-x : Applied	0.0	k-ft
Mn-x / Omega : Allowable	13.520	k-ft
Ma-y : Applied	0.0	k-ft
Mn-y / Omega : Allowable	13.520	k-ft
PASS Maximum Shear Stress Rati	**0.0**	: 1
Load Combination	0.0	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	0.0	k
Vn / Omega : Allowable	0.0	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	0.0 k

Maximum Load Deflections . . .

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

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.046	PASS	0.00 ft	1.00	1.00	113.76	113.76	0.000	PASS	0.00 ft
+D+L	0.092	PASS	0.00 ft	1.00	1.00	113.76	113.76	0.000	PASS	0.00 ft
+D+0.750L	0.080	PASS	0.00 ft	1.00	1.00	113.76	113.76	0.000	PASS	0.00 ft
+0.60D	0.028	PASS	0.00 ft	1.00	1.00	113.76	113.76	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
D Only	1.742								
+D+L	3.474								
+D+0.750L	3.041								
+0.60D	1.045								
L Only	1.732								

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
Axial @ Base	Maximum	3.474								
"	Minimum	1.045								

Steel Column

Project File: Daily Cooler.ec6

LIC# : KW-06013705, Build:20.21.10.30 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: <UNIT 1> > HSS5x5x3/16 Column

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	k @ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments @ Base	k-ft @ Top	My - End Moments @ Base	@ Top
Reaction, X-X Axis Base	Maximum	1.742								
"	Minimum	1.742								
Reaction, Y-Y Axis Base	Maximum	1.742								
"	Minimum	1.742								
Reaction, X-X Axis Top	Maximum	1.742								
"	Minimum	1.742								
Reaction, Y-Y Axis Top	Maximum	1.742								
"	Minimum	1.742								
Moment, X-X Axis Base	Maximum	1.742								
"	Minimum	1.742								
Moment, Y-Y Axis Base	Maximum	1.742								
"	Minimum	1.742								
Moment, X-X Axis Top	Maximum	1.742								
"	Minimum	1.742								
Moment, Y-Y Axis Top	Maximum	1.742								
"	Minimum	1.742								

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L	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
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Steel Section Properties : HSS5x5x3/16

Depth	=	5.000 in	I xx	=	12.60 in^4	J	=	19.900 in^4
Design Thick	=	0.174 in	S xx	=	5.03 in^3			
Width	=	5.000 in	R xx	=	1.960 in			
Wall Thick	=	0.187 in	Zx	=	5.890 in^3			
Area	=	3.280 in^2	I yy	=	12.600 in^4	C	=	8.080 in^3
Weight	=	11.970 plf	S yy	=	5.030 in^3			
			R yy	=	1.960 in			

Ycg	=	0.000 in

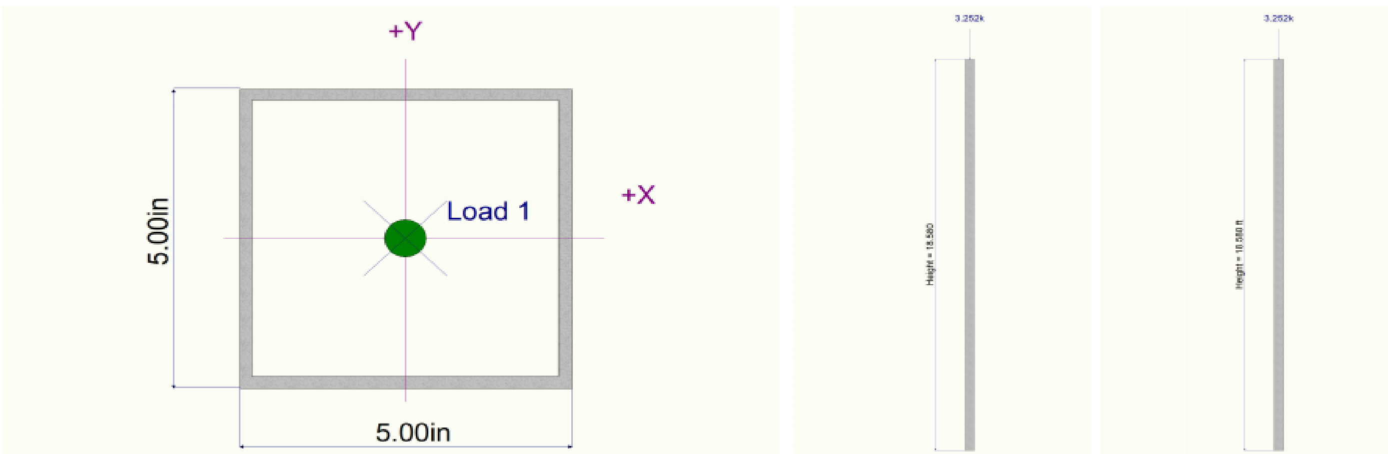
Steel Column

Project File: Daily Cooler.ec6

LIC# : KW-06013705, Build:20.21.10.30 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: <UNIT 1> > HSS5x5x3/16 Column

Sketches



Steel Beam

Project File: Meat Cooler.ec6

LIC# : KW-06013705, Build:20.21.10.30 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: < UNIT 5> W12x14 Beam

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

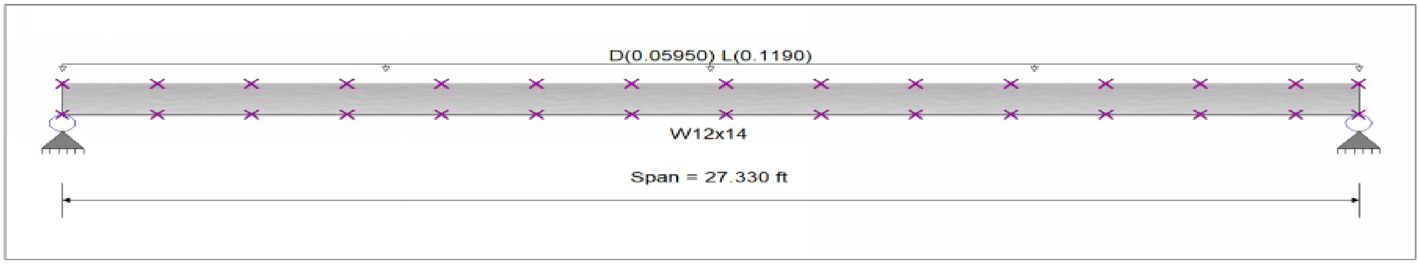
Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at ft from Left-Most support
Regular spacing of lateral supports on length of beam = 2.0 ft



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.0050, L = 0.010 ksf, Tributary Width = 11.90 ft, (Panel Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.414 : 1	Maximum Shear Stress Ratio =	0.062 : 1
Section used for this span	**W12x14**	Section used for this span	**W12x14**
Ma : Applied	17.973 k-ft	Va : Applied	2.631 k
Mn / Omega : Allowable	43.413 k-ft	Vn/Omega : Allowable	42.754 k
Load Combination	+D+L	Load Combination	+D+L
Location of maximum on span	13.665 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.584 in	Ratio =	561	>=240.	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<240.0	Span: 1 : L Only
Max Downward Total Deflection	0.945 in	Ratio =	347	>=180	Span: 1 : +D+L
Max Upward Total Deflection	0.000 in	Ratio =	0	<180	

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 1.95 ft	1	0.042	0.023	1.82		1.82	72.50	43.41	1.67	1.00	1.00	71.40	42.75
Dsgn. L = 2.03 ft	1	0.079	0.020	3.42	1.82	3.42	72.50	43.41	1.23	1.00	0.86	71.40	42.75
Dsgn. L = 1.95 ft	1	0.107	0.017	4.67	3.42	4.67	72.50	43.41	1.12	1.00	0.71	71.40	42.75
Dsgn. L = 2.03 ft	1	0.131	0.013	5.67	4.67	5.67	72.50	43.41	1.07	1.00	0.57	71.40	42.75
Dsgn. L = 2.03 ft	1	0.147	0.010	6.37	5.67	6.37	72.50	43.41	1.04	1.00	0.42	71.40	42.75
Dsgn. L = 1.95 ft	1	0.156	0.006	6.75	6.37	6.75	72.50	43.41	1.02	1.00	0.27	71.40	42.75
Dsgn. L = 2.03 ft	1	0.158	0.003	6.86	6.75	6.86	72.50	43.41	1.00	1.00	0.13	71.40	42.75
Dsgn. L = 1.95 ft	1	0.158	0.004	6.86	6.67	6.86	72.50	43.41	1.01	1.00	0.17	71.40	42.75
Dsgn. L = 2.03 ft	1	0.154	0.007	6.67	6.18	6.67	72.50	43.41	1.03	1.00	0.32	71.40	42.75
Dsgn. L = 2.03 ft	1	0.142	0.011	6.18	5.39	6.18	72.50	43.41	1.05	1.00	0.46	71.40	42.75
Dsgn. L = 1.95 ft	1	0.124	0.014	5.39	4.34	5.39	72.50	43.41	1.08	1.00	0.61	71.40	42.75
Dsgn. L = 2.03 ft	1	0.100	0.018	4.34	2.96	4.34	72.50	43.41	1.14	1.00	0.76	71.40	42.75
Dsgn. L = 1.95 ft	1	0.068	0.021	2.96	1.34	2.96	72.50	43.41	1.25	1.00	0.90	71.40	42.75
Dsgn. L = 1.41 ft	1	0.031	0.023	1.34		1.34	72.50	43.41	1.61	1.00	1.00	71.40	42.75
+D+L													
Dsgn. L = 1.95 ft	1	0.110	0.062	4.77		4.77	72.50	43.41	1.67	1.00	2.63	71.40	42.75
Dsgn. L = 2.03 ft	1	0.206	0.053	8.95	4.77	8.95	72.50	43.41	1.23	1.00	2.25	71.40	42.75
Dsgn. L = 1.95 ft	1	0.282	0.044	12.22	8.95	12.22	72.50	43.41	1.12	1.00	1.86	71.40	42.75

Steel Beam

Project File: Meat Cooler.ec6

LIC# : KW-06013705, Build:20.21.10.30 TAMARACK GROVE ENGINEERING

DESCRIPTION: < UNIT 5> W12x14 Beam

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
Dsgn. L = 2.03 ft	1	0.342	0.035	14.85	12.22	14.85	72.50	43.41	1.07	1.00	1.49	71.40	42.75
Dsgn. L = 2.03 ft	1	0.384	0.026	16.68	14.85	16.68	72.50	43.41	1.04	1.00	1.10	71.40	42.75
Dsgn. L = 1.95 ft	1	0.407	0.017	17.69	16.68	17.69	72.50	43.41	1.02	1.00	0.71	71.40	42.75
Dsgn. L = 2.03 ft	1	0.414	0.008	17.97	17.69	17.97	72.50	43.41	1.00	1.00	0.33	71.40	42.75
Dsgn. L = 1.95 ft	1	0.414	0.010	17.96	17.48	17.96	72.50	43.41	1.01	1.00	0.44	71.40	42.75
Dsgn. L = 2.03 ft	1	0.403	0.019	17.48	16.20	17.48	72.50	43.41	1.03	1.00	0.83	71.40	42.75
Dsgn. L = 2.03 ft	1	0.373	0.028	16.20	14.12	16.20	72.50	43.41	1.05	1.00	1.22	71.40	42.75
Dsgn. L = 1.95 ft	1	0.325	0.037	14.12	11.38	14.12	72.50	43.41	1.08	1.00	1.59	71.40	42.75
Dsgn. L = 2.03 ft	1	0.262	0.046	11.38	7.75	11.38	72.50	43.41	1.14	1.00	1.98	71.40	42.75
Dsgn. L = 1.95 ft	1	0.178	0.055	7.75	3.51	7.75	72.50	43.41	1.25	1.00	2.36	71.40	42.75
Dsgn. L = 1.41 ft	1	0.081	0.062	3.51		3.51	72.50	43.41	1.61	1.00	2.63	71.40	42.75
+D+0.750L													
Dsgn. L = 1.95 ft	1	0.093	0.052	4.03		4.03	72.50	43.41	1.67	1.00	2.22	71.40	42.75
Dsgn. L = 2.03 ft	1	0.174	0.045	7.57	4.03	7.57	72.50	43.41	1.23	1.00	1.91	71.40	42.75
Dsgn. L = 1.95 ft	1	0.238	0.037	10.33	7.57	10.33	72.50	43.41	1.12	1.00	1.58	71.40	42.75
Dsgn. L = 2.03 ft	1	0.289	0.029	12.55	10.33	12.55	72.50	43.41	1.07	1.00	1.26	71.40	42.75
Dsgn. L = 2.03 ft	1	0.325	0.022	14.10	12.55	14.10	72.50	43.41	1.04	1.00	0.93	71.40	42.75
Dsgn. L = 1.95 ft	1	0.344	0.014	14.96	14.10	14.96	72.50	43.41	1.02	1.00	0.60	71.40	42.75
Dsgn. L = 2.03 ft	1	0.350	0.007	15.20	14.96	15.20	72.50	43.41	1.00	1.00	0.28	71.40	42.75
Dsgn. L = 1.95 ft	1	0.350	0.009	15.19	14.78	15.19	72.50	43.41	1.01	1.00	0.37	71.40	42.75
Dsgn. L = 2.03 ft	1	0.340	0.016	14.78	13.69	14.78	72.50	43.41	1.03	1.00	0.70	71.40	42.75
Dsgn. L = 2.03 ft	1	0.315	0.024	13.69	11.94	13.69	72.50	43.41	1.05	1.00	1.03	71.40	42.75
Dsgn. L = 1.95 ft	1	0.275	0.032	11.94	9.62	11.94	72.50	43.41	1.08	1.00	1.35	71.40	42.75
Dsgn. L = 2.03 ft	1	0.222	0.039	9.62	6.55	9.62	72.50	43.41	1.14	1.00	1.68	71.40	42.75
Dsgn. L = 1.95 ft	1	0.151	0.047	6.55	2.97	6.55	72.50	43.41	1.25	1.00	2.00	71.40	42.75
Dsgn. L = 1.41 ft	1	0.068	0.052	2.97		2.97	72.50	43.41	1.61	1.00	2.22	71.40	42.75
+0.60D													
Dsgn. L = 1.95 ft	1	0.025	0.014	1.09		1.09	72.50	43.41	1.67	1.00	0.60	71.40	42.75
Dsgn. L = 2.03 ft	1	0.047	0.012	2.05	1.09	2.05	72.50	43.41	1.23	1.00	0.52	71.40	42.75
Dsgn. L = 1.95 ft	1	0.064	0.010	2.80	2.05	2.80	72.50	43.41	1.12	1.00	0.43	71.40	42.75
Dsgn. L = 2.03 ft	1	0.078	0.008	3.40	2.80	3.40	72.50	43.41	1.07	1.00	0.34	71.40	42.75
Dsgn. L = 2.03 ft	1	0.088	0.006	3.82	3.40	3.82	72.50	43.41	1.04	1.00	0.25	71.40	42.75
Dsgn. L = 1.95 ft	1	0.093	0.004	4.05	3.82	4.05	72.50	43.41	1.02	1.00	0.16	71.40	42.75
Dsgn. L = 2.03 ft	1	0.095	0.002	4.12	4.05	4.12	72.50	43.41	1.00	1.00	0.08	71.40	42.75
Dsgn. L = 1.95 ft	1	0.095	0.002	4.12	4.00	4.12	72.50	43.41	1.01	1.00	0.10	71.40	42.75
Dsgn. L = 2.03 ft	1	0.092	0.004	4.00	3.71	4.00	72.50	43.41	1.03	1.00	0.19	71.40	42.75
Dsgn. L = 2.03 ft	1	0.085	0.007	3.71	3.24	3.71	72.50	43.41	1.05	1.00	0.28	71.40	42.75
Dsgn. L = 1.95 ft	1	0.075	0.009	3.24	2.61	3.24	72.50	43.41	1.08	1.00	0.37	71.40	42.75
Dsgn. L = 2.03 ft	1	0.060	0.011	2.61	1.77	2.61	72.50	43.41	1.14	1.00	0.45	71.40	42.75
Dsgn. L = 1.95 ft	1	0.041	0.013	1.77	0.80	1.77	72.50	43.41	1.25	1.00	0.54	71.40	42.75
Dsgn. L = 1.41 ft	1	0.019	0.014	0.80		0.80	72.50	43.41	1.61	1.00	0.60	71.40	42.75

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.9448	13.743		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.631	2.631
Overall MINimum	-0.603	-0.603
D Only	1.004	1.004
+D+L	2.631	2.631
+D+0.750L	2.224	2.224
+0.60D	0.603	0.603
L Only	1.626	1.626

Steel Column

Project File: Meat Cooler.ec6

LIC# : KW-06013705, Build:20.21.10.30 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: <UNIT 5> > HSS4x4x3/16 Column

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name : **HSS4x4x3/16**
Analysis Method : Allowable Strength
Steel Stress Grade , A500, Grade B, Fy = 46 ksi, Carbon Steel
Fy : Steel Yield 46.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 19.580 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for buckling ABOUT Y-Y Axis = 19.580 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 19.580 ft, K = 1.0

Applied Loads

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

Column self weight included : 184.444 lbs * Dead Load Factor
AXIAL LOADS . . .
Reaction from W12x22 Beam: Axial Load at 19.580 ft, D = 1.004, L = 1.626 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.1668	: 1
Load Combination	+D+L	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pa : Axial	2.814	k
Pn / Omega : Allowabl	16.876	k
Ma-x : Applied	0.0	k-ft
Mn-x / Omega : Allowable	8.424	k-ft
Ma-y : Applied	0.0	k-ft
Mn-y / Omega : Allowable	8.424	k-ft
PASS Maximum Shear Stress Rati	0.0	: 1
Load Combination	0.0	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	0.0	k
Vn / Omega : Allowable	0.0	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	0.0	k

Maximum Load Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination :
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/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.070	PASS	0.00 ft	1.00	1.00	151.59	151.59	0.000	PASS	0.00 ft
+D+L	0.167	PASS	0.00 ft	1.00	1.00	151.59	151.59	0.000	PASS	0.00 ft
+D+0.750L	0.143	PASS	0.00 ft	1.00	1.00	151.59	151.59	0.000	PASS	0.00 ft
+0.60D	0.042	PASS	0.00 ft	1.00	1.00	151.59	151.59	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments (k-ft) @ Base	Mx - End Moments (k-ft) @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only	1.188								
+D+L	2.814								
+D+0.750L	2.408								
+0.60D	0.713								
L Only	1.626								

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments (k-ft) @ Base	Mx - End Moments (k-ft) @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	2.814								
"	Minimum	0.713								

Steel Column

Project File: Meat Cooler.ec6

LIC# : KW-06013705, Build:20.21.10.30 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: <UNIT 5> > HSS4x4x3/16 Column

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
Reaction, X-X Axis Base	Maximum	1.188								
"	Minimum	1.188								
Reaction, Y-Y Axis Base	Maximum	1.188								
"	Minimum	1.188								
Reaction, X-X Axis Top	Maximum	1.188								
"	Minimum	1.188								
Reaction, Y-Y Axis Top	Maximum	1.188								
"	Minimum	1.188								
Moment, X-X Axis Base	Maximum	1.188								
"	Minimum	1.188								
Moment, Y-Y Axis Base	Maximum	1.188								
"	Minimum	1.188								
Moment, X-X Axis Top	Maximum	1.188								
"	Minimum	1.188								
Moment, Y-Y Axis Top	Maximum	1.188								
"	Minimum	1.188								

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L	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
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Steel Section Properties : HSS4x4x3/16

Depth	=	4.000 in	I xx	=	6.21 in^4	J	=	10.000 in^4
Design Thick	=	0.174 in	S xx	=	3.10 in^3			
Width	=	4.000 in	R xx	=	1.550 in			
Wall Thick	=	0.187 in	Zx	=	3.670 in^3			
Area	=	2.580 in^2	I yy	=	6.210 in^4	C	=	5.070 in^3
Weight	=	9.420 plf	S yy	=	3.100 in^3			
			R yy	=	1.550 in			
Ycg	=	0.000 in						

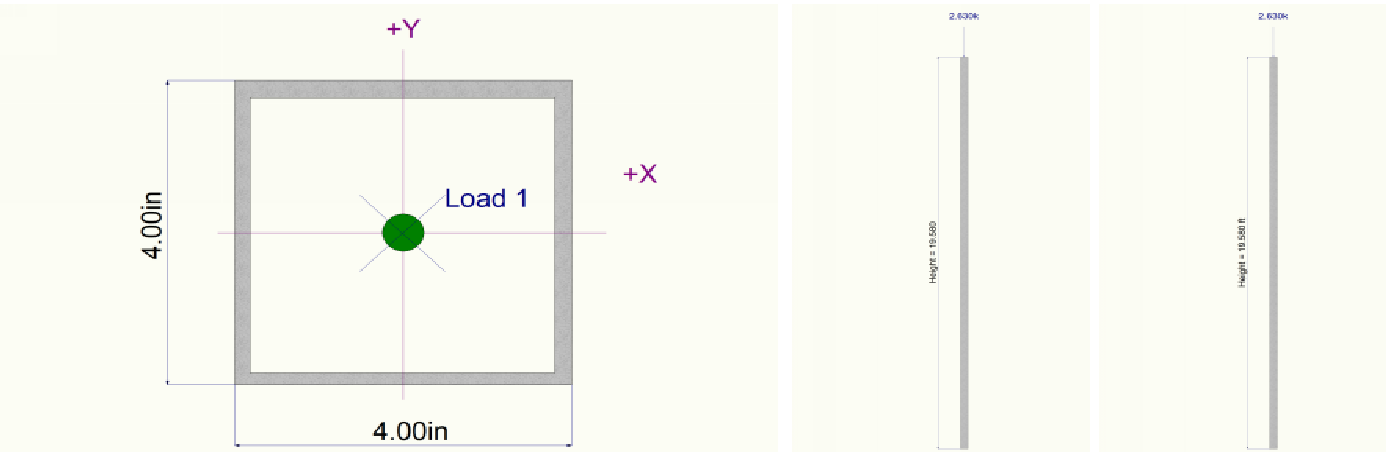
Steel Column

Project File: Meat Cooler.ec6

LIC# : KW-06013705, Build:20.21.10.30 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2021

DESCRIPTION: <UNIT 5> > HSS4x4x3/16 Column

Sketches



SOFTWARE PRINTOUTS (SIMPSON ANCHORS)

SIMPSON Strong-Tie®

Anchor Designer™ Software
Version 2.9.7376.6

Company:		Date:	12/3/2021
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: TITEN HD MAX SHEAR
Location:
Fastening description:

2. Input Data & Anchor Parameters

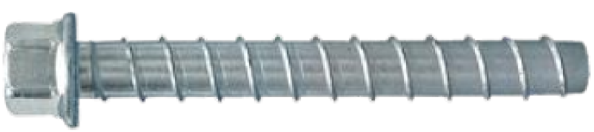
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.375
Nominal Embedment depth (inch): 3.000
Effective Embedment depth, h_{ef} (inch): 2.190
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 4.67
c_{ac} (inch): 3.31
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 6.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 2.00 x 24.00 x 0.05

Recommended Anchor
Anchor Name: Titen HD® - 3/8"Ø Titen HD, hnom:3" (76mm)
Code Report: ICC-ES ESR-2713



SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.9.7376.6

Company:		Date:	12/3/2021
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Project:			
Address:			
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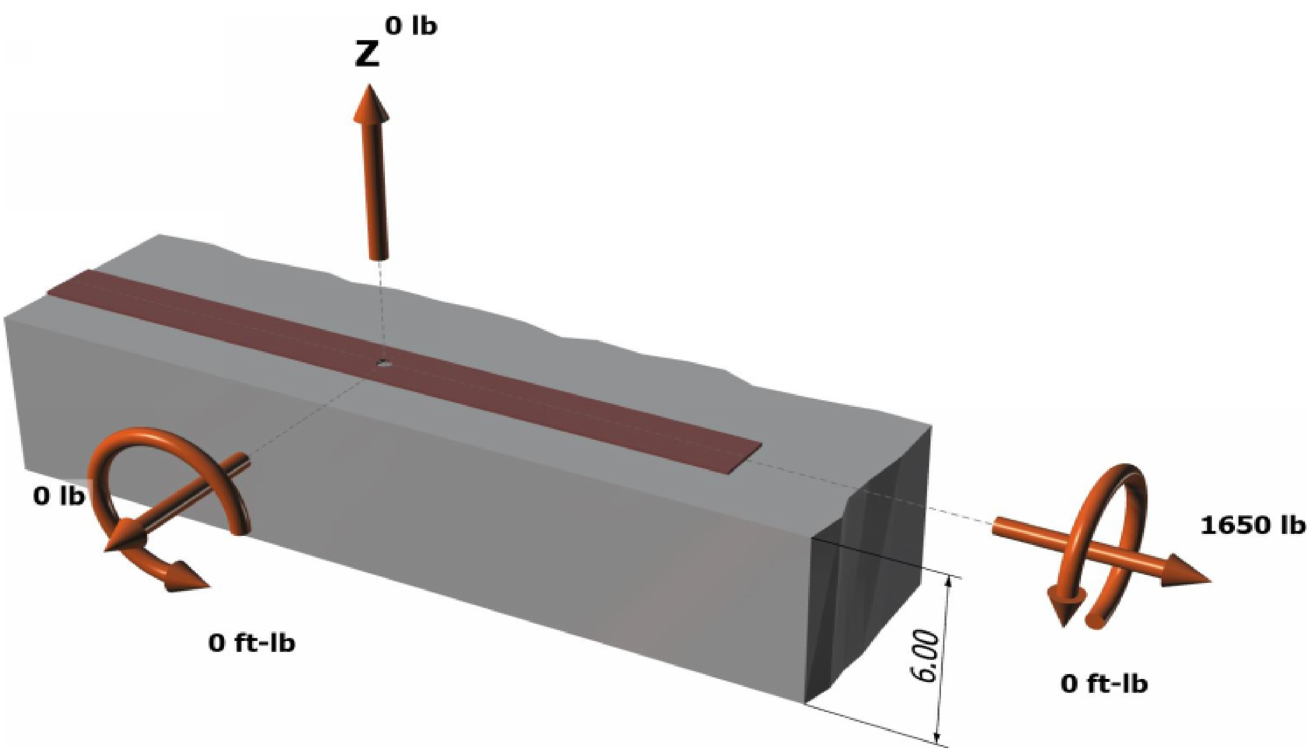
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (b) is satisfied
Ductility section for shear: 17.2.3.5.3 (b) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 0
V_{uay} [lb]: 1650
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0

<Figure 1>



SIMPSON Strong-Tie® Anchor Designer™ Software
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Company:		Date:	12/3/2021
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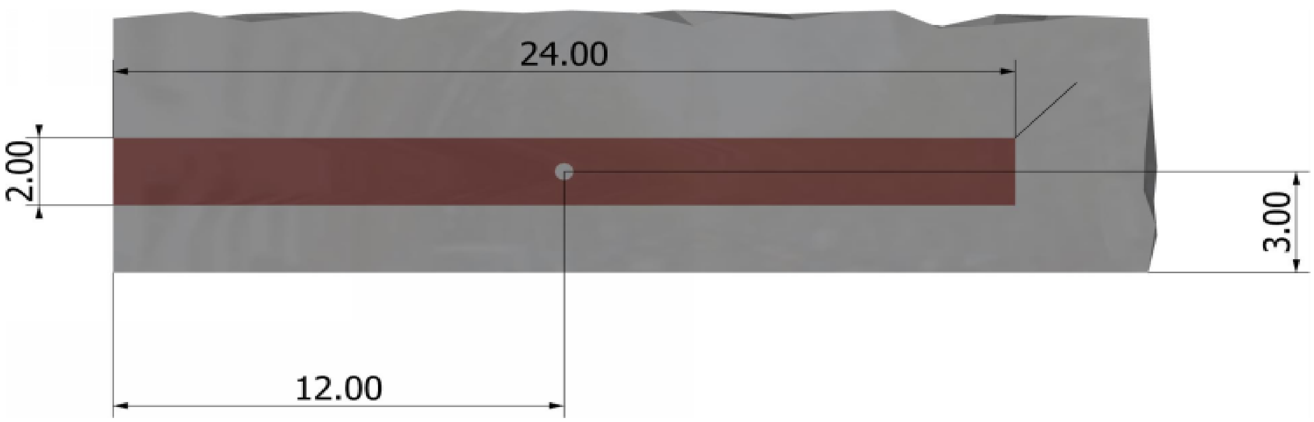
E-mail:			

<Figure 2>



SIMPSON Strong-Tie®

Anchor Designer™ Software
Version 2.9.7376.6

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3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	0.0	1650.0	1650.0
Sum	0.0	0.0	1650.0	1650.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
2855	1.0	0.60	1713

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
2.19	0.375	1.00	2500	3.00	1585

$\phi V_{cby} = \phi\,(2)(A_{Vc} / A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cby} (lb)
40.50	40.50	1.000	1.000	1.000	1585	0.70	2219

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cp} = \phi k_{cp}N_{cb} = \phi k_{cp}(A_{Nc} / A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1a)

k_{cp}	A_{Nc} (in²)	A_{Nco} (in²)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
1.0	41.29	43.16	0.974	1.000	1.000	2755	0.70	1797

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status

Steel	**1650**	**1713**	**0.96**	**Pass (Governs)**
|| Concrete breakout x+	1650	2219	0.74	Pass
Pryout	1650	1797	0.92	Pass

3/8"Ø Titen HD, hnom:3" (76mm) meets the selected design criteria.

12. Warnings

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.
- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.
- Designer must exercise own judgement to determine if this design is suitable.
- Refer to manufacturer's product literature for hole cleaning and installation instructions.

SIMPSON Strong-Tie®

Anchor Designer™
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Version 2.9.7376.7

Company:		Date:	12/3/2021
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Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: COMBO OT
Location:
Fastening description:

2. Input Data & Anchor Parameters

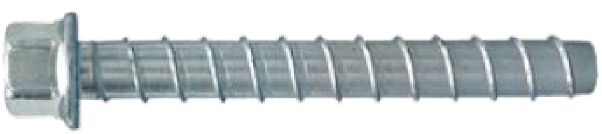
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.375
Nominal Embedment depth (inch): 3.000
Effective Embedment depth, h_{ef} (inch): 2.190
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 4.67
c_{ac} (inch): 3.31
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 6.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 2.00 x 24.00 x 0.05

Recommended Anchor
Anchor Name: Titen HD® - 3/8"Ø Titen HD, hnom:3" (76mm)
Code Report: ICC-ES ESR-2713



SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.9.7376.7

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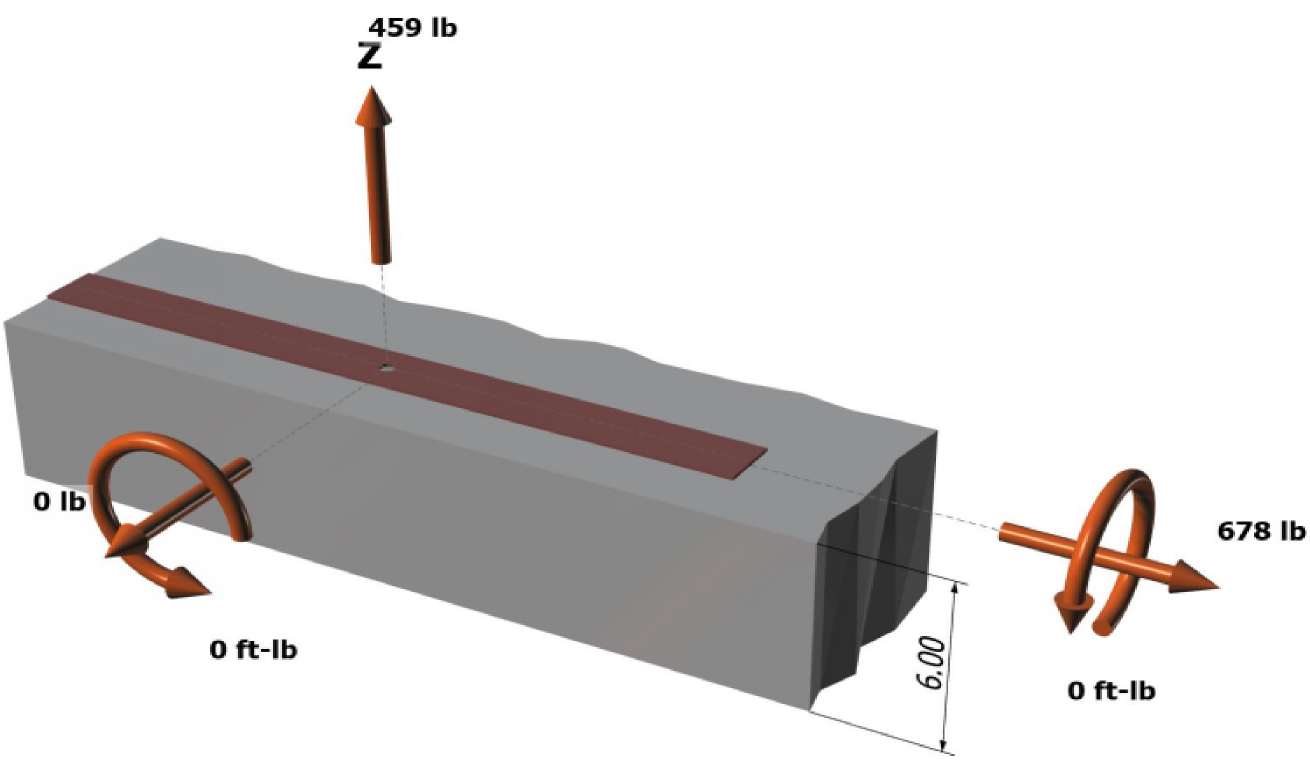
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (b) is satisfied
Ductility section for shear: 17.2.3.5.3 (b) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]: 459
V_{uax} [lb]: 0
V_{uay} [lb]: 678
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0

<Figure 1>



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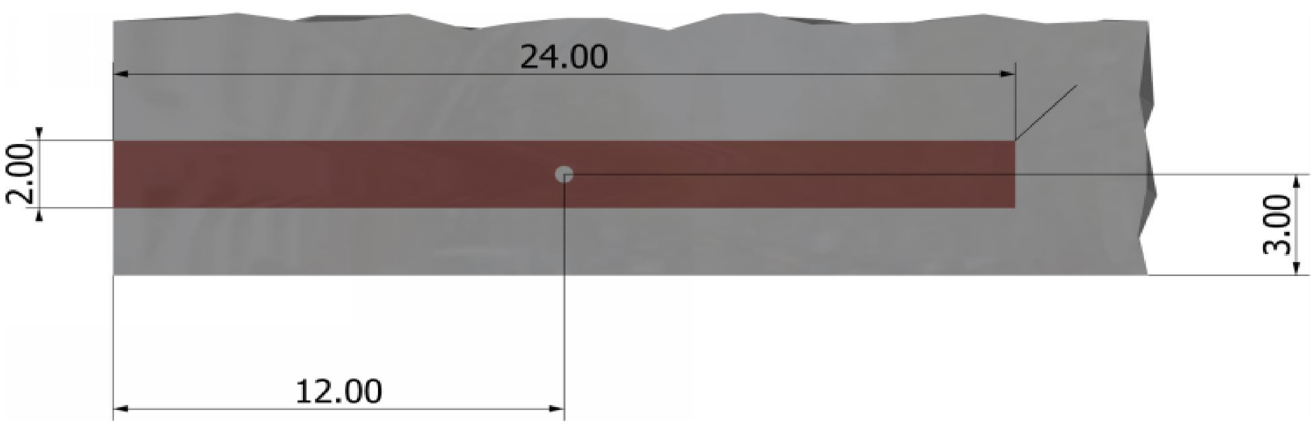
E-mail:			

<Figure 2>



SIMPSON Strong-Tie® Anchor Designer™ Software Version 2.9.7376.7

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E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	459.0	0.0	678.0	678.0
Sum	459.0	0.0	678.0	678.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 459
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
10890	0.65	7079

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	2.190	2755

$0.75\phi N_{cb} = 0.75\phi\,(A_{Nc}/A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
41.29	43.16	3.00	0.974	1.00	1.000	2755	0.65	1251

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}\lambda_a N_p (f'_c/2{,}500)^n$ (Sec. 17.3.1, Eq. 17.4.3.1 & Code Report)

$\Psi_{c,P}$	λ_a	N_p (lb)	f'_c (psi)	n	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	1.00	2212	2500	0.50	0.65	1078

SIMPSON Strong-Tie®

Anchor Designer™ Software
Version 2.9.7376.7

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Address:			
Phone:			
E-mail:			

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
2855	1.0	0.60	1713

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
2.19	0.375	1.00	2500	3.00	1585

$\phi V_{cby} = \phi\ (2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cby} (lb)
40.50	40.50	1.000	1.000	1.000	1585	0.70	2219

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cp} = \phi k_{cp}N_{cb} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1a)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
1.0	41.29	43.16	0.974	1.000	1.000	2755	0.70	1797

11. Results

Interaction of Tensile and Shear Forces (Sec. R17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	459	7079	0.06	Pass
Concrete breakout	459	1251	0.37	Pass
Pullout	**459**	**1078**	**0.43**	**Pass (Governs)**

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	**678**	**1713**	**0.40**	**Pass (Governs)**
|| Concrete breakout x+	678	2219	0.31	Pass
Pryout	678	1797	0.38	Pass

Interaction check	$(N_{ua}/\phi N_{ua})^{5/3}$	$(V_{ua}/\phi V_{ua})^{5/3}$	Combined Ratio	Permissible	Status
Sec. R17.6	0.24	0.21	45.4%	1.0	Pass

3/8"Ø Titen HD, hnom:3" (76mm) meets the selected design criteria.

SIMPSON Strong-Tie®

Anchor Designer™ Software
Version 2.9.7376.7

Company:		Date:	12/3/2021
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Address:			
Phone:			
E-mail:			

12. Warnings

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.
- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.
- Designer must exercise own judgement to determine if this design is suitable.
- Refer to manufacturer's product literature for hole cleaning and installation instructions.

SIMPSON Strong-Tie®

Anchor Designer™
Software
Version 2.9.7376.8

Company:		Date:	12/3/2021
Engineer:		Page:	1/6
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: UNIT 3 OT
Location:
Fastening description:

2. Input Data & Anchor Parameters

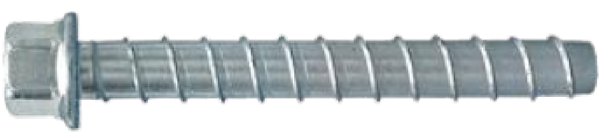
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.375
Nominal Embedment depth (inch): 3.000
Effective Embedment depth, h_{ef} (inch): 2.190
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 4.67
c_{ac} (inch): 3.31
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 6.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 2.00 x 24.00 x 0.05

Recommended Anchor
Anchor Name: Titen HD® - 3/8"Ø Titen HD, hnom:3" (76mm)
Code Report: ICC-ES ESR-2713



Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
Simpson Strong-Tie Company Inc. 5956 W. Las Positas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.847.3871 www.strongtie.com

SIMPSON Strong-Tie® Anchor Designer™ Software Version 2.9.7376.8

Company:		Date:	12/3/2021
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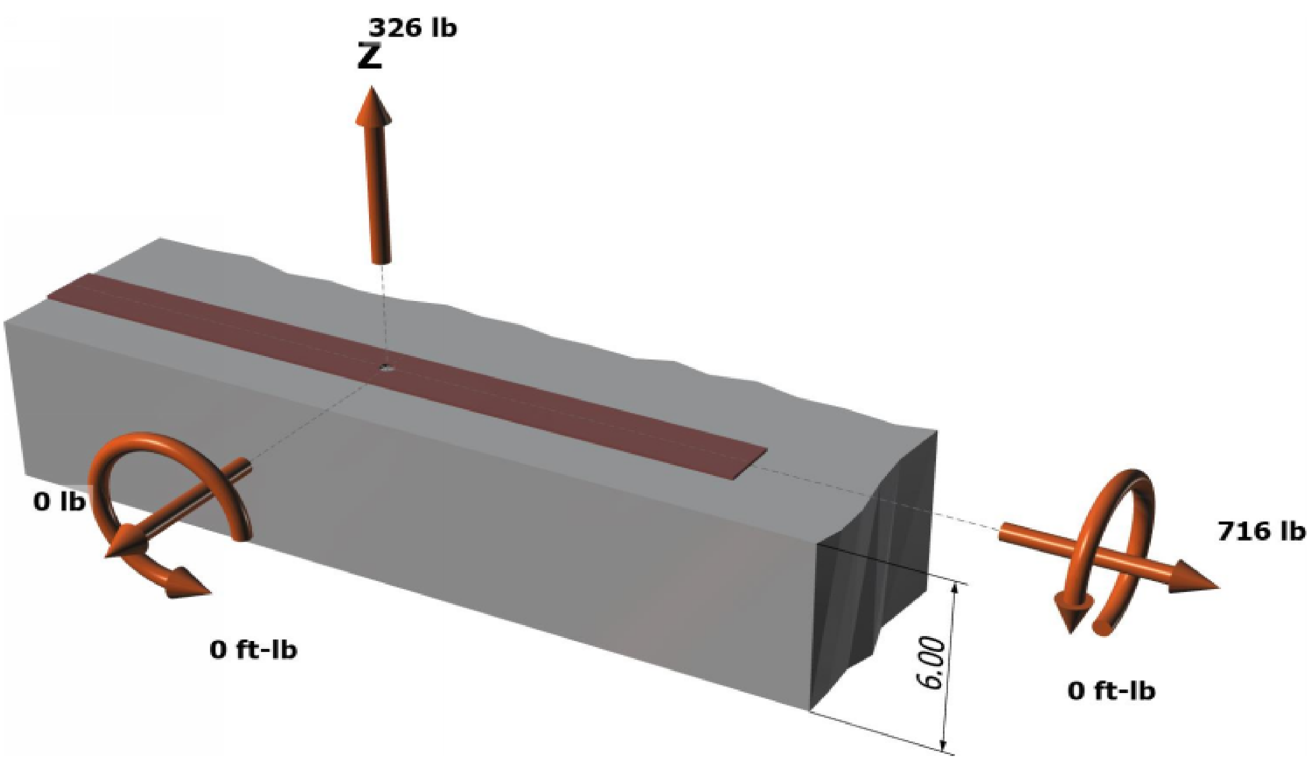
Project:			
Address:			
Phone:			
E-mail:			

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (b) is satisfied
Ductility section for shear: 17.2.3.5.3 (b) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]: 326
V_{uax} [lb]: 0
V_{uay} [lb]: 716
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0

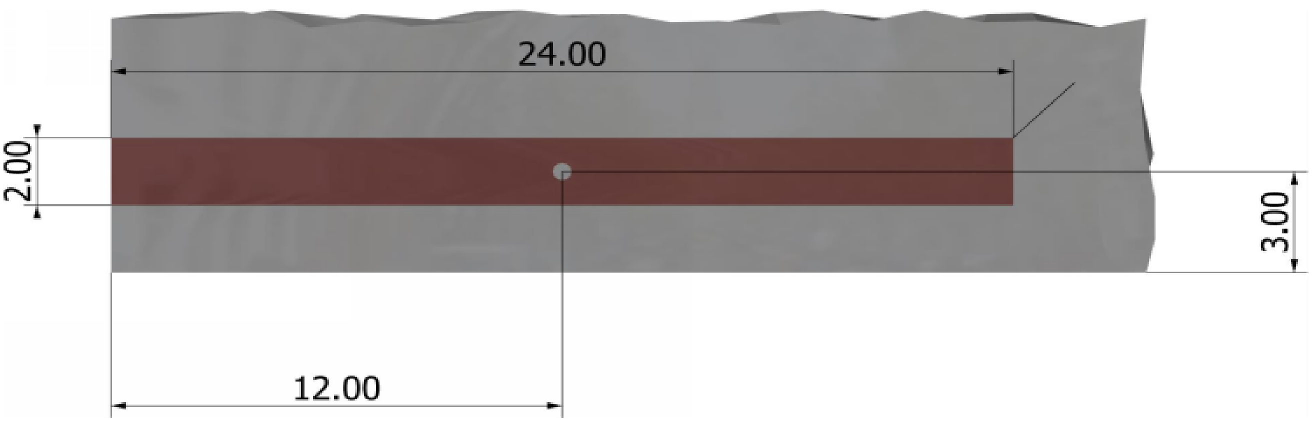
<Figure 1>



SIMPSON Strong-Tie® Anchor Designer™ Software Version 2.9.7376.8

Company:		Date:	12/3/2021
Engineer:		Page:	3/6
Project:			
Address:			
Phone:			
E-mail:			

<Figure 2>



SIMPSON Strong-Tie® Anchor Designer™ Software Version 2.9.7376.8

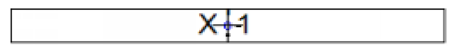
Company:		Date:	12/3/2021
Engineer:		Page:	4/6
Project:			
Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	326.0	0.0	716.0	716.0
Sum	326.0	0.0	716.0	716.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 326
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
10890	0.65	7079

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	2.190	2755

$0.75\phi N_{cb} = 0.75\phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in²)	A_{Nco} (in²	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
41.29	43.16	3.00	0.974	1.00	1.000	2755	0.65	1251

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}\lambda_a N_p(f'_c / 2{,}500)^n$ (Sec. 17.3.1, Eq. 17.4.3.1 & Code Report)

$\Psi_{c,P}$	λ_a	N_p (lb)	f'_c (psi)	n	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	1.00	2212	2500	0.50	0.65	1078

Simpson Strong-Tie Company Inc. 5956 W. Las Positas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.847.3871 www.strongtie.com

SIMPSON Strong-Tie® Anchor Designer™ Software Version 2.9.7376.8

Company:		Date:	12/3/2021
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Address:			
Phone:			
E-mail:			

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
2855	1.0	0.60	1713

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
2.19	0.375	1.00	2500	3.00	1585

$\phi V_{cby} = \phi\ (2)(A_{Vc} / A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cby} (lb)
40.50	40.50	1.000	1.000	1.000	1585	0.70	2219

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cp} = \phi k_{cp}N_{cb} = \phi k_{cp}(A_{Nc} / A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1a)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
1.0	41.29	43.16	0.974	1.000	1.000	2755	0.70	1797

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	326	7079	0.05	Pass
Concrete breakout	326	1251	0.26	Pass
Pullout	**326**	**1078**	**0.30**	**Pass (Governs)**

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	**716**	**1713**	**0.42**	**Pass (Governs)**
|| Concrete breakout x+	716	2219	0.32	Pass
Pryout	716	1797	0.40	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..2	0.00	0.42	41.8%	1.0	Pass

3/8"Ø Titen HD, hnom:3" (76mm) meets the selected design criteria.

12. Warnings

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

TESTING REPORT OR LARR

CITY OF LOS ANGELES

CALIFORNIA

ERIC GARCETTI
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

OSAMA YOUNAN, P.E.
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

KPS GLOBAL, LLC
4201 North Beach St.
Fortworth, TX 76137

Attn: Joe James
(682) 317-5305

RESEARCH REPORT: RR 24921
(CSI # 13030)

Expires: June 1, 2022
Issued Date: June 1, 2020
Code: 2020 LABC

GENERAL APPROVAL –Renewal, Clerical Modification and Technical Modification – KPS Global - Wood-Frame and Insul-Frame Wall and Ceiling Panels for Walk-in Coolers and Freezers for Indoor and Outdoor use.

DESCRIPTION

KPS Global, prefabricated wall and ceiling panels consist of minimum 3½-inch wood or urethane lumber frames with 26 gage steel and a core of urethane foam. The panels utilize "cam-lock" devices to hold them together.

The wood Frame Panels are made of S-P-F No. 2 or better, while the Insul-Frame panels are made of moulded urethane foam having a density of 10.3 pounds per cubic foot. Fusion frame panels are made of moulded urethane foam having a density of 5.25 pounds per cubic foot with a wood or non-wood structural backing. The urethane foam is Dow or BASF Class 1 foam.

Flame spread and smoke developed ratings per ASTM E-84 of 25 or less and 450 or less respectively.

KPS Global-Wood Frame, Insul-Frame and Fusion Frame fabricated walk-in coolers and freezers constructed of panels described above are approved with the following requirements:

1. Height to width ratio of units are as indicated in Table 2 and Table 4.

KPS Global
RE: Wall and Ceiling Panels for Walk-in Coolers and Freezers

2. No permanent loads, equipment or storage loads shall be carried by the ceiling panels with the exception for the evaporator. If evaporator is supported from the top panel, it must be accounted for in calculations for ceiling panel loads. For equipment loads, calculations demonstrating that the applied loads are less than the maximum allowable loads must be submitted to the structural plan check section for each project. The calculations must be prepared by a California registered Civil Engineer or Architect.

3. The panels shall be considered combustible and may be used only in areas where combustible materials are permitted by the Code.

4. An approved fire retardant roof covering shall be placed over the panels when used as an exterior roof panel.

5. The panels shall be fabricated in a shop of a licensed Type I fabricator approved by the Los Angeles City Building Department. Fabrication in unlicensed shops will invalidate this approval. If piping or other utilities are in the walls or ceilings a type II fabricators license is required.

DISCUSSION

The report is in compliance with the 2020 Los Angeles City Building Code.

The clerical modifications are to change the contact information and update this report to 2020 LABC.

The technical modifications are to modify the 26 gage steel, add Fusion Frame Panels and add Tables 3 and 4.

The approval is based on tests.

Addressee to whom this Research Report is issued is responsible for providing copies of it, complete with any attachments indicated, to architects, engineers and builders using items approved herein in design or construction which must be approved by Department of Building and Safety Engineers and Inspectors.

KPS Global
RE: Wall and Ceiling Panels for Walk-in Coolers and Freezers

This general approval of an equivalent alternate to the Code is only valid where an engineer and/or inspector of this Department has determined that all conditions of this Approval have been met in the project in which it is to be used.

____

DAVID CHANG, Chief
Engineering Research Section
201 N. Figueroa St., Room 880
Los Angeles, CA 90012
Phone- 213-202-9812
Fax- 213-202-9943

DC
RR24921/MSWord2013
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06/09/2020
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Attachment: Span Chart (4 page)

LARR 24921 TABLE 1

PANELS					TOPS/CEILINGS									
					INDOOR Defl L/180	OUTDOOR Includes 2 PSF for Membrane or Standing Seam Roof Design based on deflection criteria: L/240								
					LIVE LOADS – PSF									
FINISH	SKIN THICKNESS	PANEL THICKNESS	DEAD LOAD PSF	R VALUE	10	20	30	40	50	60	70	80	90	100
WOOD FRAME														
ALUM	0.032"	2½"	2.0	20	12'-6"	Do not use 2 1/2" panels outdoors								
		3½"	3.0	28	15'-6"	10'-10"	9'-7"	8'-9"	8'-1"	7'-7"	7'-3"	6'-11"	6'-7"	6'-3"
		4"	3.5	32	17'-7"	14'-4"	11'-9"	10'-2"	9'-1"	8'-3"	7'-7"	7'-1"	6'-8"	6'-4"
		5"	4.0	40	20'-9"	14'-10"	13'-2"	12'-1"	11'-3"	10'-7"	10'-1"	9'-7"	9'-3"	8'-7"
GALV	26 ga.	2½"	2.5	20	13'-8"	Do not use 2 1/2" panels outdoors								
		3½"	3.5	28	17'-6"	13'-3"	11'-1"	9'-9"	8'-10"	8'-2"	7'-7"	7'-2"	6'-9"	6'-5"
		4"	4.0	32	19'-3"	13'-8"	12'-1"	10'-11"	10'-0"	9'-3"	8'-7"	8'-0"	7'-6"	7'-1"
		5"	4.5	40	22'-9"	16'-5"	14'-6"	13'-2"	12'-0"	11'-2"	10'-4"	9'-8"	9'-1"	8'-7"
	20 ga.	5"	5.5	40	26'-0"*	18'-6"	15'-8"	13'-11"	12'-8"	11'-9"	11'-0"	10'-5"	9'-11"	9'-5"
INSULFRAME/HIGH DENSITY RAIL (HDR)														
ALUM	0.032"	3½"	3.0	28	14'-7"	10'-2"	8'-11"	8'-0"	7'-4"	6'-10"	6'-5"	5'-8"	5'-5"	5'-2"
		4"	3.5	32	16'-2"	13'-5"	10'-9"	9'-1"	8'-0"	7'-3"	6'-7"	6'-1"	5'-9"	5'-4"
		5"	4.0	40	19'-4"	13'-10"	12'-2"	11'-0"	10'-1"	9'-5"	8'-11"	7'-10"	7'-6"	7'-2"
GALV	26 ga.	3½"	3.5	28	16'-7"	12'-8"	10'-10"	9'-6"	8'-5"	7'-7"	6'-10"	6'-2"	5'-7"	5'-1"
		4"	4.0	32	18'-1"	12'-6"	10'-11"	9'-9"	8'-9"	8'-0"	7'-4"	6'-10"	6'-4"	5'-10"
		5"	4.5	40	21'-0"	15'-0"	13'-2"	11'-9"	10'-9"	9'-10"	9'-1"	8'-5"	7'-11"	7'-4"
	20 ga.	5"	5.5	40	23'-6"	17'-11"	15'-4"	13'-5"	12'-3"	11'-1"	10'-2"	9'-4"	8'-7"	8'-0"

PANELS					WALLS									
					INDOOR Defl L/180	OUTDOOR ASCE 7-05/10 EXP. C, ≤15' OAH 3sec Gusts Design based on deflection criteria: L/180								
					LIVE LOADS – PSF									
FINISH	SKIN THICKNESS	PANEL THICKNESS	DEAD LOAD PSF	R VALUE	5	85 MPH 110 MPH 13.9	90 MPH 115 MPH 15.5	95 MPH 120 MPH 17.4	100 MPH 130 MPH 19.5	110 MPH 140 MPH 23.2	115 MPH 150 MPH 25.9	125 MPH 160 MPH 29.9	140 MPH 180 MPH 37.6	155 MPH 200 MPH 46.3
2.2-POUND DENSITY														Fc=18ps
ALUM	0.032"	2½"	2.0	20	15'-2"	Do not use 2 1/2" panels outdoors								
		3½"	3.0	28	20'-2"	13'-11"	13'-4"	12'-8"	12'-0"	11'-1"	10'-6"	9'-9"	8'-7"	7'-7"
		4"	3.5	32	22'-6"	15'-0"	14'-4"	13'-8"	13'-0"	12'-4"	11'-9"	11'-0"	9'-11"	8'-11"
		5"	4.0	40	26'-0"*	18'-9"	17'-11"	17'-0"	16'-1"	15'-5"	14'-8"	13'-8"	12'-2"	10'-9"
GALV	26 ga.	2½"	2.5	20	16'-7"	Do not use 2 1/2" panels outdoors								
		3½"	3.5	28	22'-10"	15'-5"	14'-9"	14'-1"	13'-6"	12'-8"	12'-1"	11'-5"	10'-6"	9'-8"
		4"	4.0	32	23'-6"	16'-3"	15'-8"	15'-0"	14'-5"	13'-6"	13'-0"	12'-4"	11'-4"	10'-7"
		5"	4.5	40	26'-0"*	18'-11"	18'-4"	17'-9"	17'-1"	16'-2"	15'-7"	14'-10"	13'-7"	12'-6"
	20 ga.	5"	5.5	40	26'-0"*	26'-0"	24'-11"	22'-7"	20'-0"	15'-11"	15'-9"	15'-7"	15'-2"	14'-8"
2.2-POUND DENSITY														Fc=18ps
ALUM	0.032"	3½"	3.0	28	19'-5"	13'-2"	12'-6"	11'-10"	11'-3"	10'-3"	10'-1"	9'-4"	8'-3"	7'-3"
		4"	3.5	32	19'-9"	13'-3"	12'-8"	12'-1"	11'-7"	10'-10"	10'-4"	9'-9"	8'-11"	8'-3"
		5"	4.0	40	25'-8"	17'-0"	16'-2"	15'-3"	14'-4"	14'-3"	13'-5"	12'-5"	10'-10"	9'-5"
GALV	26 ga.	3½"	3.5	28	22'-0"	14'-11"	14'-2"	13'-6"	12'-10"	11'-11"	11'-4"	10'-8"	9'-7"	8'-9"
		4"	4.0	32	22'-3"	16'-1"	15'-7"	14'-11"	14'-4"	13'-5"	12'-11"	12'-2"	10'-11"	9'-10"
		5"	4.5	40	26'-0"*	17'-6"	16'-11"	16'-2"	15'-6"	14'-6"	13'-10"	13'-0"	11'-8"	10'-6"
	20 ga.	5"	5.5	40	26'-0"*	21'-2"	20'-2"	19'-2"	18'-3"	16'-11"	16'-1"	15'-1"	13'-8"	12'-2"

Wind speeds given in the table are for V_{ASD} and for V_{ULT} - as required by the applicable building code edition (2009/2012+)

*MAX SPAN IS BASED ON MANUFACTURING LIMITATION.

kps GLOBAL

Engineering Bulletin No.: 900

Panel Span Chart

Date: August 2, 1992
Rev: August 3, 2017

REGISTERED PROFESSIONAL ENGINEER
WILLIAM N. BOREN II
No. C32642
CIVIL
STATE OF CALIFORNIA

8-3-17

Sheet Number 147 of 147

LARR 24921 TABLE 2

KPS Global
Racking Shear - Based on PFS Load Test Report # 05-37A Maximum Allowable Shear Load of Wood Framed Panels

Height to Width Ratio	Allowable Shear PLF
4 To 1	160
3 To 1	161
2 To 1	179
1 ½ To 1	246
1 To 1	333
½ To 1	646

KPS Global
Compressive Load - Based on PFS Test Report # 05-37B

Maximum Allowable Vertical Load of Wood Framed Panels

Panel Thickness (Inches)	Panel Height (Feet)	Allowable Vertical Load (PLF)
3 ½	12	2080
3 ½	17	1946
3 ½	22	1033
5	16	2779
5	21	1582
5	26	1037

KPS Global
Racking Shear - Based on PFS Load Test Report # 05-37A

Maximum Allowable Shear Load of Insul-Frame Panels

Height to Width Ratio	Allowable Shear PLF
4 To 1	56
3 To 1	65
2 To 1	88
1 ½ To 1	108
1 To 1	136
½ To 1	208

KPS Global
Compressive Load - Based on PFS Test Report # 05-37B

Maximum Allowable Vertical Load of Insul-Frame Panels

Panel Thickness (Inches)	Panel Height (Feet)	Allowable Vertical Load (PLF)
3 ½	12	920
3 ½	17	714
3 ½	22	603
5	16	1048
5	21	800
5	26	623

terrapintesting
you build it. we'll break it.

HILL PHOENIX WALK-INS

Hill Phoenix Walk-Ins

Structural Insulated Panel Tests-Refrigeration Panels for Walk-In Coolers and Freezers

3.5-Inch, 26ga / 2.25# FIP / 26ga
5.0-Inch, 26ga / 2.25# FIP / 26ga

Report No. TT 511002

April 14, 2011
(REV 0.0)

Embossed Corporate Seal

TERRAPIN TESTING

IAS, INC. NO. TL-159
FLA TST2542
LA TA 10230

Signed:

Rick Cavanagh

Title: **Lab Manager**

Dated: **April 14, 2011**

This Report is for the exclusive use of the Client named herein. Terrapin Testing authorizes the Client to reproduce this report only if reproduced in its entirety.

The results of testing in this report apply only to the samples supplied and under the stated conditions of testing.

The hard copy of this report is the final copy, and supersedes any electronic versions of the same, in case of disputes.

IAS ACCREDITED

FLORIDA DEPARTMENT OF Community Affairs

National Sunroom Association

3206 Luyung Drive
Rancho Cordova, California 95742-6830
Phone: 916.853.9658 | Fax: 916.853.9432
Info: tt@terrapintesting.com | www.terrapintesting.com

HillPHOENIX WALK-INS

Connection Tests

Small scale connection assemblies were built to evaluate the following:

Config #

1 Wall-to-Wall: In Plane Shear
2 Wall-to-Wall: Out of Plane Shear
3 Wall-to-Wall: Tension
4 Wall-to-Roof: In Plane Shear - cam and pin locks; lag screw
5 Wall-to-Roof: Out of Plane Shear - cam and pin locks; lag screw
6 Wall-to-Roof: Uplift Connection - cam and pin locks; lag screw
7 Wall-to-Floor: Out of Plane Shear

Testing was performed by applying loads to failure. Table 3 below, documents the average ultimate load per connector. Results of the connection tests are contained within Appendix E.

Table 3
Connection Tests
Ultimate Failure Loads (lbs)

	Wall to Wall			Wall to Roof						Wall to Floor
	Config. 1	Config. 2	Config. 3	Config. 4		Config. 5		Config. 6		Config. 7
	Cam-Lock	Cam-Lock	Cam-Lock	Cam-Lock	Lags	Cam-Lock	Lags	Cam-Lock	Lags	Wood Strip
Average Ultimate load per connector	1210	2345	984	1010	856	1324	881	785	1809	1160

DOWEL BEARING STRENGTH

VARIABLE	VALUE	UNITS	DEFINITION
Screw #	14		
L	1.5	in.	Total Screw Length
t_{skin}	26		Skin Gauge
t_{angle}	18		Angle Gauge
	SPF		Wood Type
D	0.196	in.	Diameter. D = Dr for reduced body fasteners (see Table L3)
F_{em}	3350	psi	Dowel Bearing Strength of Main Member
F_{es}	61850	psi	Dowel Bearing Strength of Side Member
K_{ϑ}	1	N/A	1+(0.25*(ϑ/90)) where ϑ is maximum angle to grain for any member
t_{skin}	0.0179		Skin Thickness
t_{angle}	0.0474		Angle Thickness
T	0.484		Tapered Length
L_m	0.9507	in.	Bearing Length in Main Member, $L - t_{skin} - t_{angle} - T$
L_s	0.0474	in.	Dowel Bearing Length in Side Member
F_{yb}	70000	psi	Dowel Bending Yield Strength
R_e	0.05416		Fem/Fes
R_t	20.05696		Lm/Ls

k_1	0.448262403	$\dfrac{\sqrt{R_e + 2R_e^2(1 + R_t + R_t^2) + R_t^2 R_e^3} - R_e(1 + R_t)}{(1 + R_e)}$
k_2	0.662695038	$-1 + \sqrt{2(1 + R_e) + \dfrac{2F_{yb}(1 + 2R_e)D^2}{3F_{em} l_m^2}}$
k_3	21.98259487	$-1 + \sqrt{\dfrac{2(1 + R_e)}{R_e} + \dfrac{2F_{yb}(2 + R_e)D^2}{3F_{em} l_s^2}}$

REDUCTION TERM, R_d (0.25<=D<=1)			
R_{d1}	4		*Reduction Term for I_m and I_s*
R_{d2}	3.6		*Reduction Term for II*
R_{d3}	3.2		*Reduction Term for III_s, III_m, and IV*
REDUCTION TERM, R_d (D<0.25)			
R_d	2.46		*Reduction Term for all Yield Modes*

YIELD LIMIT STATES			
YEILD MODE	*SHEAR (LBF)*	*FAILURE MECHANISM*	*EQUATION*
I_m	253.75	*Bearing Failure of Main Member*	$\frac{Dl_mF_{em}}{R_d}$
I_s	233.58	*Bearing Failure of Side Member*	$\frac{Dl_sF_{es}}{R_d}$
II	104.71	*Fastener Rotation*	$\frac{k_1Dl_sF_{es}}{R_d}$
III_m	159.52	*Double Bolt Bends*	$\frac{k_2Dl_mF_{em}}{(1+R_e)R_d}$
III_s	135.39	*Double Bolt Bends*	$\frac{k_3Dl_sF_{em}}{(2+R_e)R_d}$
IV	190.17	*Two Double Bolt Bends*	$\frac{D^2}{R_d}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}}$

REFERENCE DESIGN VALUE (LBF)
104.7