PRCTI20210031



City of Puyallup Building APPROVED
See permit for additional requirements.
JMontgomery 06/10/2022 3:05:02 PM

THE APPROVED CONSTRUCTION PLANS, DOCUMENTS AND ALL ENGINEERING MUST BE POSTED ON THE JOB AT ALL INSPECTIONS IN A VISIBLE AND READILY ACCESSIBLE LOCATION.

FULL SIZED LEDGIBLE COLOR PLANS ARE REQUIRED TO BE PROVIDED BY THE PERMITEE ON SITE FOR INSPECTION

Approval of submitted plans is not an approval of omissions or oversights by this office or noncompliance with any applicable regulations of local government. The contractor is responsible for making sure that the building complies with all applicable codes and regulations of the local government.

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

| | |
|---|---|
| Building | Planning |
| Engineering | Public Works |
| Fire | Traffic |

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

# COSTCO #660

# WALK-INS
# PUYALLUP, WASHINGTON
# TGE PROJECT NUMBER: 22-19079

DOUGLAS DEAN HARDIN
STATE OF WASHINGTON
20105231
REGISTERED
PROFESSIONAL ENGINEER
expires 6-09-23
04/21/2022

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<br>Boise, Idaho 83705 |
| Date: | 4/21/2022 |
| Firm Registration Number: | 603490470 |
| TGE Engineer of Record: | Douglas D. Hardin, P.E. |
| Project Manager: | Ruchin Khadka, E.I. |
| Direct Phone: | (208) 779-4321 |
| Office Phone: | (208) 345-8941 |
| Office Fax: | (208) 345-8946 |
| Email: | ruchin.khadka@tamarackgrove.com |

**Project Client Information:**

| | |
| --- | --- |
| Company: | KPS Global |
| Project Number: | C18769 |
| Contact: | Glenn Shuping |
| Address: | 4201 N. Beach St.<br>Fort Worth, TX 76137 |
| Phone: | (682) 317-5357 |
| Email: | Glenn.Shuping@kpsglobal.com |
| Client Logo: | KPS GLOBAL |

**Project Site Information:**

| | |
| --- | --- |
| Name: | Costco #660 |
| Address: | 201 39th Ave. SW<br>Puyallup, Washington 98373 |
| Client Reference Number: | C18769 |

**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.

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## 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 STRUCTURTAL 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.

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# 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:**

| | | | Dead Load Calculation | |
|---|---|---|---|---|
| $DL_{panel}$ = | 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 | | | |

**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 | $F_v$ = | 1.862 | Table 11.4-2 |
| SDC = | D | Sec. 11.6 | $S_{MS}$ = | 1.522 | $S_{MS} = F_a * S_S$, Equation 11.4-1 |
| $S_S$ = | 1.268 | Per ATC Hazards | $S_{M1}$ = | 0.816 | $S_{M1} = F_v * S_1$, Equation 11.4-2 |
| $S_1$ = | 0.438 | Per ATC Hazards | $S_{DS}$ = | 1.014 | $S_{DS} = 2/3 * S_{MS}$, Equation 11.4-3 |
| $T_L$ = | 6 | Per ATC Hazards | $S_{D1}$ = | 0.544 | $S_{D1} = 2/3 * S_{M1}$, Equation 11.4-4 |

**Snow Design Information Per ASCE 7:**

***Not Applicable***

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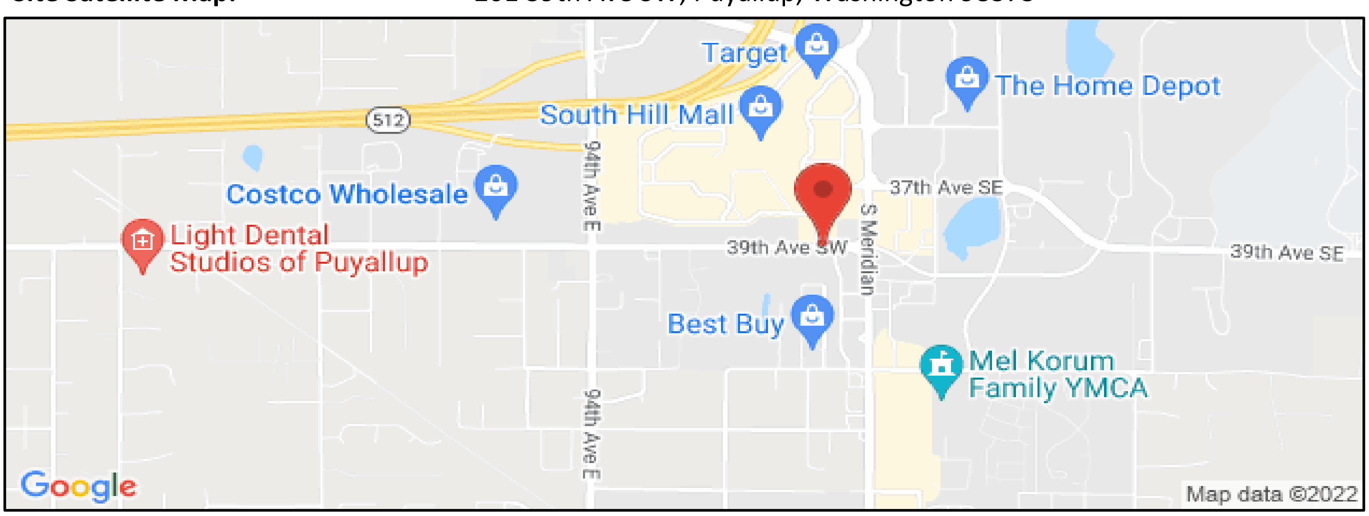
| | |
|---|---|
| Building | Planning |
| Engineering | Public Works |
| Fire | Traffic |

**Wind Design Information:**

***Not Applicable***

**Site Satellite Map:** 201 39th Ave SW, Puyallup, Washington 98373



# STRUCTURAL CALCULATIONS CBX

## JURISDICTION INFORMATION

JURISDICTION: PULLAYUP, WASINGTON
STRUCTURAL CODE: 2018 INTERNATIONAL BUILDING CODE

PASS = 1.0 FAILURE = 0

## DESIGN CRITERIA

LOAD DESIGN VALUES:

| | |
|---|---|
| $DL_{panel} := 4.5\ psf$ | Panel Dead Load |
| $LL_{panel} := 10\ psf$ | Panel Live Load - Not Accessible |
| $LL_{panel\_2} := 20\ psf$ | Panel Live Load - Accessible |
| $P_{LL} := 300\ lbf$ | Maintenance Worker Live Load |
| $P_{internal} := 5\ psf$ | Minimum Transverse Load (ASCE 7 1.4.5) |

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

## ASD LOAD COMBINATIONS (ASCE 7-16)

| | |
|---|---|
| $LC_3 := DL_{panel} + LL_{panel} = 14.5\ psf$ | Load Combination 3: D+(Lr, S, or R) |
| $LC_{3\_acc} := DL_{panel} + LL_{panel\_2} = 24.5\ psf$ | Load Combination 3: D+(Lr, S, or R) |

## WALK-IN DESIGN CRITERIA

| | |
|---|---|
| $Width := 41.54\ ft$ | Unit Width |
| $Length := 71.67\ ft$ | Unit Length |
| $H := 10.46\ ft$ | 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.

## NON-ACCESSIBLE CEILING PANEL ANALYSIS

$L := 17.85\ ft$ — Ceiling Panel Span

$L_{all} := 22.75\ ft$ — Allowable Span (Per LARR/Testing Report)

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

LOADS:

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

$w_{design\_ceiling} := LL_{panel} \cdot T_{width\_panel} = 39.2\ plf$ — Distributed Live Load

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

$$m_{max} := \max\left(\frac{w_{design\_ceiling} \cdot L^2}{8}, \frac{P_{LL} \cdot L}{4}\right) = 1561.25\ ft \cdot lbf$$ Maximum Moment

$w_{all} := 10\ psf \cdot T_{width\_panel} = 39.2\ plf$ — Allowable Panel Load (Per LARR/Testing Report)

$$M_{allow} := \frac{w_{all} \cdot {L_{all}}^2}{8} = 2536.06\ ft \cdot lbf$$ Allowable Moment

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

***SUMMARY: USE SPECIFIED CEILING PANELS PER PLANS.***

## CEILING SUPPORT BEAMS AND COLUMNS

Material Specification
BEAM W18x35 <LONG> — ASTM A992

$L := 41.47\ ft$ — Design Length

$T_{width} := \frac{35.88\ ft}{2} = 17.94\ ft$ — Tributary Width of Ceiling on Beam

LOADS:

$w_{DL} := DL_{panel} \cdot T_{width} = 80.73\ plf$ — Distributed Dead Load on Beam

$w_{LL} := LL_{panel} \cdot T_{width} = 179.4\ plf$ — Distributed Live Load on Beam

REACTIONS (ENERCALC):

$R_{DL\_1} := 2400\ lbf$ — Dead Load Reaction on Beam

$R_{LL\_1} := 3720\ lbf$ — Live Load Reaction on Beam

$\delta_{DL} := 1.334\ in - 0.810\ in = 0.524\ in$ — Deflection due to Dead Load

BEAM STRAPS ANALYSIS:

$d := 18\ in$ — Beam Depth

$M_{max} := 63.444\ kip \cdot ft$ — Maximum Moment (ENERCALC)

LOADS:

$P_{axial} := \frac{M_{max}}{d} = 42296\ lbf$ — Maximum Axial Force

$P_{design\_brace} := 0.02 \cdot P_{axial} = 845.92\ lbf$ — 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:

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

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

TENSILE RUPTURE:

$F_u := 45\ ksi$ Tensile Strength

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

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

$\Omega := 2.00$ ASD Adjustment Factor

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

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

BEARING STRENGTH:

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

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

BEAM FASTENER CAPACITY:

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

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

CEILING FASTENER CAPACITY:

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

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

REQUIRED NUMBER OF BRACES:

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

$n_{reqd} := \text{ceil}\left(\frac{P_{design\_brace}}{P_{all}}\right) = 12$ Minimum Number of Braces Required

***NOTE: PROVIDE BRACES AT EACH END AND EVENLY SPACED.***

COLUMN SUPPORT:

SHAPE: HSS 5X5X3/16

$H_{col} := H_w = 10.06\ ft$ — Height of Column

LOADS:

$R_{COL\_DL} := R_{DL\_1} = 2400\ lbf$ — Dead Load

$R_{COL\_LL} := R_{LL\_1} = 3720\ lbf$ — Live Load

COLUMN REACTIONS:

$R_{DL\_BASE} := 2520\ lbf$ — Dead Load Reaction (Enercalc)

$R_{LL\_BASE} := 3720\ lbf$ — Live Load Reaction (Enercalc)

BASE PLATE THICKNESS:

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

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

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

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

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

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

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

$F_y := 36\ ksi$ — Base Plate Yield Strength

$\Omega := 1.67$ — ASD Factor

$t_{min} := l \cdot \sqrt{\dfrac{2 \cdot \Omega \cdot \left(R_{DL\_BASE} + R_{LL\_BASE}\right)}{F_y \cdot B \cdot N}} = 0.26\ in$ — Minimum Thickness of Plate

$t_{actual} := 0.5\ in$ — Actual Thickness of Base Plate

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

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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

$P_u := 1.2 \cdot R_{DL\_BASE} + 1.6 \cdot R_{LL\_BASE} = 8976\ lbf$ — Factored Load on Slab

*CHECK* $v_n \geq P_u = 1$

***SUMMARY: USE SPECIFIED STRUCTURAL SUPPORT PER PLANS.***

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WOOD COLUMN SUPPORT (DETAIL 19)

Material Specification

(3) 2X6 — Spruce-Pine-Fir

$H_{column} := 10.06\ ft$ — Height of a Column

*LOADS:*

$P_{column\_DL} := R_{DL\_1} = 2400\ lbf$ — Dead Load on Column

$P_{column\_LL} := R_{LL\_1} = 3720\ lbf$ — Live Load on Column

NOTE: SEE ENERCALC FOR FULL ANALYSIS.

REACTIONS (ENERCALC):

$R_{DL\_base} := 2445\ lbf$ — Dead Load at Base

$R_{LL\_base} := 3720\ lbf$ — Live Load at Base

COMPRESSION STRENGTH OF WOOD BASE:

$l_b := 3 \cdot 1.5\ in = 4.5\ in$ — Length of Column Base

$w_b := 5\ in$ — Width of Column Base

$A_{col} := l_b \cdot w_b = 22.5\ in^2$ — Area At Column Base

$f_c := 425\ psi$ — Compression Strength of Spruce-Pine-Fir No. 2 Perpendicular to Grain (NDS)

$C_M := 1$ — Wet Service Factor

$C_t := 1$ — Temperature Factor

$C_i := 1$ — Incising Factor

$C_b := \frac{l_b + 0.375\ in}{l_b} = 1.08$ — Bearing Area Factor

$F'_c := f_c \cdot C_M \cdot C_t \cdot C_i \cdot C_b = 460.42\ psi$ — Allowable Compressive Strength

$F_{plate} := \frac{R_{DL\_base} + R_{LL\_base}}{A_{col}} = 274\ psi$ — Compressive Force Acting on Wood Base (ASD)

CHECK: $F'_c > F_{plate} = 1$

***<u>SUMMARY: THE GRAVITY FRAMES PER PLANS ARE SUFFICIENT TO SUPPORT THE IMPOSED LOADS.</u>***

Material Specification
BEAM W18x35 (MID) — ASTM A992

$L := 37.31\ ft$ — Design Length

$T_{width} := \frac{35.88\ ft}{2} = 17.94\ ft$ — Tributary Width of Ceiling on Beam

LOADS:

$w_{DL} := DL_{panel} \cdot T_{width} = 80.73\ plf$ — Distributed Dead Load on Beam

$w_{LL} := LL_{panel} \cdot T_{width} = 179.4\ plf$ — Distributed Live Load on Beam

REACTIONS (ENERCALC):

$R_{DL\_1} := 2159\ lbf$ — Dead Load Reaction on Beam

$R_{LL\_1} := 3347\ lbf$ — Live Load Reaction on Beam

$\delta_{DL} := 0.874\ in - 0.531\ in = 0.343\ in$ — Deflection due to Dead Load

BEAM STRAPS ANALYSIS:

$d := 18\ in$ — Beam Depth

$M_{max} := 51.354\ kip \cdot ft$ — Maximum Moment (ENERCALC)

LOADS:

$P_{axial} := \frac{M_{max}}{d} = 34236\ lbf$ — Maximum Axial Force

$P_{design\_brace} := 0.02 \cdot P_{axial} = 684.72\ lbf$ — 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:

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

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

TENSILE RUPTURE:

$F_u := 45\ ksi$ Tensile Strength

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

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

$\Omega := 2.00$ ASD Adjustment Factor

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

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

BEARING STRENGTH:

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

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

BEAM FASTENER CAPACITY:

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

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

CEILING FASTENER CAPACITY:

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

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

REQUIRED NUMBER OF BRACES:

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

$n_{reqd} := ceil\left(\frac{P_{design\_brace}}{P_{all}}\right) = 10$ Minimum Number of Braces Required

***NOTE: PROVIDE BRACES AT EACH END AND EVENLY SPACED.***

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BEAM W18X35 ANALYSIS BRIDGE BEAM:

$L := 29.67\ ft$ — Design Length

LOADS:

$R_{DL\_1} = 2159\ lbf$ — Dead Load Reaction on Beam

$R_{LL\_1} = 3347\ lbf$ — Live Load Reaction on Beam

$a_1 := 5.54\ ft$ $a_2 := 23.48\ ft$ — Locations of W12x40 Reactions

$H_{PW} := 3.94\ ft$ — Height of Parefet wall

$R_{PW} := DL_{panel} \cdot H_{PW} = 17.73\ plf$ — Point Dead Load on Beam Parefet wall

REACTIONS (ENERCALC):

$R_{DL\_2} := 2989\ lbf$ — Dead Load Reaction on Column

$R_{LL\_2} := 3420\ lbf$ — Live Load Reaction on Column

$\delta_{DL} := 0.458\ in - 0.240\ in = 0.218\ in$ — Deflection due to Dead Load

PARAPET WALL TO BRIDGE BEAM CONNECTION (#14 TEK SCREWS):

$T_{top\_header\_height} := \frac{H_{PW}}{2} = 1.97\ ft$ — Tributary height of panel acting on screws

$S_{screw} := 48\ in$ — Spacing of Tek Screws

$P_{lat} := T_{top\_header\_height} \cdot P_{internal} \cdot S_{screw} = 39.4\ lbf$ — Lateral force acting on conduit bracing

#14 TEK SCREW:

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

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

*CHECK* $P_{lat} \le V_{all\_screw} = 1$ $P_{lat} \le T_{all\_screw} = 1$

***SUMMARY: USE SPECIFIED STRUCTURAL SUPPORT PER PLANS.***

Material Specification
BEAM W18x35 (CORNER) ASTM A992

$L := 4.67\ ft$ Design Length

$T_{width} := \frac{12.95\ ft}{2} = 6.48\ ft$ Tributary Width of Ceiling on Beam

LOADS:

$w_{DL} := DL_{panel} \cdot T_{width} = 29.14\ plf$ Distributed Dead Load on Beam

$w_{LL} := LL_{panel} \cdot T_{width} = 64.75\ plf$ Distributed Live Load on Beam

REACTIONS (ENERCALC):

$R_{DL\_3} := 150\ lbf$ Dead Load Reaction on Beam

$R_{LL\_3} := 151\ lbf$ Live Load Reaction on Beam

$\delta_{DL} := 0\ in - 0\ in = 0\ in$ Deflection due to Dead Load

***<u>SUMMARY: USE SPECIFIED STRUCTURAL SUPPORT PER PLANS.</u>***

Material Specification
HSS 5x5x3/16"(SP-2) A500 GRADE B

$H_{column} := H_w + 18\ in = 11.56\ ft$ Height of a Column

LOADS:

$p_{column\_DL} := R_{DL\_2} + R_{DL\_3} = 3139\ lbf$ Dead Load on Column

$p_{column\_LL} := R_{LL\_2} + R_{LL\_3} = 3571\ lbf$ Live Load on Column

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

COLUMN REACTIONS (ENERCALC):

$R_{DL\_base} := 3277\ lbf$ Dead Load at Base Plate

$R_{LL\_base} := 3571\ lbf$ 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

$t_{min} := l \cdot \sqrt{\frac{2 \cdot \Omega \cdot (R_{DL\_base} + R_{LL\_base})}{F_y \cdot B \cdot N}} = 0.26\ in$ Minimum Thickness of Plate

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

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

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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 |
| $P_u := 1.2 \cdot R_{DL\_base} + 1.6 \cdot R_{LL\_base} = 9646\ lbf$ | Factored Load on Slab |

*CHECK* $v_n \geq P_u = 1$

***<u>SUMMARY: USE SPECIFIED STRUCTURAL SUPPORT PER PLANS.</u>***

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## SUSPENDED PARAPET WALL ALL THREAD ANALYSIS

$H_p := 3.94\ ft$ — Heigth of Parapet Wall

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

LOADS:

$P_{DL} := DL_{panel} \cdot H_p \cdot S_{all\_thread} = 70.92\ lbf$ — 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*

$P_{DL} = 70.92\ lbf$ — Total Load on All Threaded Rod

*CHECK* $P_{DL} \le T_{all} = 1$

***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 HEADER

HEADER PANEL CALCULATIONS :

| | |
|---|---|
| $L_h := 5\ ft$ | Length of Header |
| $D_h := H_w - 7.08\ ft = 2.98\ ft$ | Depth/Height of Header |
| $R := \frac{L_h}{D_h} = 1.68$ | Header Aspect Ratio |
| $LC_3 := DL_{panel} + LL_{panel} = 14.5\ psf$ | Load Combination 3: D+(Lr, S, or R) |
| $v_{allow} := 222\ plf$ | Allowable Shear (Per LARR/Testing Report) |
| $T_{width} := \frac{17.89\ ft}{2} = 8.95\ ft$ | Tributary Width Acting on Header |
| $w_{design} := T_{width} \cdot LC_3 + DL_{panel} \cdot D_h = 143.11\ plf$ | Load Applied to Header |

HEADER PANEL CAPACITY:

| | |
|---|---|
| $w_{allow\_1} := \frac{8 \cdot v_{allow} \cdot D_h}{L_h} = 1058.5\ plf$ | Allowable Distributed Load due to Bending ( $M := v_{allow} \cdot D_h \cdot L_h = \frac{w_{allow\_1} \cdot L_h^2}{8}$ ) |
| $w_{allow\_2} := \frac{v_{allow} \cdot 2 \cdot D_h}{L_h} = 264.62\ plf$ | Allowable Distributed Load due to Shear ( $V := v_{allow} \cdot D_h = \frac{w_{allow\_2} \cdot L_h}{2}$ ) |

CAM-LOCK CONNECTION CAPACITY:

| | |
|---|---|
| $FOS := 3$ | Factor of Safety |
| $V_{all\_cam} := \frac{1210}{FOS}\ lbf = 403.33\ lbf$ | Allowable In-Plane Shear on Camlock (Per LARR/Testing Report) |
| $n_{cam} := 2$ | Minimum Number of Camlocks per Support |
| $w_{allow\_3} := \frac{2 \cdot (V_{all\_cam} \cdot n_{cam})}{L_h} = 322.67\ plf$ | Allowable Distributed Load ( $V_{conn} := V_{all\_cam} \cdot n_{cam} = \frac{w_{allow\_2} \cdot L_h}{2}$ ) |
| $w_{allow} := min\left(w_{allow\_1}, w_{allow\_2}, w_{allow\_3}\right) = 264.62\ plf$ | Allowable Distributed Load |

CHECK: $w_{allow} \geq w_{design} = 1$

***SUMMARY: THE ALLOWABLE DISTRIBUTED LOAD IS GREATER THAN THE IMPOSED DISTRIBUTED LOAD. THEREFORE, THE HEADER PANELS ARE ACCEPTABLE FOR TYPICAL LOAD-BEARING OPENING.***

## WALL PANEL ANALYSIS

$H_w = 10.06\ ft$ — Design Height

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

$T_{width\_wall} := \dfrac{17.94\ ft}{2} = 8.97\ ft$ — Tributary Width of Ceiling Panel Acting on Wall

AXIAL LOADS:

$w_{design\_ceiling} = 39.2\ plf$ — Total Axial Load From Ceiling

$v_{max} := \max\left(w_{design\_ceiling} \cdot T_{width\_wall}, P_{LL}\right) = 351.62\ lbf$ — Governing Live Load

$p_{design\_wall} := \dfrac{v_{max}}{T_{width\_panel}} + DL_{panel} \cdot T_{width\_wall} = 130.07\ plf$ — Ceiling Panel Total Axial Load

$H_{all\_axial} := 26\ ft$ — Allowable Height for Axial Load (Per LARR/Testing Report)

$P_{all\_axial} := 1037\ plf$ — Allowable Axial Load (Per LARR/Testing Report)

TRANSVERSE LOADS:

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

$m_{max} := \dfrac{w_{wall} \cdot H_w^{\ 2}}{8} = 247.95\ ft \cdot lbf$ — Maximum Moment

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

$P_{all\_trans} := 46.3\ psf \cdot T_{width\_panel} = 181.5\ plf$ — Allowable Transverse Load (Per LARR/Testing Report)

$M_{allow} := \dfrac{P_{all\_trans} \cdot H_{all\_trans}^{\ 2}}{8} = 3544.84\ ft \cdot lbf$ — Allowable Moment

$P_{comb} := \dfrac{p_{design\_wall}}{P_{all\_axial}} + \dfrac{m_{max}}{M_{allow}} = 0.2$ — 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.***

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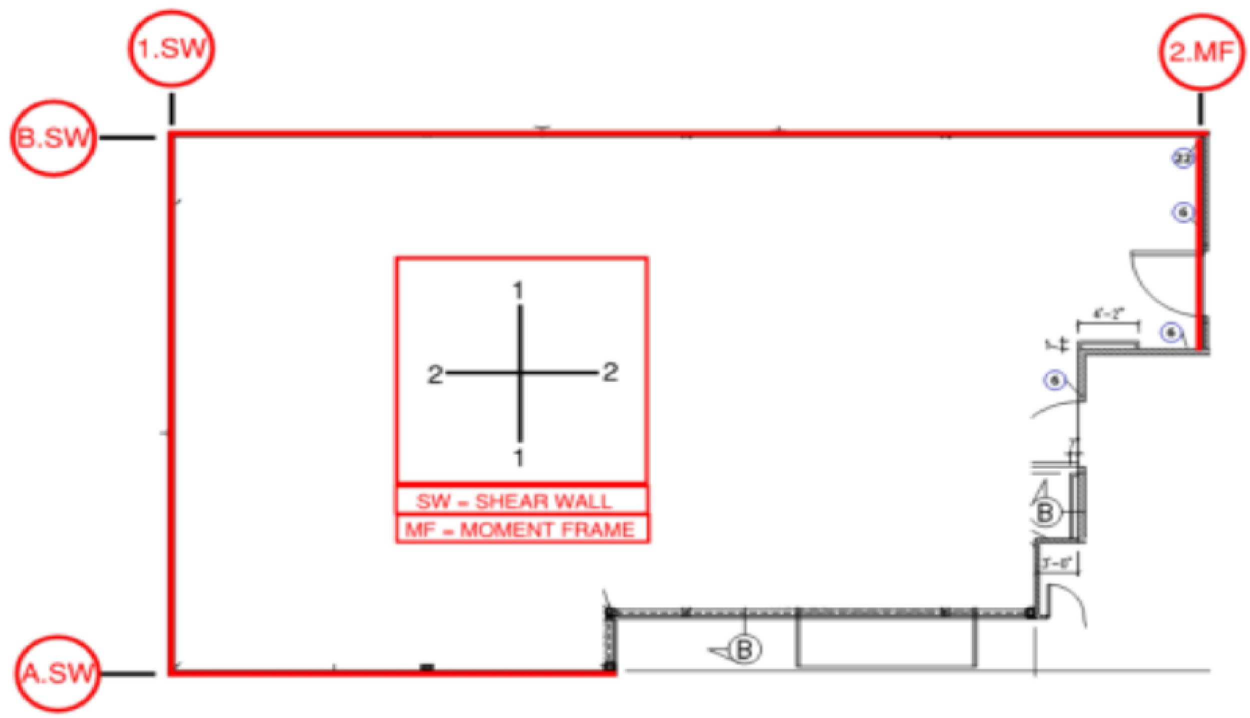
## LATERAL ANALYSIS

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

$S_s := 1.268$ Mapped Spectral Response Acceleration Parameter at Short Periods

$S_1 := 0.438$ Mapped Spectral Response Acceleration Parameter at a Period of 1 s

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

$S_{D1} := 0.544$ 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

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

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

$$T_s := \frac{S_{D1}}{S_{DS}} = 0.54$$

DESIGN CRITERIA :

$h_n := \frac{H_w}{ft} = 10.06$ Height of Structure

$A_{ceiling} := 2616.06\ ft^2$ Total Area of Ceiling

$L_{wall} := 2 \cdot Width + 2 \cdot Length = 226.42\ ft$ Total Length of Walls

$W_{18x35} := (41.47\ ft + 2 \cdot 37.31\ ft + 29.67\ ft + 4.67\ ft) \cdot 35\ plf = 5265.05\ lbf$ Weight of W18x35 Beam

$Wt_{wood\_col} := 3 \cdot 0.5 \cdot H_w \cdot 26.22\ pcf \cdot 24.75\ in^2 = 68\ lbf$ Weight of 3-2x6 Wooden Columnn

$Wt_{col} := 4 \cdot 0.5 \cdot H_w \cdot 11.97\ plf = 240.84\ lbf$ Weight of HSS Columnn

$Wt_{MF\_STEEL} := 16.35\ ft \cdot 36\ plf + 2 \cdot 0.5 \cdot H_w \cdot 45\ plf = 1041.3\ lbf$ Weight of Moment Frame

$Wt_{steel} := W_{18x35} + Wt_{wood\_col} + Wt_{col} + Wt_{MF\_STEEL} = 6615.19\ lbf$ Total Support Weight

$Wt_{coil} := 0\ lbf$ Total Coil Weight

LATERAL FORCE GENERATION:

$$Wt := (A_{ceiling} \cdot DL_{panel}) + \left(\frac{H_w}{2} \cdot L_{wall} \cdot DL_{panel}\right) + Wt_{coil} + Wt_{steel} = 23512.48\ lbf$$ Effective Seismic Weight

$T_a := C_t \cdot h_n{}^x = 0.113$ Approximate Fundamental Period

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

$S_{DS\_max} := \mathbf{if}\ (T_a \le 0.5, 1, 0.7 \cdot S_{DS}) = 1$ Max Sds Value in determination of Cs and Ev (12.8.1.3)

$S_{DS} := min\ (S_{DS\_max}, S_{DS}) = 1$ 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)

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

$$C_{s\_max} := \mathbf{if}\left(T_a \le T_L, 1.5 \cdot \frac{S_{D1}}{T_a \cdot \left(\frac{R_p}{I_e}\right)}, 1.5 \cdot \frac{S_{D1} \cdot T_L}{T_a{}^2 \cdot \left(\frac{R_p}{I_e}\right)}\right) = 3.61$$ Maximum Coefficient

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

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

$V_p := C_s \cdot Wt = 11756.24 \; lbf$ — Seismic Base Shear

$V_{p\_asd} := 0.7 \cdot V_p = 8229.37 \; lbf$ — ASD Seismic Base Shear

$w_{design\_1} := \frac{V_{p\_asd}}{Length} = 114.82 \; plf$ — Distributed Design Load (1-1 Direction)

$w_{design\_2} := \frac{V_{p\_asd}}{Width} = 198.11 \; plf$ — Distributed Design Load (2-2 Direction)

DIAPHRAGM CHECK ( 1-1 ):

$Width_1 := Width = 41.54 \; ft$ — Width of Diaphragm ( 1-1 )

$Length_1 := Length = 71.67 \; ft$ — Length of Diaphragm ( 1-1 )

$R_1 := \frac{Length_1}{Width_1} = 1.73$ — Aspect Ratio (1-1)

$F_{all\_1} := 215 \; plf$ — Allowable Diaphragm Capacity (Per LARR/Testing Report)

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

3/8" LAG BOLT:

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

$V := \frac{w_{design\_1} \cdot Length_1}{2} = 4114.68 \; lbf$ — Max Shear at Diaphram Edge

$S_{lag} := 24 \; in$ — Spacing of Lag bolt

$N_{lag} := \text{ceil}\left(\frac{Width_1}{S_{lag}}\right) = 21$ — Number of Lag Bolt Connecting panels

$V_{all\_bolt} := C_D \cdot \frac{p}{8 \cdot D} \cdot 180 \; lbf = 144 \; lbf$ — Allowable Shear of Lag Bolt Parallel to Grain (NDS)

$V_{all\_inplane\_lag} := N_{lag} \cdot V_{all\_bolt} = 3024 \; lbf$ — Total In-Plane Shear on Lagbolt

CHECK: $V_{all\_inplane\_lag} \geq \frac{V}{2} = 1$

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CHORD FORCE:

$$F_{chord\_1} := \frac{\frac{w_{design\_1} \cdot Length_1^2}{8}}{Width_1} = 1774.79\ lbf$$

Chord Force

$$f_{chord\_1\_1} := \frac{F_{chord\_1}}{0.5 \cdot Length_1} = 49.53\ plf$$

Chord Force on 2-2 Shear Walls

DIAPHRAGM CHECK ( 2-2 ):

$$Width_2 := Width = 41.54\ ft$$

Width of Diaphragm ( 2-2 )

$$Length_2 := Length = 71.67\ ft$$

Length of Diaphragm ( 2-2 )

$$R_2 := \frac{Width_2}{Length_2} = 0.58$$

Aspect Ratio (2-2)

$$F_{all\_2} := 596\ plf$$

Allowable Diaphragm Capacity
(Per LARR/Testing Report)

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

CAM-LOCK:

$$V := \frac{w_{design\_2} \cdot Width_2}{2} = 4114.68\ lbf$$

Max Shear at Diaphram Edge

$$N_{cam} := ceil\left(\frac{Length_2 - 2\ ft}{48\ in} + 1\right) = 19$$

Number of Camlocks Connecting panels

$$FOS := 3$$

Factor of Safety

$$V_{all\_inplane} := N_{cam} \cdot \frac{1210}{FOS}\ lbf = 7663.33\ lbf$$

Allowable In-Plane Shear on Camlock
(Per LARR/Testing Report)

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

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CHORD FORCE:

$$F_{chord\_2} := \frac{\frac{w_{design\_2} \cdot Width_2^2}{8}}{Length_2} = 596.22\ lbf$$ Max Chord Force

$$f_{chord\_2\_1} := \frac{F_{chord\_2}}{0.5 \cdot Width_2} = 28.71\ plf$$ Chord Force on 1-1 Shear Walls

SHEAR WALL CALCULATIONS:

$L_1 := Width = 41.54\ ft$ Length of Wall 1

$T_{width\_1} := \frac{Length}{2} = 35.84\ ft$ Tributary Width

$$f_1 := \frac{w_{design\_1} \cdot T_{width\_1}}{L_1} = 99.05\ plf$$ In-Plane Force on Wall 1

$L_A := 31\ ft$ Length of Existing Wall A

$T_{width\_A} := \frac{Width}{2} = 20.77\ ft$ Tributary Width

$$f_A := \frac{w_{design\_2} \cdot T_{width\_A}}{L_A} = 132.73\ plf$$ In-Plane Force on Wall A

$$R := \frac{H_w}{L_A} = 0.32$$ Worst Case Shape Ratio

$F_{all\_inplane} := 0.5 \cdot 646\ plf = 323\ plf$ Half of Allowable In-Plane Shear (Per LARR/ Testing Report) for existing wall

CHECK: $F_{all\_inplane} \geq f_A = 1$

$L_B := Length = 71.67\ ft$ Length of Wall B

$T_{width\_B} := \frac{Width}{2} = 20.77\ ft$ Tributary Width

$$f_B := \frac{w_{design\_2} \cdot T_{width\_B}}{L_B} = 57.41\ plf$$ In-Plane Force on Wall B

$$R := \frac{H_w}{min\left(L_1, L_B\right)} = 0.24$$ Worst Case Shape Ratio

$F_{all\_inplane} := 646\ plf$ Allowable In-Plane Shear (Per LARR/Testing Report)

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

SHEAR LOAD FOR MOMENT FRAME CALCULATIONS:

$L_{2\_MF} := 16.35\ ft$ Length of Frame 2

$T_{width\_2} := \frac{Length}{2} = 35.84\ ft$ Tributary Width

$$f_{2\_MF} := \frac{w_{design\_1} \cdot T_{width\_2}}{L_{2\_MF}} = 251.66\ plf$$ In-Plane Force on Frame 2

***SUMMARY: USE SPECIFIED WALL PANELS PER PLANS.***

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## WIDE FLANGE MOMENT FRAME DESIGN (GRIDLINE B)

DESIGN PARAMETERS:

$\varphi := 0.75$ — LRFD Reduction Factor

$L_{MF} := L_{2\_MF} = 16.35\ ft$ — Length of Moment Frame

$f_{MF} := \dfrac{f_{2\_MF}}{0.7} = 359.5\ plf$ — Distributed Load on Moment Frame

$T_{width} := \dfrac{17.89\ ft}{2} = 8.95\ ft$ — Tributary Width of Ceiling Panel on Moment Frame

$f_{B\_DL} := DL_{panel} \cdot T_{width} = 40.25\ plf$ — Dead Load of Ceiling Panel on Moment Frame

$f_{B\_LL} := LL_{panel} \cdot T_{width} = 89.45\ plf$ — Live Load of Ceiling Panel on Moment Frame

Material Specification

Columns: W10X45 — ASTM A992

Beams: W16X36 — ASTM A992

MOMENT FRAME TO CEILING CONNECTION

#14 TEK SCREW:

$S_{screw} := 3\ in$ — Spacing of Screws

$n_{screw} := 1$ — Spacing of Screws

$V_{all\_screw} := \dfrac{n_{screw} \cdot 76\ lbf}{S_{screw}} = 364.8\ plf$ — Allowable Shear on Screw (ESR-1976)

$L_{conn} := 16.35\ ft$ — Length of the Connection

$f_{m\_dist} := \dfrac{f_{MF} \cdot L_{MF}}{L_{conn}} = 359.52\ plf$ — Distributed Load Across Moment Frame Connection

CHECK: $f_{m\_dist} \le V_{all\_screw} = 1$

BEAM-COLUMN MOMENT CONNECTION ANALYSIS:

*NOTE: SEE RISA CONNECTION REPORT FOR THE BEAM-MOMENT CONNECTION ANALYSIS.*

CONNECTION OF MOMENT FRAME TO FLOOR:

LOADS:

$T_{conn} := 6269\ lbf$ — Maximum Tension on Connection (Risa)

$V_{conn} := 5994\ lbf$ — Maximum Shear on Connection (Risa)

$C_{conn} := 8907\ lbf$ — Maximum Compression on Connection (Risa) (Base Plate Attachment)

*Note: See Hilti Report for Anchor Analysis*

BASE PLATE THICKNESS:

$b_f := 8.02\ in$ $d_f := 10.1\ in$ Column Dimensions

$\lambda := 1$ Normal Weight Concrete Factor

$B := 8.25\ in$ $N := 13.5\ in$ Base Plate Dimensions

$n := (B - 0.8 \cdot b_f) \cdot 0.5 = 0.92\ in$

$m := (N - 0.95 \cdot d_f) \cdot 0.5 = 1.95\ in$

$n' := \frac{\sqrt{b_f \cdot d_f}}{4} = 2.25\ in$ Yield-Line Theory Cantilever Distance From Column Web or Column Flange

$l := \max(m, n, \lambda \cdot n') = 2.25\ in$

$F_y := 36\ ksi$ Base Plate Yield Strength

$\varphi := 0.9$ LRFD Factor

$t_{min} := l \cdot \sqrt{\frac{2 \cdot C_{conn}}{\varphi \cdot F_y \cdot B \cdot N}} = 0.16\ in$ Minimum Thickness of Plate

$t_{actual} := 0.5\ in$ 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 |
| $B = 8.25\ in$ | Base Plate Length | |
| $N = 13.5\ in$ | Base Plate Width | |
| $b_f = 8.02\ in$ | Column Dimension along B | |
| $d_f = 10.1\ in$ | Column Dimension along N | |
| $b := b_f + \frac{B - b_f}{2} = 8.14\ in$ | Equivalent Loaded Length | |
| $c := d_f + \frac{N - d_f}{2} = 11.8\ in$ | Equivalent Loaded Width | |
| $\beta := \frac{\max(b, c)}{\min(b, c)} = 1.45$ | | 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 |
| $b_0 := 2 \cdot (b + d) + 2 \cdot (c + d) = 51.87\ in$ | | Effective Perimeter around Baseplate |
| $\varphi := 0.75$ | | LRFD Shear factor |

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

$$v_2 := \left(2 + \frac{4}{\beta}\right) \cdot \lambda \cdot \sqrt{f'_c}\ psi = 260.59\ psi$$

$$v_3 := \left(2 + \frac{\alpha_s \cdot d}{b_0}\right) \cdot \lambda \cdot \sqrt{f'_c}\ psi = 172.9\ psi$$

$v_n := \min(v_1, v_2, v_3) \cdot \varphi \cdot b_0 \cdot d = 20178.92\ lbf$ — Two-way Shear Strength of Slab

$P_u := 1.2 \cdot R_{DL\_base} + 1.6 \cdot R_{LL\_base} = 9646\ lbf$ — Factored Load on Slab

*CHECK* $v_n \geq P_u = 1$

***SUMMARY: USE SPECIFIED STRUCTURAL SUPPORT PER PLANS.***

STORY DRIFT CHECK:

$h := H_w = 10.06\ ft$ — Height of Column

$C_d := 3$ — Deflection Amplification Factor (Table 12.8-15)

$\delta_{xe} := 0.456\ in$ — Max Seismic Load Deflection (RISA)

$\Delta_{x\_seismic} := \frac{C_d \cdot \delta_{xe}}{I_e} = 1.37\ in$ — Story Drift of Entire Structure (12.8.6)

$\Delta_{limit\_seismic} := 0.025 \cdot h = 3.02\ in$ — Allowable Story Drift (Table 12.12-1)

Check: $\Delta_{limit\_seismic} \geq \Delta_{x\_seismic} = 1$

DEFLECTION:

$\delta_{allow\_beam} := \frac{L_{MF}}{180} = 1.09\ in$ — Allowable Beam Deflection

$\delta_{beam} := 0.028\ in$ — Actual Beam Deflection (RISA)

Check: $\delta_{allow\_beam} \geq \delta_{beam} = 1$

***SUMMARY: THE STORY DRIFT OF THE FRAME , IS WITHIN THE ALLOWABLE DISPLACEMENT LIMITS PER CODE, THEREFORE THE STRUCTURE IS ACCEPTABLE TO RESIST IMPOSED LOADS AND DEFLECTIONS.***

PRCTI20210031



## CEILING PANEL TO WALL PANEL CONNECTION (DETAIL 1 )

$H_w = 10.06\ ft$ — Design Height

LOADS:

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

$p_{trans} := P_{internal} \cdot \frac{H_w}{2} = 25.15\ plf$ — Transverse Shear Force on Wall-Ceiling Connection

$f_{inplane} := \max(f_1, f_B) = 99.05\ plf$ — In-Plane Shear Force on Wall-Ceiling Connection

$f_{max} := \max(p_{trans}, f_{inplane}) = 99.05\ plf$ — Governing Shear Force on Wall-Ceiling Connection

3/8" LAG BOLT:

$D := 0.375\ in$ — Lag Bolt Diameter

$S_{bolt} := 24\ in$ — Spacing of Bolts

$p := 1.5\ in$ — Penetration Length of Screw

$C_D := 1.6$ — Load Duration Factor

$$V_{all\_bolt\_par} := \frac{C_D \cdot \frac{p}{8 \cdot D} \cdot 180\ lbf}{S_{bolt}} = 72\ plf$$

Allowable Shear on Bolt - Parallel to Grain SPF No.2 (NDS)

$$V_{all\_bolt\_perp} := \frac{C_D \cdot \frac{p}{8 \cdot D} \cdot 110\ lbf}{S_{bolt}} = 44\ plf$$

Allowable Shear on Bolt - Perp. to Grain SPF No.2 (NDS)

#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*

$$\frac{p_{trans}}{V_{all\_bolt\_par} + T_{all\_screw}} + \frac{f_{max}}{V_{all\_bolt\_par} + V_{all\_screw}} \le 1 = 1$$

***SUMMARY: THE MAXIMUM FORCE ON THE CONNECTIONS IS LESS THAN THE ALLOWABLE FORCE. THEREFORE, THE ROOF-WALL CONNECTIONS ARE ACCEPTABLE FOR RESISTING IMPOSED LOADS.***

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| | |
|---|---|
| Building | Planning |
| Engineering | Public Works |
| Fire | Traffic |

## CEILING PANEL TO WALL PANEL CONNECTION (DETAIL 6)

$H_w = 10.06\ ft$ — Design Height

$T_{width} := \frac{17.89\ ft}{2} = 8.95\ ft$ — Tributary Width of Ceiling on Connection

LOADS:

$LC_3 := DL_{panel} + LL_{panel} = 14.5\ psf$ — Load Combination 3: D+(Lr, S, or R)

$p_{grav} := LC_3 \cdot T_{width} = 129.7\ plf$ — Force on Wall-Ceiling Connection

$f_{max} := p_{trans} = 25.15\ plf$ — Governing Shear Force on Wall-Ceiling Connection

#14 TEK SCREW:

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

$C_D := 1.6$ — Load Duration Factor

$S_{screw} := 6\ in$ — Spacing of #14 Tek Screw Through Angle

$V_{all\_screw} := \frac{C_D \cdot 104.7\ lbf}{S_{screw}} = 335.04\ plf$ — Allowable Shear of #14 TEK (Dowel Bearing Strength - NDS)

$T_{all\_screw} := \frac{C_D \cdot 0.95\ in \cdot 121\ \frac{lbf}{in}}{S_{screw}} = 367.84\ plf$ — Allowable Tension of #14 TEK (NDS)

$R_{combined} := \frac{p_{trans}}{T_{all\_screw}} + \frac{p_{grav}}{V_{all\_screw}} = 0.46$ — Combined Stress Ratio for Connection

*CHECK* $R_{combined} \le 1.0 = 1$

***<u>SUMMARY: THE MAXIMUM FORCE ON THE CONNECTION IS LESS THAN THE ALLOWABLE FORCE. THUS, CEILING-WALL CONNECTION IS ACCEPTABLE FOR RESISTING IMPOSED LOADS.</u>***

## CEILING PANEL TO WALL PANEL CONNECTION (DETAIL 7)

$H_w = 10.06\ ft$ — Design Height

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

LOADS:

$LC_3 := DL_{panel} + LL_{panel} = 14.5\ psf$ — Load Combination 3: D+(Lr, S, or R)

$p_{grav} := LC_3 \cdot T_{width} = 56.84\ plf$ — Force on Wall-Ceiling Connection

$f_{inplane} := f_A = 132.73\ plf$ — In-Plane Shear Force on Wall-Ceiling Connection

$f_{inplane\_res} := \sqrt{{p_{grav}}^2 + {f_{inplane}}^2} = 144.39\ plf$ — Resultant Shear on Tek Screws

$f_{max} := \max\left(p_{trans}, f_{inplane\_res}\right) = 144.39\ plf$ — Governing Shear Force on Wall-Ceiling Connection

#14 TEK SCREW:

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

$C_D := 1.6$ — Load Duration Factor

$S_{screw} := 3\ in$ — Spacing of #14 Tek Screw Through Angle

$V_{all\_screw} := \frac{76\ lbf}{S_{screw}} = 304\ plf$ — Allowable Shear of #14 TEK (ESR-1976)

$T_{all\_screw} := \frac{57\ lbf}{S_{screw}} = 228\ plf$ — Allowable Tension of #14 TEK (ESR-1976)

$R_{combined} := \frac{p_{trans}}{T_{all\_screw}} + \frac{\sqrt{{p_{grav}}^2 + {f_{inplane}}^2}}{V_{all\_screw}} = 0.59$ — Combined Stress Ratio for Connection

*CHECK* $R_{combined} \le 1.0 = 1$

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

## WALL TO FLOOR CONNECTION (DETAIL 2)

$H_w = 10.06\ ft$ — Design Height

LOADS:

$p_{trans} := P_{internal} \cdot \frac{H_w}{2} = 25.15\ plf$ — Transverse Shear Force on Floor-Wall Connection

$f_{inplane} := \max(f_1, f_B) = 99.05\ plf$ — In-Plane Shear Force on Wall-Ceiling Connection

$f_{max} := \max(p_{trans}, f_{inplane}) = 99.05\ plf$ — 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)

1/4" HILTI KH-EZ:

$S_{anchor} := 12\ in$ — Spacing of Anchor Through Angle

$\Omega := 2.0$ — Overstrength Factor

$V_{anchor} := \frac{\Omega \cdot f_{max} \cdot S_{anchor}}{0.7} = 283.01\ lbf$ — LRFD Shear Force on Anchor

$V_{all\_anchor} := 750\ lbf$ — Allowable Shear on Anchor

*NOTE: SEE HILTI 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

$T_{width\_ceiling} := \frac{17.89\ ft}{2} = 8.95\ ft$ — Tributary Width of Ceiling

$H_w = 10.06\ ft$ — Height of Wall Panel

LOADS:

$f := f_1 = 99.05\ plf$ — In-Plane Force on Wall

$L := L_1 = 41.54\ ft$ — Length of Wall

$Wt_{wall} := (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{panel} \cdot H_w \cdot L = 961.15\ lbf$ — Weight of Wall

$Wt_{ceiling} := (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 854.62\ lbf$ — Weight of Ceiling

$w_R := \frac{Wt_{wall} + Wt_{ceiling}}{L} = 43.71\ plf$ — Weight Resisting Overturning

$M_{wall} := f \cdot L \cdot H_w - w_R \cdot \frac{L^2}{2} = 3680.05\ lbf \cdot ft$ — Overturning Moment Acting on Wall

$w := \frac{3 \cdot M_{wall}}{L^2} = 6.4\ plf$ — Maximum Value of Overturning Force at End
$\left( M_{wall} := \frac{1}{2} \cdot w \cdot L \cdot \frac{2\ L}{3} = 3680.05\ lbf \cdot ft \right)$

***NOTE: THE WALL TO FLOOR CONTINIOUS ANGLE CONNECTION IS ADEQUATE TO RESIST THE NOMINAL OVERTURNING FORCE.***

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# SOFTWARE PRINTOUTS (ENERCALC)

## Steel Beam

Project File: 22-19079.ec6

LIC# : KW-06013705, Build:20.22.3.31 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2022

**DESCRIPTION:** LONG W18x35 Beam

### CODE REFERENCES

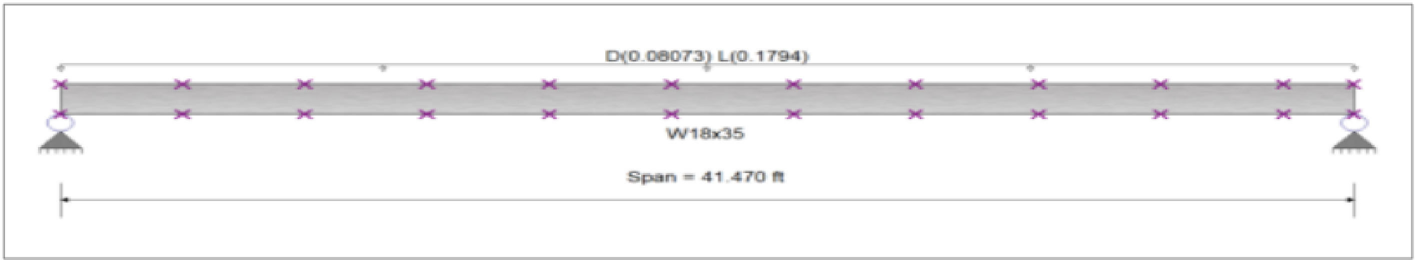
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

### Material Properties

| | | | |
|---|---|---|---|
| Analysis Method : | Allowable Strength Design | Fy : Steel Yield : | 50.0 ksi |
| Beam Bracing : | Beam bracing is defined as a set spacing over all spans | E: Modulus : | 29,000.0 ksi |
| Bending Axis : | Major Axis Bending | | |

### Unbraced Lengths

First Brace starts at ft from Left-Most support
Regular spacing of lateral supports on length of beam = 3.920 ft




### Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.00450, L = 0.010 ksf, Tributary Width = 17.940 ft, (Panel Load)

### DESIGN SUMMARY

**Design OK**

| | | | |
|---|---|---|---|
| Maximum Bending Stress Ratio = | 0.382 : 1 | Maximum Shear Stress Ratio = | 0.058 : 1 |
| Section used for this span | W18x35 | Section used for this span | W18x35 |
| Ma : Applied | 63.444 k-ft | Va : Applied | 6.120 k |
| Mn / Omega : Allowable | 165.918 k-ft | Vn/Omega : Allowable | 106.20 k |
| Load Combination | +D+L | Load Combination | +D+L |
| | | Location of maximum on span | 41.470 ft |
| Span # where maximum occurs | Span # 1 | Span # where maximum occurs | Span # 1 |

**Maximum Deflection**

| | | | | | |
|---|---|---|---|---|---|
| Max Downward Transient Deflection | 0.810 in | Ratio = | 613 | >=240. | |
| Max Upward Transient Deflection | 0.000 in | Ratio = | 0 | <240.0 | Span: 1 : L Only |
| Max Downward Total Deflection | 1.334 in | Ratio = | 373 | >=180 | Span: 1 : +D+L |
| Max Upward Total Deflection | 0.000 in | Ratio = | 0 | <180 | |

### Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| +D+L | 1 | 1.3340 | 20.853 | | 0.0000 | 0.000 |

### Vertical Reactions

Support notation : Far left is #  Values in KIPS

| Load Combination | Support 1 | Support 2 |
|---|---|---|
| Overall MAXimum | 6.120 | 6.120 |
| Overall MINimum | 1.440 | 1.440 |
| D Only | 2.400 | 2.400 |
| +D+L | 6.120 | 6.120 |
| +D+0.750L | 5.190 | 5.190 |
| +0.60D | 1.440 | 1.440 |
| L Only | 3.720 | 3.720 |

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| | |
|---|---|
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| Engineering | Public Works |
| Fire | Traffic |

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## Steel Column

Project File: 22-19079.ec6

LIC# : KW-06013705, Build:20.22.3.31 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2022

**DESCRIPTION:** HSS5x5x3/16 Column

### Code References

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

### 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 10.060 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 = 10.060 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 10.060 ft, K = 1.0

### Applied Loads

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

Column self weight included : 120.418 lbs * Dead Load Factor
AXIAL LOADS . . .
Reaction from Beam: Axial Load at 10.060 ft, D = 2.40, L = 3.720 k

### DESIGN SUMMARY

**Bending & Shear Check Results**

| | | | |
|---|---|---|---|
| PASS | Max. Axial+Bending Stress Ratio = | 0.08915 | : 1 |
| | Load Combination | +D+L | |
| | Location of max.above base | 0.0 | ft |
| | At maximum location values are . . . | | |
| | Pa : Axial | 6.240 | k |
| | Pn / Omega : Allowabl | 69.999 | 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 :

### Maximum Reactions

Note: Only non-zero reactions are listed.

| Load Combination | Axial Reaction @ Base | X-X Axis Reaction k @ Base | X-X Axis Reaction @ Top | Y-Y Axis Reaction @ Base | Y-Y Axis Reaction @ Top | Mx - End Moments k-ft @ Base | Mx - End Moments @ Top | My - End Moments @ Base | My - End Moments @ Top |
|---|---|---|---|---|---|---|---|---|---|
| D Only | 2.520 | | | | | | | | |
| +D+L | 6.240 | | | | | | | | |
| +D+0.750L | 5.310 | | | | | | | | |
| +0.60D | 1.512 | | | | | | | | |
| L Only | 3.720 | | | | | | | | |

### Extreme Reactions

| Item | Extreme Value | Axial Reaction @ Base | X-X Axis Reaction k @ Base | X-X Axis Reaction @ Top | Y-Y Axis Reaction @ Base | Y-Y Axis Reaction @ Top | Mx - End Moments k-ft @ Base | Mx - End Moments @ Top | My - End Moments @ Base | My - End Moments @ Top |
|---|---|---|---|---|---|---|---|---|---|---|
| Axial @ Base | Maximum | 6.240 | | | | | | | | |
| " | Minimum | 1.512 | | | | | | | | |
| Reaction, X-X Axis Base | Maximum | 2.520 | | | | | | | | |
| " | Minimum | 2.520 | | | | | | | | |
| Reaction, Y-Y Axis Base | Maximum | 2.520 | | | | | | | | |
| " | Minimum | 2.520 | | | | | | | | |
| Reaction, X-X Axis Top | Maximum | 2.520 | | | | | | | | |
| " | Minimum | 2.520 | | | | | | | | |
| Reaction, Y-Y Axis Top | Maximum | 2.520 | | | | | | | | |
| " | Minimum | 2.520 | | | | | | | | |
| Moment, X-X Axis Base | Maximum | 2.520 | | | | | | | | |

## Steel Column

Project File: 22-19079.ec6

LIC# : KW-06013705, Build:20.22.3.31 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2022

**DESCRIPTION:** HSS5x5x3/16 Column

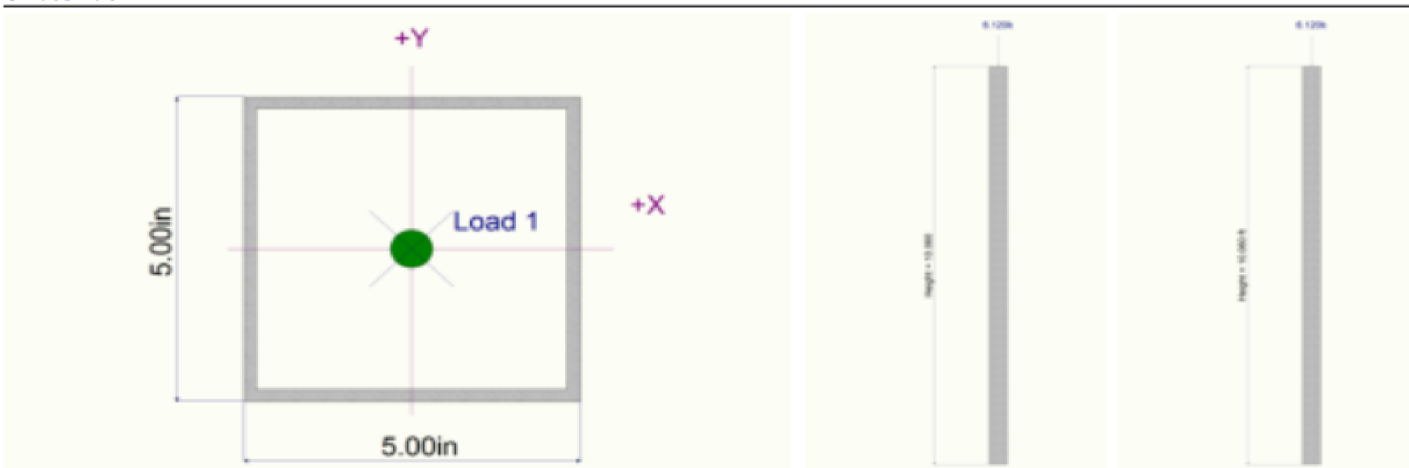
### Extreme Reactions

| 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 |
|---|---|---|---|---|---|---|---|---|---|---|
| " | Minimum | 2.520 | | | | | | | | |
| Moment, Y-Y Axis Base | Maximum | 2.520 | | | | | | | | |
| " | Minimum | 2.520 | | | | | | | | |
| Moment, X-X Axis Top | Maximum | 2.520 | | | | | | | | |
| " | Minimum | 2.520 | | | | | | | | |
| Moment, Y-Y Axis Top | Maximum | 2.520 | | | | | | | | |
| " | Minimum | 2.520 | | | | | | | | |

### 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 |

### Sketches



PRCTI20210031

| Structural Engineering Calculations | TAMARACK GROVE ENGINEERING | Project Name: | Costco #660 |
|---|---|---|---|
| | | Location: | Puyallup, WA |
| | | Job Number: | 22-19079 |

## Wood Column

Project File: 22-19079.ec6

LIC# : KW-06013705, Build:20.22.3.31 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2022

**DESCRIPTION:** (3) 2x6 (UNIT 4.1)

### *Code References*

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

### General Information

| | | | |
|---|---|---|---|
| Analysis Method | Allowable Stress Design | Wood Section Name | **3-2x6** |
| End Fixities | Top & Bottom Pinned | Wood Grading/Manuf. | Graded Lumber |
| Overall Column Height | 10.06 ft | Wood Member Type | Sawn |

*( Used for non-slender calculations )*

| | | | |
|---|---|---|---|
| Wood Species | Spruce-Pine-Fir | | |
| Wood Grade | No. 1/No. 2 | | |
| Fb + | 875.0 psi | Fv | 135.0 psi |
| Fb - | 875.0 psi | Ft | 450.0 psi |
| Fc - Prll | 1,150.0 psi | Density | 26.220 pcf |
| Fc - Perp | 425.0 psi | | |

| E : Modulus of Elasticity . . . | x-x Bending | y-y Bending | Axial |
|---|---|---|---|
| Basic | 1,400.0 | 1,400.0 | 1,400.0 ksi |
| Minimum | 510.0 | 510.0 | |

| | |
|---|---|
| Exact Width | **4.50** in |
| Exact Depth | **5.50** in |
| Area | 24.750 in^2 |
| Ix | 62.391 in^4 |
| Iy | 41.766 in^4 |

| Allow Stress Modification Factors | |
|---|---|
| Cf or Cv for Bending | 1.30 |
| Cf or Cv for Compressio | 1.10 |
| Cf or Cv for Tension | 1.30 |
| Cm : Wet Use Factor | 1.0 |
| Ct : Temperature Fact | 1.0 |
| Cfu : Flat Use Factor | 1.0 |
| Kf : Built-up columns | 1.0 *NDS 15.:* |
| Use Cr : Repetitive ? | No |

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for buckling ABOUT Y-Y Axis = 1(
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 1(

### Applied Loads

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

Column self weight included : 45.336 lbs * Dead Load Factor
AXIAL LOADS . . .
DL and LL rxn from beam: Axial Load at 10.060 ft, D = 2.40, L = 3.720 k

### *DESIGN SUMMARY*

Bending & Shear Check Results

| | | |
|---|---|---|
| PASS | Max. Axial+Bending Stress Ratio = | **0.4859 : 1** |
| | Load Combination | +D+L |
| | Governing NDS Forumla | Comp Only, fc/Fc' |
| | Location of max.above base | 0.0 ft |
| | At maximum location values are . | |
| | Applied Axial | 6.165 k |
| | Applied Mx | 0.0 k-ft |
| | Applied My | 0.0 k-ft |
| | Fc : Allowable | 512.65 psi |
| PASS | Maximum Shear Stress Ratio = | **0.0 : 1** |
| | Load Combination | +0.60D |
| | Location of max.above base | 10.060 ft |
| | Applied Design Shear | 0.0 psi |
| | Allowable Shear | 216.0 psi |

**Maximum SERVICE Lateral Load Reactions . .**

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

**Maximum SERVICE Load Lateral Deflections . . .**

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

**Other Factors used to calculate allowable stresses . . .**

| Bending | Compression | Tension |
|---|---|---|

### Load Combination Results

| | | | Maximum Axial + Bending Stress Ratios | | | Maximum Shear Ratios | | |
|---|---|---|---|---|---|---|---|---|
| Load Combination | $C_D$ | $C_P$ | Stress Ratio | Status | Location | Stress Ratio | Status | Location |
| D Only | 0.900 | 0.442 | 0.1965 | PASS | 0.0 ft | 0.0 | PASS | 10.060 ft |
| +D+L | 1.000 | 0.405 | 0.4859 | PASS | 0.0 ft | 0.0 | PASS | 10.060 ft |
| +D+0.750L | 1.250 | 0.335 | 0.3997 | PASS | 0.0 ft | 0.0 | PASS | 10.060 ft |
| +0.60D | 1.600 | 0.268 | 0.1092 | PASS | 0.0 ft | 0.0 | PASS | 10.060 ft |

### Maximum Reactions

Note: Only non-zero reactions are listed.

| | X-X Axis Reaction k | | Y-Y Axis Reaction | | Axial Reaction | My - End Moments k-ft | | Mx - End Moments | |
|---|---|---|---|---|---|---|---|---|---|
| Load Combination | @ Base | @ Top | @ Base | @ Top | @ Base | @ Base | @ Top | @ Base | @ Top |
| D Only | | | | | 2.445 | | | | |
| +D+L | | | | | 6.165 | | | | |
| +D+0.750L | | | | | 5.235 | | | | |

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| | |
|---|---|
| Building | Planning |
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## Wood Column

Project File: 22-19079.ec6

LIC# : KW-06013705, Build:20.22.3.31 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2022

**DESCRIPTION:** (3) 2x6 (UNIT 4.1)

### Maximum Reactions

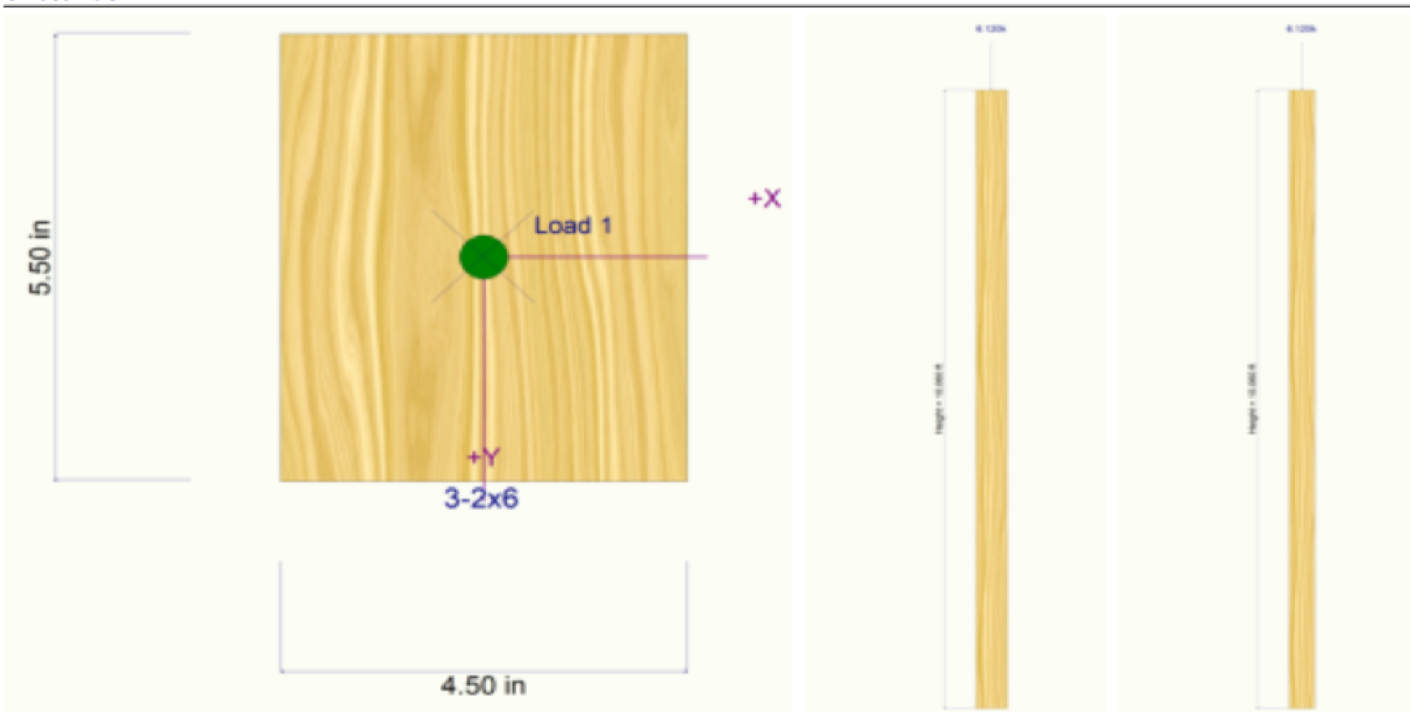
Note: Only non-zero reactions are listed.

| Load Combination | X-X Axis Reaction k @ Base | X-X Axis Reaction k @ Top | Y-Y Axis Reaction @ Base | Y-Y Axis Reaction @ Top | Axial Reaction @ Base | My - End Moments k-ft @ Base | My - End Moments k-ft @ Top | Mx - End Moments @ Base | Mx - End Moments @ Top |
|---|---|---|---|---|---|---|---|---|---|
| +0.60D | | | | | 1.467 | | | | |
| L Only | | | | | 3.720 | | | | |

### 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 |

### Sketches



## Steel Beam

Project File: 22-19079.ec6

LIC# : KW-06013705, Build:20.22.3.31 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2022

**DESCRIPTION:** MID W18x35 Beam

### CODE REFERENCES

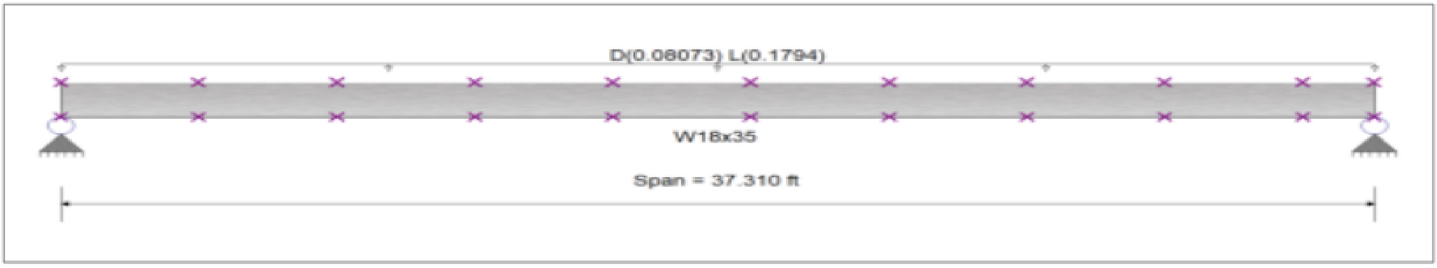
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

### Material Properties

| | | | |
|---|---|---|---|
| Analysis Method : | Allowable Strength Design | Fy : Steel Yield : | 50.0 ksi |
| Beam Bracing : | Beam bracing is defined as a set spacing over all spans | E: Modulus : | 29,000.0 ksi |
| Bending Axis : | Major Axis Bending | | |

### Unbraced Lengths

First Brace starts at ft from Left-Most support
Regular spacing of lateral supports on length of beam = 3.920 ft




### Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.00450, L = 0.010 ksf, Tributary Width = 17.940 ft, (Panel Load)

### DESIGN SUMMARY

**Design OK**

| | | | |
|---|---|---|---|
| Maximum Bending Stress Ratio = | 0.310 : 1 | Maximum Shear Stress Ratio = | 0.052 : 1 |
| Section used for this span | W18x35 | Section used for this span | W18x35 |
| Ma : Applied | 51.354 k-ft | Va : Applied | 5.506 k |
| Mn / Omega : Allowable | 165.918 k-ft | Vn/Omega : Allowable | 106.20 k |
| Load Combination | +D+L | Load Combination | +D+L |
| | | 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 | 0.531 in | Ratio = | 842 | >=240. | |
| Max Upward Transient Deflection | 0.000 in | Ratio = | 0 | <240.0 | Span: 1 : L Only |
| Max Downward Total Deflection | 0.874 in | Ratio = | 512 | >=180 | Span: 1 : +D+L |
| Max Upward Total Deflection | 0.000 in | Ratio = | 0 | <180 | |

### Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| +D+L | 1 | 0.8740 | 18.762 | | 0.0000 | 0.000 |

### Vertical Reactions

Support notation : Far left is #' Values in KIPS

| Load Combination | Support 1 | Support 2 |
|---|---|---|
| Overall MAXimum | 5.506 | 5.506 |
| Overall MINimum | 1.295 | 1.295 |
| D Only | 2.159 | 2.159 |
| +D+L | 5.506 | 5.506 |
| +D+0.750L | 4.669 | 4.669 |
| +0.60D | 1.295 | 1.295 |
| L Only | 3.347 | 3.347 |

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## Steel Beam

Project File: 22-19079.ec6

LIC# : KW-06013705, Build:20.22.3.31 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2022

**DESCRIPTION:** W18x35 Beam (BRIDGE BEAM)

### CODE REFERENCES

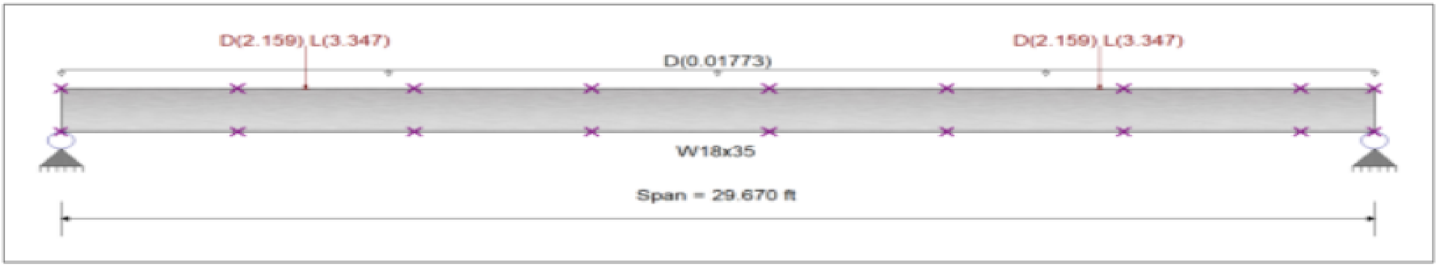
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

### Material Properties

| | | | |
|---|---|---|---|
| Analysis Method : | Allowable Strength Design | Fy : Steel Yield : | 50.0 ksi |
| Beam Bracing : | Beam bracing is defined as a set spacing over all spans | E: Modulus : | 29,000.0 ksi |
| Bending Axis : | Major Axis Bending | | |

### Unbraced Lengths

First Brace starts at 4.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 4.0 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 = 2.159, L = 3.347 k @ 5.540 ft, (Reaction from W18x35 beam)

Point Load : D = 2.159, L = 3.347 k @ 23.480 ft, (Reaction from W18x35 beam)

Uniform Load : D = 0.01773 k/ft, Tributary Width = 1.0 ft, (parafet wall)

### DESIGN SUMMARY

**Design OK**

| | | | |
|---|---|---|---|
| Maximum Bending Stress Ratio = | 0.230 : 1 | Maximum Shear Stress Ratio = | 0.060 : 1 |
| Section used for this span | **W18x35** | Section used for this span | **W18x35** |
| Ma : Applied | 38.233 k-ft | Va : Applied | 6.409 k |
| Mn / Omega : Allowable | 165.918 k-ft | Vn/Omega : Allowable | 106.20 k |
| Load Combination | +D+L | Load Combination | +D+L |
| | | 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 | 0.240 in | Ratio = | 1,481 | >=240. | |
| Max Upward Transient Deflection | 0.000 in | Ratio = | 0 | <240.0 | Span: 1 : L Only |
| Max Downward Total Deflection | 0.458 in | Ratio = | 778 | >=180 | Span: 1 : +D+L |
| Max Upward Total Deflection | 0.000 in | Ratio = | 0 | <180 | |

### Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| +D+L | 1 | 0.4577 | 14.920 | | 0.0000 | 0.000 |

### Vertical Reactions

Support notation : Far left is #' Values in KIPS

| Load Combination | Support 1 | Support 2 |
|---|---|---|
| Overall MAXimum | 6.409 | 6.168 |
| Overall MINimum | 1.793 | 1.736 |
| D Only | 2.989 | 2.894 |
| +D+L | 6.409 | 6.168 |
| +D+0.750L | 5.554 | 5.349 |
| +0.60D | 1.793 | 1.736 |
| L Only | 3.420 | 3.274 |

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## Steel Beam

Project File: 22-19079.ec6

LIC# : KW-06013705, Build:20.22.3.31 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2022

**DESCRIPTION:** CORNER W18x35 Beam

### CODE REFERENCES

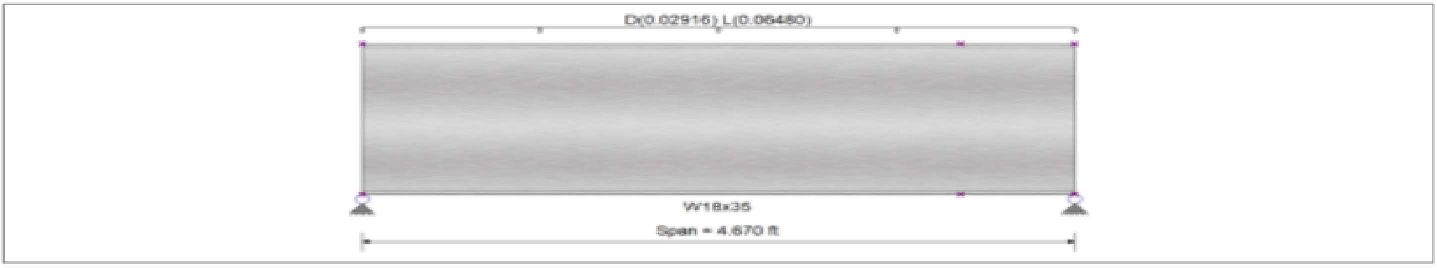
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

### Material Properties

| | | | |
|---|---|---|---|
| Analysis Method : | Allowable Strength Design | Fy : Steel Yield : | 50.0 ksi |
| Beam Bracing : | Beam bracing is defined as a set spacing over all spans | E: Modulus : | 29,000.0 ksi |
| Bending Axis : | Major Axis Bending | | |

### Unbraced Lengths

First Brace starts at ft from Left-Most support
Regular spacing of lateral supports on length of beam = 3.920 ft




### Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.00450, L = 0.010 ksf, Tributary Width = 6.480 ft, (Panel Load)

### DESIGN SUMMARY

**Design OK**

| | | | |
|---|---|---|---|
| Maximum Bending Stress Ratio = | **0.002** : 1 | Maximum Shear Stress Ratio = | **0.003** : 1 |
| Section used for this span | **W18x35** | Section used for this span | **W18x35** |
| Ma : Applied | 0.352 k-ft | Va : Applied | 0.3011 k |
| Mn / Omega : Allowable | 165.918 k-ft | Vn/Omega : Allowable | 106.20 k |
| Load Combination | +D+L | Load Combination | +D+L |
| | | Location of maximum on span | 4.670 ft |
| Span # where maximum occurs | Span # 1 | Span # where maximum occurs | Span # 1 |

**Maximum Deflection**

| | | | | | |
|---|---|---|---|---|---|
| Max Downward Transient Deflection | 0.000 in | Ratio = | 0 | <240.0 | |
| Max Upward Transient Deflection | 0.000 in | Ratio = | 0 | <240.0 | Span: 1 : L Only |
| Max Downward Total Deflection | 0.000 in | Ratio = | 597834 | >=180 | Span: 1 : +D+L |
| Max Upward Total Deflection | 0.000 in | Ratio = | 0 | <180 | |

### Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| +D+L | 1 | 0.0001 | 2.348 | | 0.0000 | 0.000 |

### Vertical Reactions

Support notation : Far left is #' Values in KIPS

| Load Combination | Support 1 | Support 2 |
|---|---|---|
| Overall MAXimum | 0.301 | 0.301 |
| Overall MINimum | 0.090 | 0.090 |
| D Only | 0.150 | 0.150 |
| +D+L | 0.301 | 0.301 |
| +D+0.750L | 0.263 | 0.263 |
| +0.60D | 0.090 | 0.090 |
| L Only | 0.151 | 0.151 |

## Steel Column

Project File: 22-19079.ec6

LIC# : KW-06013705, Build:20.22.3.31 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2022

**DESCRIPTION:** 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 11.560 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 = 11.560 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 11.560 ft, K = 1.0

### Applied Loads

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

Column self weight included : 138.373 lbs * Dead Load Factor
AXIAL LOADS . . .
Reaction from W12x22 Beam: Axial Load at 11.560 ft, D = 3.139, L = 3.571 k

### DESIGN SUMMARY

**Bending & Shear Check Results**

| | | | |
|---|---|---|---|
| PASS | Max. Axial+Bending Stress Ratio = | 0.1062 | : 1 |
| | Load Combination | +D+L | |
| | Location of max.above base | 0.0 | ft |
| | At maximum location values are . . . | | |
| | Pa : Axial | 6.848 | k |
| | Pn / Omega : Allowable | 64.503 | 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 Ratio | 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 :

### 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 | 3.277 | | | | | | | | |
| +D+L | 6.848 | | | | | | | | |
| +D+0.750L | 5.956 | | | | | | | | |
| +0.60D | 1.966 | | | | | | | | |
| L Only | 3.571 | | | | | | | | |

### 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 | 6.848 | | | | | | | | |
| " | Minimum | 1.966 | | | | | | | | |
| Reaction, X-X Axis Base | Maximum | 3.277 | | | | | | | | |
| " | Minimum | 3.277 | | | | | | | | |
| Reaction, Y-Y Axis Base | Maximum | 3.277 | | | | | | | | |
| " | Minimum | 3.277 | | | | | | | | |
| Reaction, X-X Axis Top | Maximum | 3.277 | | | | | | | | |
| " | Minimum | 3.277 | | | | | | | | |
| Reaction, Y-Y Axis Top | Maximum | 3.277 | | | | | | | | |
| " | Minimum | 3.277 | | | | | | | | |
| Moment, X-X Axis Base | Maximum | 3.277 | | | | | | | | |

## Steel Column

Project File: 22-19079.ec6

LIC# : KW-06013705, Build:20.22.3.31 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2022

**DESCRIPTION:** HSS5x5x3/16 Column

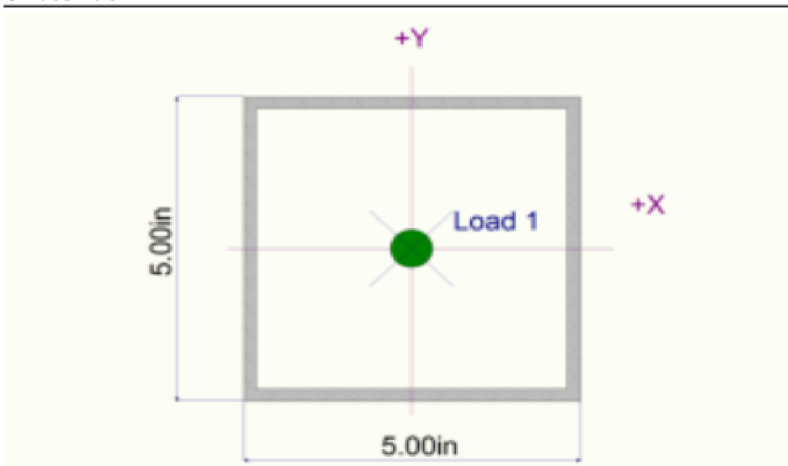
### Extreme Reactions

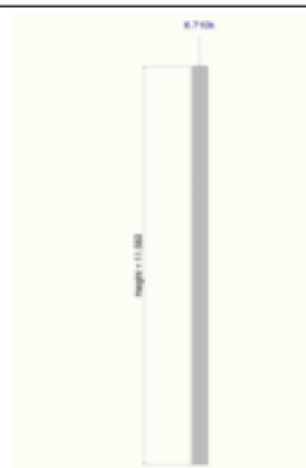
| Item | Extreme Value | Axial Reaction @ Base | X-X Axis Reaction k @ Base | X-X Axis Reaction k @ Top | Y-Y Axis Reaction @ Base | Y-Y Axis Reaction @ Top | Mx - End Moments k-ft @ Base | Mx - End Moments k-ft @ Top | My - End Moments @ Base | My - End Moments @ Top |
|---|---|---|---|---|---|---|---|---|---|---|
| " | Minimum | 3.277 | | | | | | | | |
| Moment, Y-Y Axis Base | Maximum | 3.277 | | | | | | | | |
| " | Minimum | 3.277 | | | | | | | | |
| Moment, X-X Axis Top | Maximum | 3.277 | | | | | | | | |
| " | Minimum | 3.277 | | | | | | | | |
| Moment, Y-Y Axis Top | Maximum | 3.277 | | | | | | | | |
| " | Minimum | 3.277 | | | | | | | | |

### 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 |

### Sketches

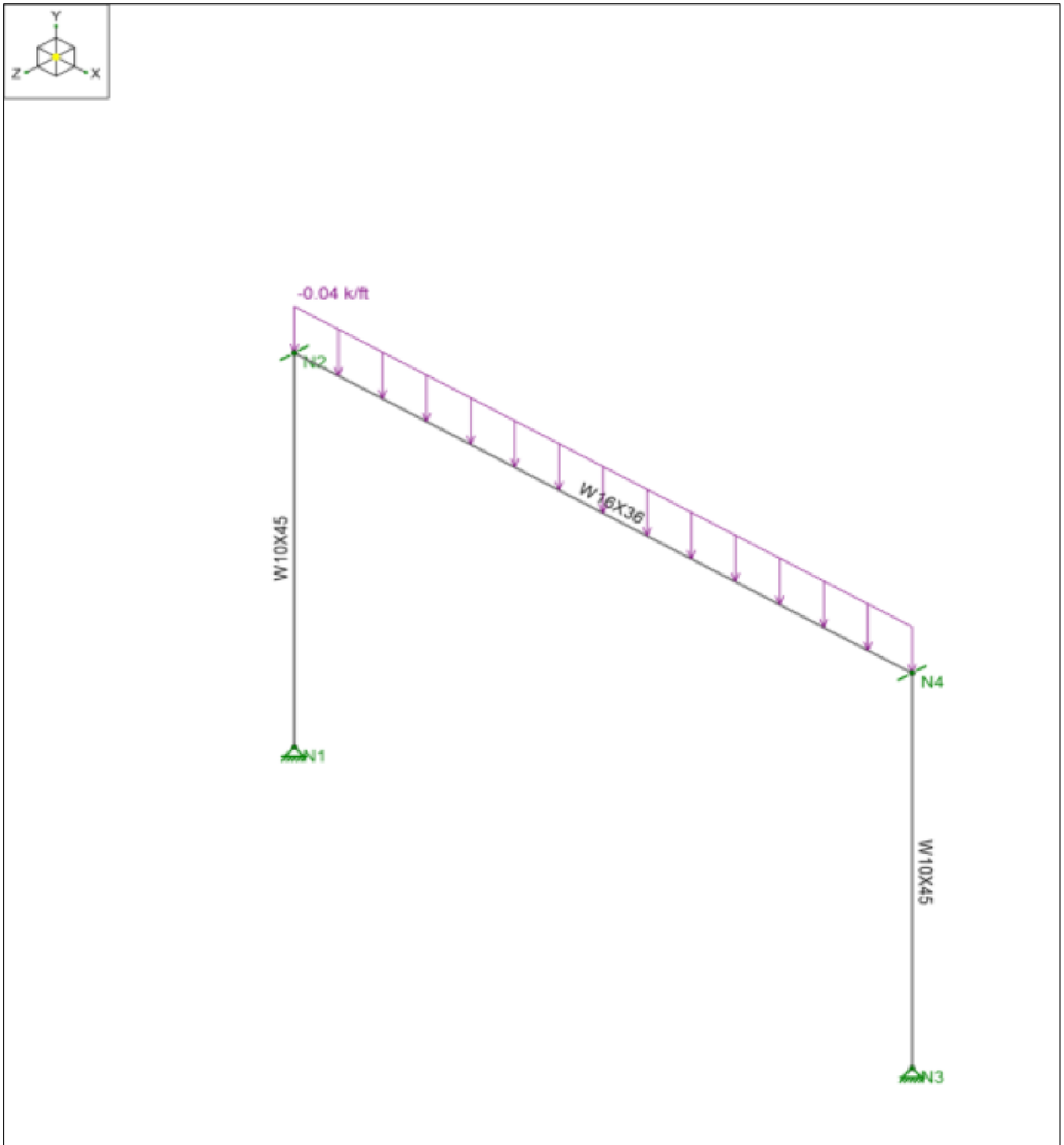


PRCTI20210031

| Structural Engineering Calculations | TAMARACK GROVE ENGINEERING | Project Name: Costco #660<br>Location: Puyallup, WA<br>Job Number: 22-19079 |
|---|---|---|

## SOFTWARE PRINTOUTS (RISA)




| | | |
|---|---|---|
| TGE | | SK-4 |
| | | Apr 20, 2022 |
| 22-19079 | | 22-19079.r3d |

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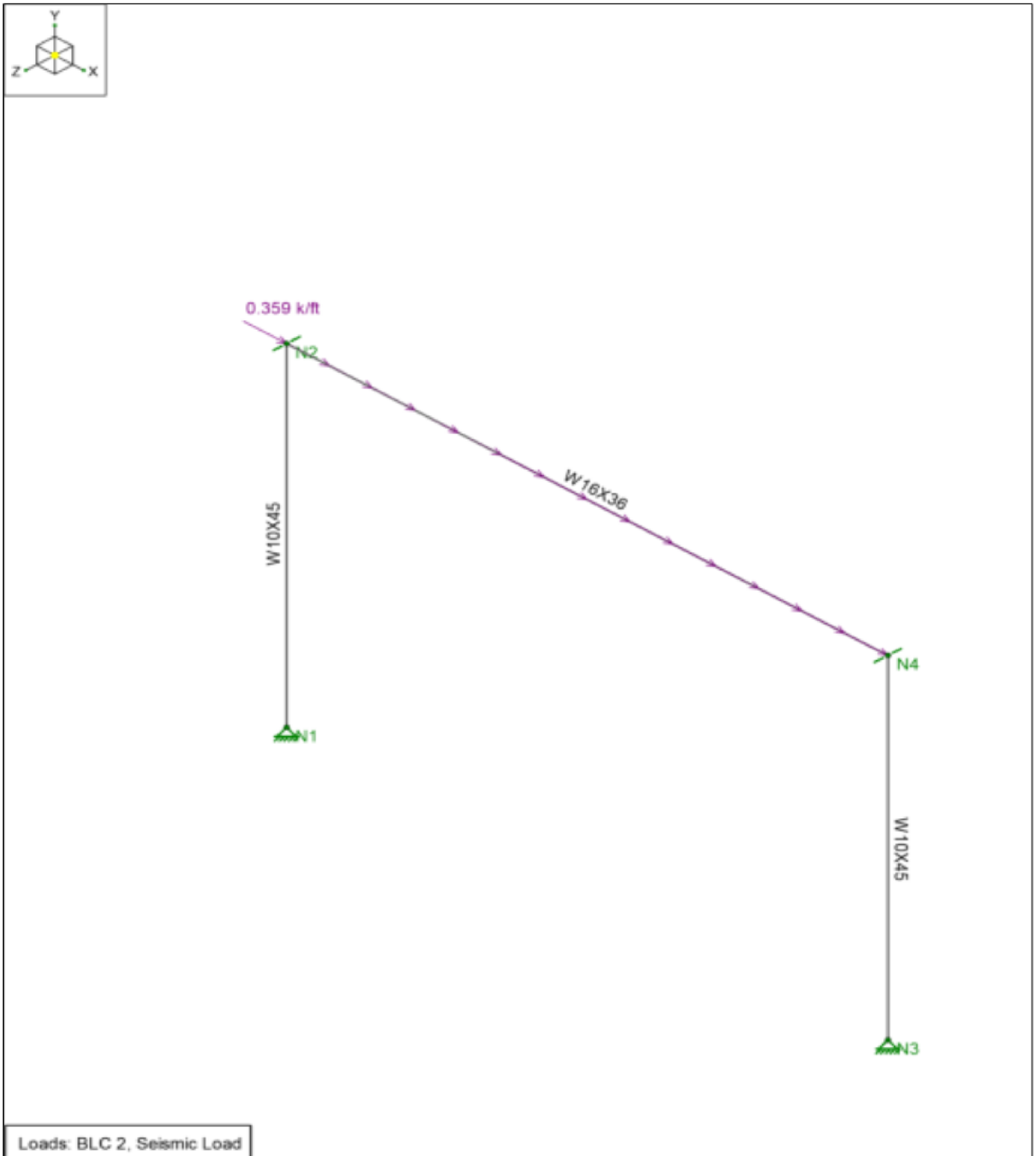
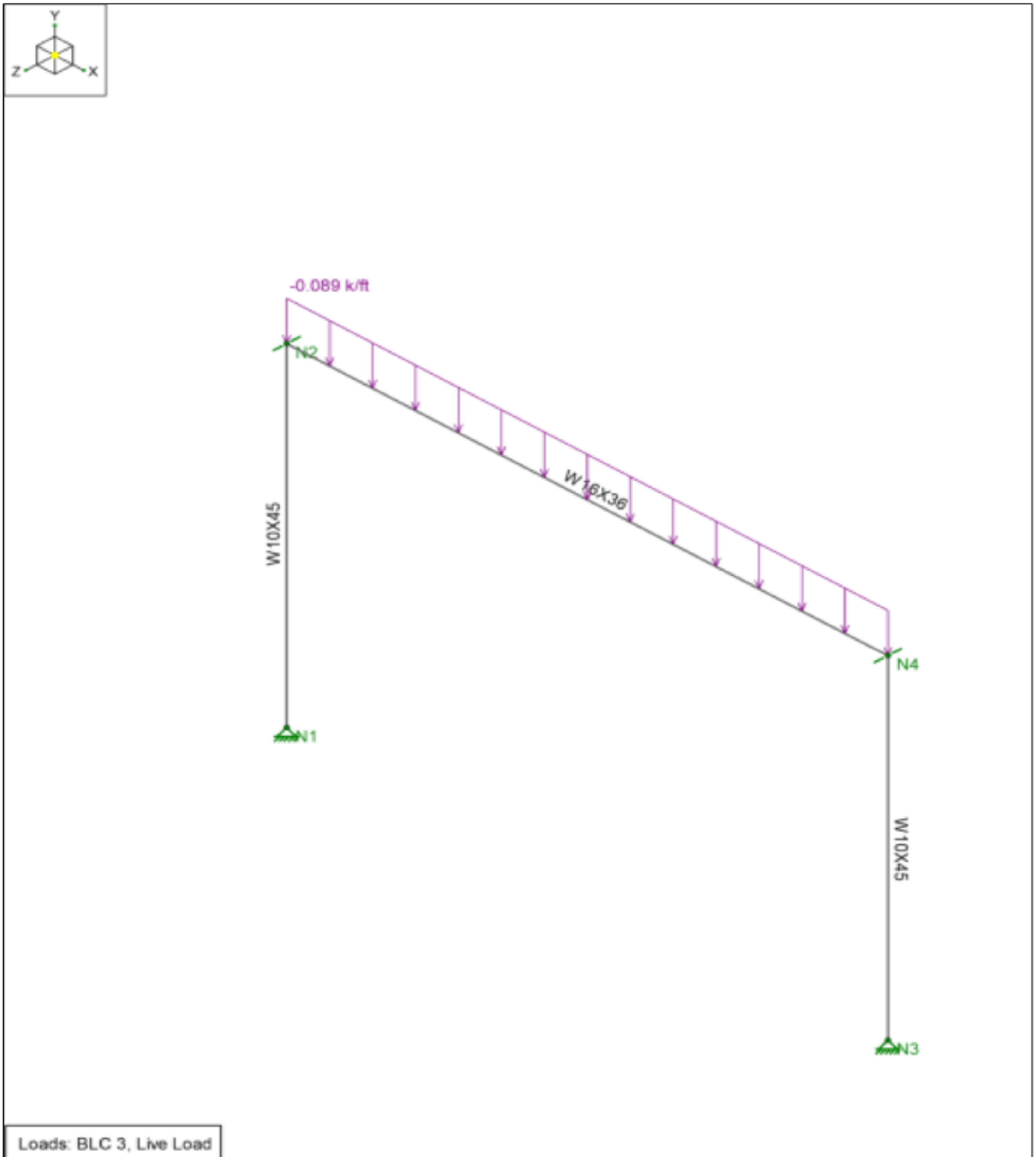



| | | |
|---|---|---|
| TGE | | SK-5 |
| | | Apr 20, 2022 |
| 22-19079 | | 22-19079.r3d |




| | | |
|---|---|---|
| TGE | | SK-6 |
| | | Apr 20, 2022 |
| 22-19079 | | 22-19079.r3d |

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|---|---|
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| Engineering | Public Works |
| Fire | Traffic |

RISA A NEMETSCHEK COMPANY

Company : TGE
Designer :
Job Number : 22-19079
Model Name :

4/20/2022
12:06:02 PM
Checked By : __________

### Hot Rolled Steel Properties

| | Label | E [ksi] | G [ksi] | Nu | Therm. Coeff. [1e⁵°F⁻¹] | Density [k/ft³] | Yield [ksi] | Ry | Fu [ksi] | Rt |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | A992 | 29000 | 11154 | 0.3 | 0.65 | 0.49 | 50 | 1.1 | 65 | 1.1 |

### Hot Rolled Steel Section Sets

| | Label | Shape | Type | Design List | Material | Design Rule | Area [in²] | Iyy [in⁴] | Izz [in⁴] | J [in⁴] |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | BEAM | W16X36 | Beam | Wide Flange | A992 | Typical | 10.6 | 24.5 | 448 | 0.545 |
| 2 | COLUMN | W10X45 | Column | Wide Flange | A992 | Typical | 13.3 | 53.4 | 248 | 1.51 |

### Hot Rolled Steel Design Parameters

| | Label | Shape | Length [ft] | Lb y-y [ft] | Lb z-z [ft] | Lcomp top [ft] | Function |
|---|---|---|---|---|---|---|---|
| 1 | M1 | COLUMN | 10.06 | | | Lbyy | Lateral |
| 2 | M3 | COLUMN | 10.06 | | | Lbyy | Lateral |
| 3 | M4 | BEAM | 16.35 | 1.96 | 13.223 | 1.96 | Lateral |

### Node Coordinates

| | Label | X [ft] | Y [ft] | Z [ft] | Detach From Diaphragm |
|---|---|---|---|---|---|
| 1 | N1 | 0 | 0 | 0 | |
| 2 | N2 | 0 | 10.06 | 0 | |
| 3 | N3 | 16.35 | 0 | 0 | |
| 4 | N4 | 16.35 | 10.06 | 0 | |

### Basic Load Cases

| | BLC Description | Category | Y Gravity | Distributed |
|---|---|---|---|---|
| 1 | DEAD LOAD | DL | -1 | 1 |
| 2 | Seismic Load | EL | | 1 |
| 3 | Live Load | LL | | 1 |

### Member Distributed Loads (BLC 1 : DEAD LOAD)

| | Member Label | Direction | Start Magnitude [k/ft, F, ksf, k-ft/ft] | End Magnitude [k/ft, F, ksf, k-ft/ft] | Start Location [(ft, %)] | End Location [(ft, %)] |
|---|---|---|---|---|---|---|
| 1 | M4 | Y | -0.04 | -0.04 | 0 | %100 |

### Member Distributed Loads (BLC 2 : Seismic Load)

| | Member Label | Direction | Start Magnitude [k/ft, F, ksf, k-ft/ft] | End Magnitude [k/ft, F, ksf, k-ft/ft] | Start Location [(ft, %)] | End Location [(ft, %)] |
|---|---|---|---|---|---|---|
| 1 | M4 | X | 0.36 | 0.36 | 0 | %100 |

### Member Distributed Loads (BLC 3 : Live Load)

| | Member Label | Direction | Start Magnitude [k/ft, F, ksf, k-ft/ft] | End Magnitude [k/ft, F, ksf, k-ft/ft] | Start Location [(ft, %)] | End Location [(ft, %)] |
|---|---|---|---|---|---|---|
| 1 | M4 | Y | -0.089 | -0.089 | 0 | %100 |

### Load Combinations

| | Description | Solve | P-Delta | BLC | Factor | BLC | Factor | BLC | Factor | BLC | Factor |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | SEISMIC DRIFT(+) | | Y | DL | 1 | EL | 1 | LL | 0.75 | | |
| 2 | SEISMIC DRIFT(-) | | Y | DL | 1 | EL | -1 | LL | 0.75 | | |
| 3 | Deflection 1 | Yes | Y | DL | 1 | | | | | | |





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RISA A NEMETSCHEK COMPANY

Company : TGE
Designer :
Job Number : 22-19079
Model Name :

4/20/2022
12:06:02 PM
Checked By : __________

### Load Combinations (Continued)

| | Description | Solve | P-Delta | BLC | Factor | BLC | Factor | BLC | Factor | BLC | Factor |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 4 | Deflection 2 | Yes | Y | LL | 1 | | | | | | |
| 5 | Deflection 3 | Yes | Y | DL | 1 | LL | 1 | | | | |
| 6 | IBC 16-1 | Yes | Y | DL | 1.4 | | | | | | |
| 7 | IBC 16-2 (a) | Yes | Y | DL | 1.2 | LL | 1.6 | LLS | 1.6 | RLL | 0.5 |
| 8 | IBC 16-2 (b) | Yes | Y | DL | 1.2 | LL | 1.6 | LLS | 1.6 | | |
| 9 | IBC 16-3 (a) | Yes | Y | DL | 1.2 | RLL | 1.6 | LL | 0.5 | LLS | 1 |
| 10 | IBC 16-5 | Yes | Y | DL | 1.2 | EL | 1 | LL | 0.5 | LLS | 1 |
| 11 | IBC 16-7 | Yes | Y | DL | 0.9 | EL | 1 | | | | |
| 12 | IBC 16-5 | Yes | Y | DL | 1.2 | Om*EL | 1 | LL | 0.5 | LLS | 1 |
| 13 | IBC 16-7 | Yes | Y | DL | 0.9 | Om*EL | 1 | | | | |

### Envelope Node Reactions - Overstrength or Capacity Limit

| | Node Label | | X [k] | LC | Y [k] | LC | Z [k] | LC | MX [k-ft] | LC | MY [k-ft] | LC | MZ [k-ft] | LC |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | N4 | max | 0 | 13* | 0 | 13* | 0 | 13* | 0 | 13* | 0 | 13* | 0 | 13* |
| 2 | | min | 0 | 12* | 0 | 12* | 0 | 12* | 0 | 12* | 0 | 12* | 0 | 12* |
| 3 | N2 | max | 0 | 13* | 0 | 13* | 0 | 13* | 0 | 13* | 0 | 13* | 0 | 13* |
| 4 | | min | 0 | 12* | 0 | 12* | 0 | 12* | 0 | 12* | 0 | 12* | 0 | 12* |
| 5 | N1 | max | -5.762 | 12* | -5.585 | 12* | 0 | 13* | 0 | 13* | 0 | 13* | 0 | 13* |
| 6 | | min | -5.846 | 13* | -6.269 | 13* | 0 | 12* | 0 | 12* | 0 | 12* | 0 | 12* |
| 7 | N3 | max | -5.909 | 13* | 8.907 | 12* | 0 | 13* | 0 | 13* | 0 | 13* | 0 | 13* |
| 8 | | min | -5.994 | 12* | 8.212 | 13* | 0 | 12* | 0 | 12* | 0 | 12* | 0 | 12* |
| 9 | Totals: | max | -11.756 | 13* | 3.321 | 12* | 0 | 13* | | | | | | |
| 10 | | min | -11.756 | 12* | 1.943 | 13* | 0 | 12* | | | | | | |

### Envelope Beam Deflections

| | Member Label | Span | | Location [ft] | y' [in] | (n) L'/y' Ratio | LC |
|---|---|---|---|---|---|---|---|
| 1 | M4 | 1 | max | 0.681 | -0.001 | NC | 6 |
| 2 | | 1 | min | 4.088 | -0.028 | 6898 | 10 |

### Envelope Member Section Forces

| | Member | Sec | | Axial[k] | LC | y Shear[k] | LC | z Shear[k] | LC | Torque[k-ft] | LC | y-y Moment[k-ft] | LC | z-z Moment[k-ft] | LC |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | M1 | 1 | max | 2.465 | 8 | 5.798 | 13* | 0 | 11 | 0 | 11 | 0 | 11 | 0 | 11 |
| 2 | | | min | -6.269 | 13* | -0.294 | 7 | 0 | 3 | 0 | 3 | 0 | 3 | 0 | 3 |
| 3 | | 2 | max | 2.328 | 8 | 5.798 | 13* | 0 | 11 | 0 | 11 | 0 | 11 | 0.739 | 8 |
| 4 | | | min | -6.371 | 13* | -0.294 | 7 | 0 | 3 | 0 | 3 | 0 | 3 | -14.581 | 13* |
| 5 | | 3 | max | 2.192 | 8 | 5.798 | 13* | 0 | 11 | 0 | 11 | 0 | 11 | 1.478 | 8 |
| 6 | | | min | -6.474 | 13* | -0.294 | 7 | 0 | 3 | 0 | 3 | 0 | 3 | -29.162 | 13* |
| 7 | | 4 | max | 2.055 | 8 | 5.798 | 13* | 0 | 11 | 0 | 11 | 0 | 11 | 2.217 | 8 |
| 8 | | | min | -6.576 | 13* | -0.294 | 7 | 0 | 3 | 0 | 3 | 0 | 3 | -43.742 | 13* |
| 9 | | 5 | max | 1.919 | 8 | 5.798 | 13* | 0 | 11 | 0 | 11 | 0 | 11 | 2.955 | 8 |
| 10 | | | min | -6.679 | 13* | -0.294 | 7 | 0 | 3 | 0 | 3 | 0 | 3 | -58.323 | 13* |
| 11 | M3 | 1 | max | 8.36 | 12* | -0.096 | 3 | 0 | 11 | 0 | 11 | 0 | 11 | -0.961 | 3 |
| 12 | | | min | 0.624 | 3 | -6.059 | 12* | 0 | 3 | 0 | 3 | 0 | 3 | -60.953 | 12* |
| 13 | | 2 | max | 8.497 | 12* | -0.096 | 3 | 0 | 11 | 0 | 11 | 0 | 11 | -0.721 | 3 |
| 14 | | | min | 0.731 | 4 | -6.059 | 12* | 0 | 3 | 0 | 3 | 0 | 3 | -45.715 | 12* |
| 15 | | 3 | max | 8.634 | 12* | -0.096 | 3 | 0 | 11 | 0 | 11 | 0 | 11 | -0.481 | 3 |
| 16 | | | min | 0.731 | 4 | -6.059 | 12* | 0 | 3 | 0 | 3 | 0 | 3 | -30.477 | 12* |
| 17 | | 4 | max | 8.77 | 12* | -0.096 | 3 | 0 | 11 | 0 | 11 | 0 | 11 | -0.24 | 3 |
| 18 | | | min | 0.731 | 4 | -6.059 | 12* | 0 | 3 | 0 | 3 | 0 | 3 | -15.238 | 12* |
| 19 | | 5 | max | 8.907 | 12* | -0.096 | 3 | 0 | 11 | 0 | 11 | 0 | 11 | 0 | 11 |
| 20 | | | min | 0.731 | 4 | -6.059 | 12* | 0 | 3 | 0 | 3 | 0 | 3 | 0 | 3 |
| 21 | M4 | 1 | max | 0.294 | 8 | 1.919 | 8 | 0 | 11 | 0 | 11 | 0 | 11 | 2.955 | 8 |





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***Envelope Member Section Forces (Continued)***

| | Member | Sec | | Axial[k] | LC | y Shear[k] | LC | z Shear[k] | LC | Torque[k-ft] | LC | y-y Moment[k-ft] | LC | z-z Moment[k-ft] | LC |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 22 | | | min | -2.867 | 11 | -3.059 | 11 | 0 | 3 | 0 | 3 | 0 | 3 | -28.729 | 11 |
| 23 | | 2 | max | 0.294 | 8 | 0.959 | 8 | 0 | 11 | 0 | 11 | 0 | 11 | -0.952 | 3 |
| 24 | | | min | -1.397 | 11 | -3.339 | 11 | 0 | 3 | 0 | 3 | 0 | 3 | -16.508 | 10 |
| 25 | | 3 | max | 0.294 | 8 | 0 | 9 | 0 | 11 | 0 | 11 | 0 | 11 | -1.43 | 11 |
| 26 | | | min | 0.072 | 11 | -3.623 | 10 | 0 | 3 | 0 | 3 | 0 | 3 | -4.887 | 7 |
| 27 | | 4 | max | 1.626 | 10 | -0.312 | 3 | 0 | 11 | 0 | 11 | 0 | 11 | 13.941 | 11 |
| 28 | | | min | 0.096 | 3 | -4.18 | 10 | 0 | 3 | 0 | 3 | 0 | 3 | -2.927 | 7 |
| 29 | | 5 | max | 3.096 | 10 | -0.624 | 3 | 0 | 11 | 0 | 11 | 0 | 11 | 31.335 | 10 |
| 30 | | | min | 0.096 | 3 | -4.737 | 10 | 0 | 3 | 0 | 3 | 0 | 3 | 0.961 | 3 |

***Envelope AISC 15TH (360-16): LRFD Member Steel Code Checks***

| | Member | Shape | Code Check | Loc[ft] | LC | Shear Check | Loc[ft] | Dir | LC | phi*Pnc [k] | phi*Pnt [k] | phi*Mn y-y [k-ft] | phi*Mn z-z [k-ft] | Cb | Eqn |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | M1 | W10X45 | 0.289 | 10.06 | 13* | 0.055 | 10.06 | y | 13* | 458.993 | 598.5 | 76.125 | 205.875 | 1.667 | H1-1b |
| 2 | M3 | W10X45 | 0.305 | 0 | 12* | 0.057 | 10.06 | y | 12* | 458.993 | 598.5 | 76.125 | 205.875 | 1.667 | H1-1b |
| 3 | M4 | W16X36 | 0.136 | 16.35 | 10 | 0.034 | 16.35 | y | 10 | 265.715 | 477 | 40.5 | 240 | 2.194 | H1-1b |

City of Puyallup
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RISA A NEMETSCHEK COMPANY

Company : TGE
Designer :
Job Number : 22-19079
Model Name :

4/20/2022
12:07:29 PM
Checked By : ___________

***Node Displacements (By Combination)***

| | LC | Node Label | X [in] | Y [in] | Z [in] | X Rotation [rad] | Y Rotation [rad] | Z Rotation [rad] |
|---|---|---|---|---|---|---|---|---|
| 1 | 2 | N1 | 0 | 0 | 0 | 0 | 0 | 5.005e-3 |
| 2 | 2 | N2 | -0.456 | -0.002 | 0 | 0 | 0 | 1.049e-3 |
| 3 | 2 | N3 | 0 | 0 | 0 | 0 | 0 | 4.864e-3 |
| 4 | 2 | N4 | -0.456 | 0.001 | 0 | 0 | 0 | 1.363e-3 |

# SOFTWARE PRINTOUTS (RISA CONNECTION)

RISA A NEMETSCHEK COMPANY

***RISA Connection version 12.0.2***

*04.20.2022*

## *Global Parameters - Description:*

| | |
|---|---|
| Project Title | 22-19079 |
| Company | TGE |
| Designer | |
| Job Number | 22-19079 |
| Notes | |

## *Global Parameters - Solution:*

| | |
|---|---|
| Design Method | AISC 15th (360-16): LRFD |
| Bolt Group Analysis Method | Center of Rotation |
| Weld Analysis Method | Elastic |
| Consider Bolt Hole Deformation? | Yes |
| Check Rotational Ductility? | Yes |
| Check Weld Filler Metal Matching? | Yes |
| Full Shear Eccentricity Considered? | No |
| Panel-Zone Shear Deformation Considered? | No |
| Check Weld Base Material Thickness? | Yes |
| Reduce Available Bolt Strength by Prying Effects Factor Q? | No |





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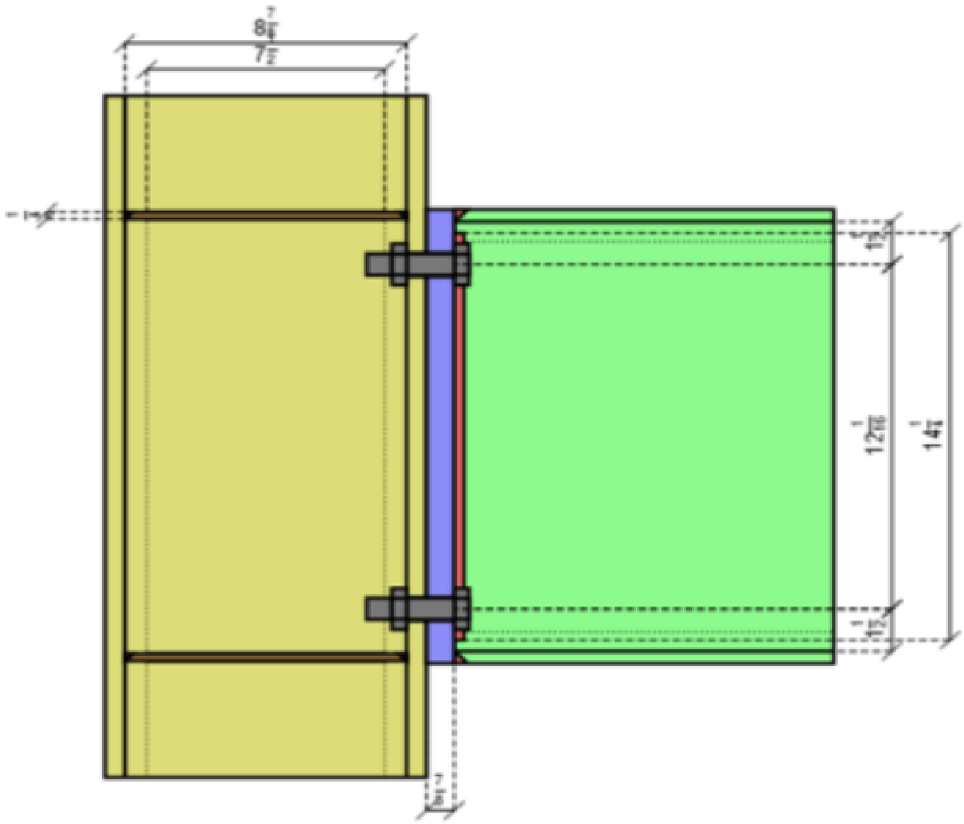
## *M4 I - M1: 2D Views Report*

*Column/Beam Flush End Plate Moment Connection*

Side view



*continued on next page...*





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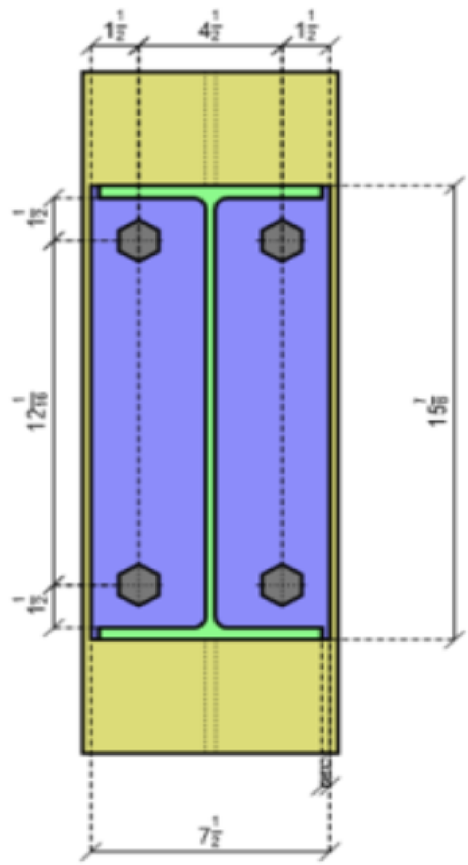
### *M4 I - M1: 2D Views Report (continued):*

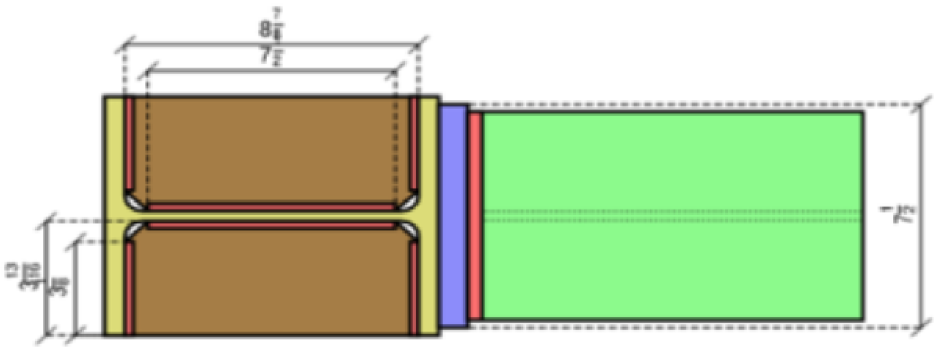
Front view



*continued on next page...*

## *M4 I - M1: 2D Views Report (continued):*

Top view







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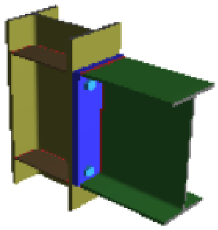
**LRFD**

## M4 I - M1: LRFD Results Report

*Column/Beam Flush End Plate Moment Connection*



| Material Properties: | | | | |
|---|---|---|---|---|
| **Column** | W10X45 | A992 | $F_y$ = 50.00 ksi | $F_u$ = 65.00 ksi |
| **Beam** | W16X36 | A992 | $F_y$ = 50.00 ksi | $F_u$ = 65.00 ksi |
| **Plate** | P0.88x7.50x15.90 | A36 | $F_y$ = 36.00 ksi | $F_u$ = 58.00 ksi |
| **Transverse Stiffener** | P0.25x3.83x8.86 | A36 | $F_y$ = 36.00 ksi | $F_u$ = 58.00 ksi |

| Input Data: | | |
|---|---|---|
| **Shear Load** | -6.68 kips | *User Input Shear Load* |
| **Moment** | -58.32 kips-ft | *User Input Moment* |
| **Axial Load** | -5.85 kips | *User Input Axial Force (tension)* |
| **Puf_c** | 42.32 kips | *Required Flange Force (compression)* |
| **Puf_t** | 48.16 kips | *Required Flange Force (tension)* |
| **Top Column Dist** | 0 in | *User Input Top Column Dist* |
| **Column Force** | -6.67 kips | *User Input Column Force* |
| **Story Shear** | -5.80 kips | *User Input Story Shear* |

Governing LC: 3D - 13* - IBC 16-7

Note: Unless specified, all code references are from AISC 360-16

| Limit State | Required | Available | Unity Check | Result |
|---|---|---|---|---|
| **Geometry Restrictions at Column** | | | | **PASS** |
| **Bolt Shear Strength** | 6.68 kips | 35.78 kips | **0.19** | **PASS** |
| **Bolt Bearing on Column** | 6.68 kips | 35.78 kips | **0.19** | **PASS** |
| **Bolt Bearing on Plate at Column** | 6.68 kips | 35.78 kips | **0.19** | **PASS** |
| **Beam Web Weld Strength** | | | | **PASS** |
| **Beam Flange Weld Strength** | | | | **PASS** |
| **Bolt Moment Strength (no prying)** | 48.16 kips | 53.03 kips | **0.91** | **PASS** |
| **Verify Bolt Prying Assumption** | | | | **PASS** |
| **End Plate Flexural Yielding** | 60.21 kips | 126.79 kips | **0.47** | **PASS** |
| **End Plate Shear Yielding** | 60.21 kips | 141.75 kips | **0.42** | **PASS** |
| **End Plate Shear Rupture** | 60.21 kips | 131.32 kips | **0.46** | **PASS** |
| **Column Flexural Yielding** | 60.21 kips | 90.72 kips | **0.66** | **PASS** |
| **Column Web Yielding** | 48.16 kips | 65.32 kips | **0.74** | **PASS** |
| **Column Web Buckling** | 42.32 kips | 70.94 kips | **0.60** | **PASS** |
| **Column Web Crippling** | 42.32 kips | 68.58 kips | **0.62** | **PASS** |
| **Column Panel Zone Shear** | 51.04 kips | 95.44 kips | **0.53** | **PASS** |





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## *M4 I - M1: Members Report*

*Column/Beam Flush End Plate Moment Connection*

| | | |
|---|---|---|
| ***Column*** | ***W10X45*** | |
| **Material** | | |
| **Name** | A992 | *Material name* |
| $F_y$ | 50.00 ksi | *Minimum yield stress of material* |
| $F_u$ | 65.00 ksi | *Minimum tensile stress of material* |
| **E** | 29000.00 ksi | *Modulus of elasticity* |
| **Member Properties** | | |
| $b_f$ | 8 in | *Flange width* |
| **d** | 10 1/8 in | *Overall depth* |
| $t_w$ | 3/8 in | *Web thickness* |
| $t_f$ | 5/8 in | *Flange thickness* |
| **a** | 13.30 $in^2$ | *Area* |
| $k_{des}$ | 1 1/8 in | *Kdes* |
| $k_{det}$ | 1 5/16 in | *Kdet* |
| $k_1$ | 13/16 in | *K1* |
| **Hole** | | |
| **Hole type** | Standard | |
| $D_x$ | 13/16 in | *Hole width* |
| $D_y$ | 13/16 in | *Hole height* |
| **R** | 1 | *Number of rows of holes* |
| **C** | 1 | *Number of holes per row* |
| $R_s$ | 3 in | *Row Spacing* |
| $C_s$ | 3 in | *Column Spacing* |
| ***Beam*** | ***W16X36*** | |
| **Material** | | |
| **Name** | A992 | *Material name* |
| $F_y$ | 50.00 ksi | *Minimum yield stress of material* |
| $F_u$ | 65.00 ksi | *Minimum tensile stress of material* |
| **E** | 29000.00 ksi | *Modulus of elasticity* |
| **Member Properties** | | |
| $b_f$ | 7 in | *Flange width* |
| **d** | 15 7/8 in | *Overall depth* |
| $t_w$ | 5/16 in | *Web thickness* |
| $t_f$ | 7/16 in | *Flange thickness* |
| **a** | 10.60 $in^2$ | *Area* |
| $k_{des}$ | 13/16 in | *Kdes* |
| $k_{det}$ | 1 1/8 in | *Kdet* |
| $k_1$ | 3/4 in | *K1* |





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## M4 I - M1: Connection Properties Report

*Column/Beam Flush End Plate Moment Connection*

| | |
|---|---|
| **Connection** | |
| **Connection Title** | M4 I - M1 |
| **Connection Type** | Column/Beam Flush End Plate Moment Connection |
| **Connection Category** | |
| **Tension Side** | Top |
| **Bolt Pattern** | 2 Bolts |
| **Transverse Stiffeners** | Yes |
| **Web Doublers** | No |
| **Loading (LRFD)** | |
| **Custom?** | No |
| **Shear Load** | -6.68 kips |
| **Axial Load** | -5.85 kips |
| **Moment Load** | -58.32 kips-ft |
| **Top Column Dist** | 0 in |
| **Column Force** | -6.67 kips |
| **Story Shear** | -5.80 kips |
| **Components** | |
| **Column Section** | W10X45 |
| Material | A992 |
| Hole Type | STD |
| **Beam Section** | W16X36 |
| Material | A992 |
| **Plate Section** | P0.88x7.50x15.90 |
| Material | A36 |
| Thickness | 7/8 in |
| Width | 7 1/2 in |
| Depth | 15 7/8 in |
| Hole Type | STD |
| **Transverse Stiffener Section** | P0.25x3.83x8.86 |
| Material | A36 |
| Fy | 36.00 ksi |
| Fu | 58.00 ksi |
| E | 29000.00 ksi |
| Full Depth Stiffener | Yes |
| Thickness | 1/4 in |
| Width | 3 13/16 in |
| Depth | 8 7/8 in |
| **Column Bolts** | 3/4" Group A-N |
| Column Bolts | Group A-N |
| Diameter, in. | 3/4" |
| **Beam Web Weld** | E70 |
| Type | CJP |
| **Beam Flange Weld** | E70 |
| Type | CJP |
| **Transverse Stiffener Weld** | E70 |
| Type | CJP |
| **Assembly** | |
| **Plate Beam Clearance** | 1/4 in |
| **Column Bolts Flange Pitch** | 1 1/2 in |
| **Compression Bolts Flange Pitch** | 1 1/2 in |
| **Column Bolts Horz Edge Dist** | 1 1/2 in |
| **Column Bolts Horizontal Gage** | 4 1/2 in |





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# SOFTWARE PRINTOUTS (ANCHOR)

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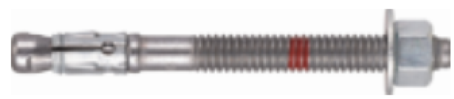
| | | | |
|---|---|---|---|
| Company: | | Page: | 1 |
| Address: | | Specifier: | |
| Phone I Fax: | \| | E-Mail: | |
| Design: | Drafts_Wide-Flange-MF | Date: | 4/20/2022 |
| Fastening point: | wide flange MF | | |

**Specifier's comments:**

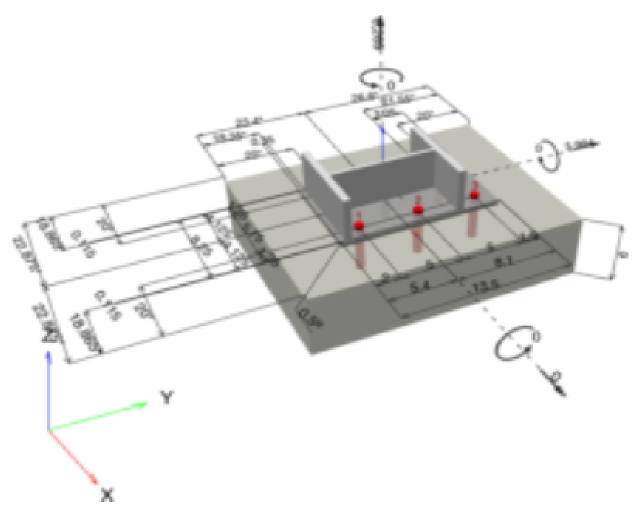
## 1 Input data



| | |
|---|---|
| **Anchor type and diameter:** | **Kwik Bolt TZ2 - CS 3/4 (3 3/4) hnom2** |
| Item number: | 2210312 KB-TZ2 3/4x6 1/4 |
| Effective embedment depth: | $h_{ef,act}$ = 3.750 in., $h_{nom}$ = 4.500 in. |
| Material: | Carbon Steel |
| Evaluation Service Report: | ESR-4266 |
| Issued I Valid: | 12/17/2021 \| 12/1/2023 |
| Proof: | Design Method ACI 318-14 / Mech |
| Stand-off installation: | $e_b$ = 0.000 in. (no stand-off); t = 0.500 in. |
| Anchor plate$^R$ : | $l_x$ x $l_y$ x t = 8.250 in. x 13.500 in. x 0.500 in.; (Recommended plate thickness: not calculated) |
| Profile: | W shape (AISC), W10X45; (L x W x T x FT) = 10.100 in. x 8.020 in. x 0.350 in. x 0.620 in. |
| Base material: | cracked concrete, 3000, $f_c'$ = 3,000 psi; h = 6.000 in. |
| **Installation:** | **hammer drilled hole, Installation condition: Dry** |
| Reinforcement: | tension: condition B, shear: condition B; no supplemental splitting reinforcement present<br>edge reinforcement: none or < No. 4 bar |
| Seismic loads (cat. C, D, E, or F) | Tension load: yes (17.2.3.4.3 (d))<br>Shear load: yes (17.2.3.5.3 (c)) |

$^R$ - The anchor calculation is based on a rigid anchor plate assumption.

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



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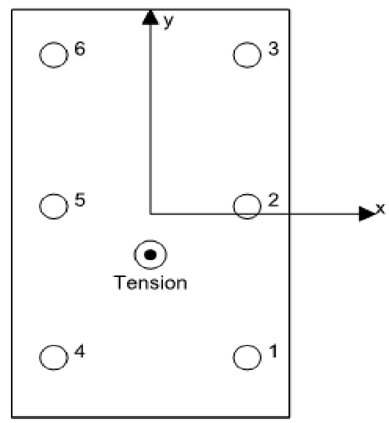
| | | | |
|---|---|---|---|
| Company: | | Page: | 2 |
| Address: | | Specifier: | |
| Phone I Fax: | \| | E-Mail: | |
| Design: | Drafts_Wide-Flange-MF | Date: | 4/20/2022 |
| Fastening point: | wide flange MF | | |

### 1.1 Design results

| Case | Description | Forces [lb] / Moments [in.lb] | Seismic | Max. Util. Anchor [%] |
|---|---|---|---|---|
| 1 | Combination 1 | $N = 6{,}269$; $V_x = 0$; $V_y = 5{,}994$; $M_x = 0$; $M_y = 0$; $M_z = 0$; | yes | 99 |

## 2 Load case/Resulting anchor forces




**Anchor reactions [lb]**

Tension force: (+Tension, -Compression)

| Anchor | Tension force | Shear force | Shear force x | Shear force y |
|---|---|---|---|---|
| 1 | 1,546 | 999 | 0 | 999 |
| 2 | 1,045 | 999 | 0 | 999 |
| 3 | 543 | 999 | 0 | 999 |
| 4 | 1,546 | 999 | 0 | 999 |
| 5 | 1,045 | 999 | 0 | 999 |
| 6 | 543 | 999 | 0 | 999 |

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/-1.350): 6,269 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

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

## 3 Tension load

| | Load $N_{ua}$ [lb] | Capacity $\phi$ $N_n$ [lb] | Utilization $\beta_N = N_{ua}/\phi$ $N_n$ | Status |
|---|---|---|---|---|
| Steel Strength* | 1,546 | 19,009 | 9 | OK |
| Pullout Strength* | N/A | N/A | N/A | N/A |
| Concrete Breakout Failure** | 6,269 | 9,049 | 70 | OK |

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

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| Fastening point: | wide flange MF | | |

## 3.1 Steel Strength

$N_{sa}$ = ESR value    refer to ICC-ES ESR-4266

$\phi\ N_{sa} \geq N_{ua}$    ACI 318-14 Table 17.3.1.1

### Variables

| $A_{se,N}$ [in.$^2$] | $f_{uta}$ [psi] |
|---|---|
| 0.24 | 105,904 |

### Calculations

| $N_{sa}$ [lb] |
|---|
| 25,345 |

### Results

| $N_{sa}$ [lb] | $\phi_{steel}$ | $\phi_{nonductile}$ | $\phi\ N_{sa}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|---|
| 25,345 | 0.750 | 1.000 | 19,009 | 1,546 |

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| Fastening point: | wide flange MF | | |

### 3.2 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-14 Eq. (17.4.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-14 Table 17.3.1.1

$A_{Nc}$ see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{1.5}$ — ACI 318-14 Eq. (17.4.2.2a)

**Variables**

| $h_{ef}$ [in.] | $e_{c1,N}$ [in.] | $e_{c2,N}$ [in.] | $c_{a,min}$ [in.] | $\psi_{c,N}$ |
|---|---|---|---|---|
| 3.750 | 0.000 | 1.600 | 20.000 | 1.000 |

| $c_{ac}$ [in.] | $k_c$ | $\lambda_a$ | $f_c'$ [psi] |
|---|---|---|---|
| 10.000 | 21 | 1.000 | 3,000 |

**Calculations**

| $A_{Nc}$ [in.$^2$] | $A_{Nc0}$ [in.$^2$] | $\psi_{ec1,N}$ | $\psi_{ec2,N}$ | $\psi_{ed,N}$ | $\psi_{cp,N}$ | $N_b$ [lb] |
|---|---|---|---|---|---|---|
| 361.25 | 126.56 | 1.000 | 0.779 | 1.000 | 1.000 | 8,353 |

**Results**

| $N_{cbg}$ [lb] | $\phi_{concrete}$ | $\phi_{seismic}$ | $\phi_{nonductile}$ | $\phi\ N_{cbg}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|---|---|
| 18,562 | 0.650 | 0.750 | 1.000 | 9,049 | 6,269 |

Input data and results must be checked for conformity with the existing conditions and for plausibility!

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| Fastening point: | wide flange MF | | |

## 4 Shear load

| | Load $V_{ua}$ [lb] | Capacity $\phi$ $V_n$ [lb] | Utilization $\beta_V = V_{ua}/\phi$ $V_n$ | Status |
|---|---|---|---|---|
| Steel Strength* | 999 | 8,977 | 12 | OK |
| Steel failure (with lever arm)* | N/A | N/A | N/A | N/A |
| Pryout Strength** | 5,994 | 33,378 | 18 | OK |
| Concrete edge failure in direction y+** | 5,994 | 9,781 | 62 | OK |

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

### 4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-4266

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

**Variables**

| $A_{se,V}$ [in.$^2$] | $f_{uta}$ [psi] | $\alpha_{V,seis}$ |
|---|---|---|
| 0.24 | 105,904 | 1.000 |

**Calculations**

| $V_{sa,eq}$ [lb] |
|---|
| 13,811 |

**Results**

| $V_{sa,eq}$ [lb] | $\phi_{steel}$ | $\phi_{nonductile}$ | $\phi\ V_{sa,eq}$ [lb] | $V_{ua}$ [lb] |
|---|---|---|---|---|
| 13,811 | 0.650 | 1.000 | 8,977 | 999 |

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| Fastening point: | wide flange MF | | |

## 4.2 Pryout Strength

$$V_{cpg} = k_{cp}\left[\left(\frac{A_{Nc}}{A_{Nc0}}\right)\psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$$ ACI 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

$A_{Nc}$ see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$$\psi_{ec,N} = \left(\frac{1}{1+\frac{2\ e_N}{3\ h_{ef}}}\right) \leq 1.0$$ ACI 318-14 Eq. (17.4.2.4)

$$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$$ ACI 318-14 Eq. (17.4.2.5b)

$$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

### Variables

| $k_{cp}$ | $h_{ef}$ [in.] | $e_{c1,N}$ [in.] | $e_{c2,N}$ [in.] | $c_{a,min}$ [in.] |
|---|---|---|---|---|
| 2 | 3.750 | 0.000 | 0.000 | 20.000 |

| $\psi_{c,N}$ | $c_{ac}$ [in.] | $k_c$ | $\lambda_a$ | $f'_c$ [psi] |
|---|---|---|---|---|
| 1.000 | 10.000 | 21 | 1.000 | 3,000 |

### Calculations

| $A_{Nc}$ [in.$^2$] | $A_{Nc0}$ [in.$^2$] | $\psi_{ec1,N}$ | $\psi_{ec2,N}$ | $\psi_{ed,N}$ | $\psi_{cp,N}$ | $N_b$ [lb] |
|---|---|---|---|---|---|---|
| 361.25 | 126.56 | 1.000 | 1.000 | 1.000 | 1.000 | 8,353 |

### Results

| $V_{cpg}$ [lb] | $\phi_{concrete}$ | $\phi_{seismic}$ | $\phi_{nonductile}$ | $\phi\ V_{cpg}$ [lb] | $V_{ua}$ [lb] |
|---|---|---|---|---|---|
| 47,683 | 0.700 | 1.000 | 1.000 | 33,378 | 5,994 |

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### 4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ ACI 318-14 Eq. (17.5.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

$A_{Vc}$ see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_V}{3c_{a1}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.5)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-14 Eq. (17.5.2.8)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\ c_{a1}^{1.5}$ ACI 318-14 Eq. (17.5.2.2a)

**Variables**

| $c_{a1}$ [in.] | $c_{a2}$ [in.] | $e_{cV}$ [in.] | $\psi_{c,V}$ | $h_a$ [in.] |
|---|---|---|---|---|
| 13.333 | 20.000 | 0.000 | 1.000 | 6.000 |

| $l_e$ [in.] | $\lambda_a$ | $d_a$ [in.] | $f'_c$ [psi] | $\psi_{parallel,V}$ |
|---|---|---|---|---|
| 3.750 | 1.000 | 0.750 | 3,000 | 1.000 |

**Calculations**

| $A_{Vc}$ [in.$^2$] | $A_{Vc0}$ [in.$^2$] | $\psi_{ec,V}$ | $\psi_{ed,V}$ | $\psi_{h,V}$ | $V_b$ [lb] |
|---|---|---|---|---|---|
| 274.50 | 800.00 | 1.000 | 1.000 | 1.826 | 22,304 |

**Results**

| $V_{cbg}$ [lb] | $\phi_{concrete}$ | $\phi_{seismic}$ | $\phi_{nonductile}$ | $\phi\ V_{cbg}$ [lb] | $V_{ua}$ [lb] |
|---|---|---|---|---|---|
| 13,973 | 0.700 | 1.000 | 1.000 | 9,781 | 5,994 |

## 5 Combined tension and shear loads

| $\beta_N$ | $\beta_V$ | $\zeta$ | Utilization $\beta_{N,V}$ [%] | Status |
|---|---|---|---|---|
| 0.693 | 0.613 | 5/3 | 99 | OK |

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

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## 6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by $\omega_0$.
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-14, Section 17.8.1.

**Fastening meets the design criteria!**

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## 7 Installation data

Profile: W shape (AISC), W10X45; (L x W x T x FT) = 10.100 in. x 8.020 in. x 0.350 in. x 0.620 in.

Hole diameter in the fixture: $d_f$ = 0.812 in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

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

Anchor type and diameter: Kwik Bolt TZ2 - CS 3/4 (3 3/4) hnom2

Item number: 2210312 KB-TZ2 3/4x6 1/4

Maximum installation torque: 1,324 in.lb

Hole diameter in the base material: 0.750 in.

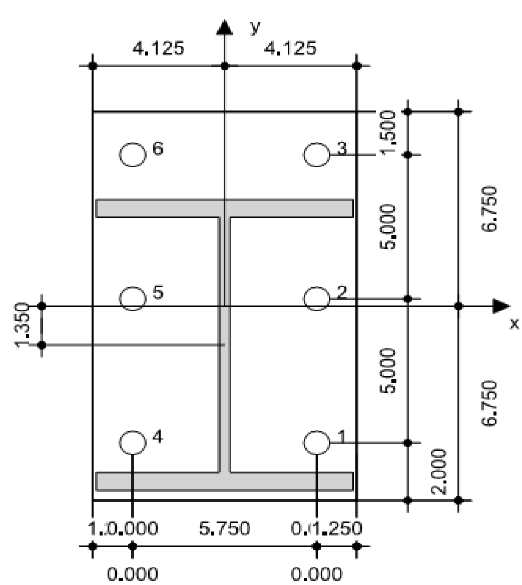
Hole depth in the base material: 4.750 in.

Minimum thickness of the base material: 6.000 in.

Hilti KB-TZ2 stud anchor with 4.5 in embedment, 3/4 (3 3/4) hnom2, Carbon steel, installation per ESR-4266

### 7.1 Recommended accessories

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




**Coordinates Anchor [in.]**

| Anchor | x | y | $c_{-x}$ | $c_{+x}$ | $c_{-y}$ | $c_{+y}$ |
|---|---|---|---|---|---|---|
| 1 | 2.875 | -4.750 | 25.750 | 20.000 | 20.000 | 30.000 |
| 2 | 2.875 | 0.250 | 25.750 | 20.000 | 25.000 | 25.000 |
| 3 | 2.875 | 5.250 | 25.750 | 20.000 | 30.000 | 20.000 |
| 4 | -2.875 | -4.750 | 20.000 | 25.750 | 20.000 | 30.000 |
| 5 | -2.875 | 0.250 | 20.000 | 25.750 | 25.000 | 25.000 |
| 6 | -2.875 | 5.250 | 20.000 | 25.750 | 30.000 | 20.000 |

Input data and results must be checked for conformity with the existing conditions and for plausibility!

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## 8 Remarks; Your Cooperation Duties

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

Input data and results must be checked for conformity with the existing conditions and for plausibility!



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| ign: | | Date: | 4/1/2 |
| tening point: | WALL TO FLOOR | | |

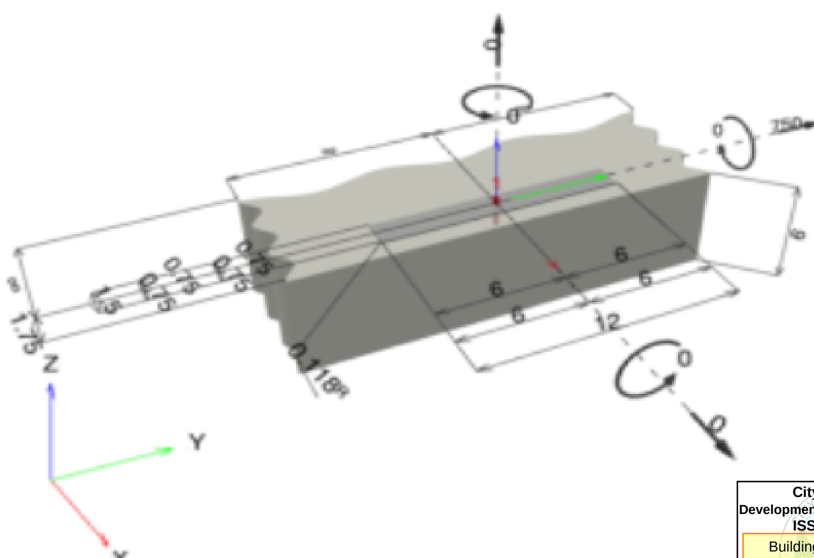
pecifier's comments:

## Input data

| | |
|---|---|
| **ichor type and diameter:** | **KWIK HUS-EZ (KH-EZ) 1/4 (1 5/8)** |
| m number: | 423473 KH-EZ 1/4"x1 7/8" |
| fective embedment depth: | $h_{ef,act}$ = 1.180 in., $h_{nom}$ = 1.625 in. |
| aterial: | Carbon Steel |
| aluation Service Report: | ESR-3027 |
| sued I Valid: | 1/1/2021 \| 12/1/2021 |
| oof: | Design Method ACI 318-14 / Mech |
| and-off installation: | $e_b$ = 0.000 in. (no stand-off); t = 0.118 in. |
| ichor plate$^R$ : | $l_x$ x $l_y$ x t = 1.500 in. x 12.000 in. x 0.118 in.; (Recommended plate thickness: not calculated) |
| ofile: | no profile |
| ise material: | cracked concrete, 3000, $f_c'$ = 3,000 psi; h = 6.000 in. |
| **stallation:** | **hammer drilled hole, Installation condition: Dry** |
| einforcement: | tension: condition B, shear: condition B; no supplemental splitting reinforcement present |
| | edge reinforcement: none or < No. 4 bar |
| eismic loads (cat. C, D, E, or F) | Tension load: yes (17.2.3.4.3 (d)) |
| | Shear load: yes (17.2.3.5.3 (c)) |

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

**ometry [in.] & Loading [lb, in.lb]**

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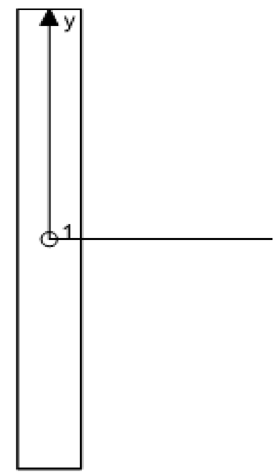
| tening point: | WALL TO FLOOR | | |

**Design results**

| Case | Description | Forces [lb] / Moments [in.lb] | Seismic | Max. Util. Anchor |
|---|---|---|---|---|
| 1 | Combination 1 | $N = 0; V_x = 0; V_y = 750;$ $M_x = 0; M_y = 0; M_z = 0;$ | yes | 91 |

## Load case/Resulting anchor forces




**chor reactions [lb]**
ısion force: (+Tension, -Compression)

| Anchor | Tension force | Shear force | Shear force x | Shear force y |
|---|---|---|---|---|
| 1 | 0 | 750 | 0 | 750 |

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

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

## Tension load

| | Load $N_{ua}$ [lb] | Capacity $\phi$ $N_n$ [lb] | Utilization $\beta_N = N_{ua}/\phi$ $N_n$ | Status |
|---|---|---|---|---|
| ≥el Strength* | N/A | N/A | N/A | N/A |
| ıllout Strength* | N/A | N/A | N/A | N/A |
| ıncrete Breakout Failure** | N/A | N/A | N/A | N/A |

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

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| tening point: | WALL TO FLOOR | | |

## Shear load

| | Load $V_{ua}$ [lb] | Capacity $\phi$ $V_n$ [lb] | Utilization $\beta_V = V_{ua}/\phi$ $V_n$ | Status |
|---|---|---|---|---|
| eel Strength* | 750 | 837 | 90 | OK |
| eel failure (with lever arm)* | N/A | N/A | N/A | N/A |
| yout Strength** | 750 | 828 | 91 | OK |
| ncrete edge failure in direction x+** | 750 | 847 | 89 | OK |

ighest loaded anchor **anchor group (relevant anchors)

### Steel Strength

$_{a,eq}$ = ESR value refer to ICC-ES ESR-3027
$V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

**iriables**

| $A_{se,V}$ [in.$^2$] | $f_{uta}$ [psi] | $\alpha_{V,seis}$ |
|---|---|---|
| 0.05 | 125,000 | 0.900 |

**ilculations**

| $V_{sa,eq}$ [lb] |
|---|
| 1,395 |

**esults**

| $V_{sa,eq}$ [lb] | $\phi_{steel}$ | $\phi_{nonductile}$ | $\phi$ $V_{sa,eq}$ [lb] | $V_{ua}$ [lb] |
|---|---|---|---|---|
| 1,395 | 0.600 | 1.000 | 837 | 750 |

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**Pryout Strength**

p $= k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$ ACI 318-14 Eq. (17.5.3.1a)

$V_{cp} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Nc see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

Nc0 $= 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

ed,N $= 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

cp,N $= \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$= k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

**riables**

| $k_{cp}$ | $h_{ef}$ [in.] | $c_{a,min}$ [in.] | $\psi_{c,N}$ |
|---|---|---|---|
| 1 | 1.180 | 1.750 | 1.000 |

| $c_{ac}$ [in.] | $k_c$ | $\lambda_a$ | $f'_c$ [psi] |
|---|---|---|---|
| 2.000 | 17 | 1.000 | 3,000 |

**lculations**

| $A_{Nc}$ [in.$^2$] | $A_{Nc0}$ [in.$^2$] | $\psi_{ed,N}$ | $\psi_{cp,N}$ | $N_b$ [lb] |
|---|---|---|---|---|
| 12.46 | 12.53 | 0.997 | 1.000 | 1,194 |

**sults**

| $V_{cp}$ [lb] | $\phi_{concrete}$ | $\phi_{seismic}$ | $\phi_{nonductile}$ | $\phi\ V_{cp}$ [lb] | $V_{ua}$ [lb] |
|---|---|---|---|---|---|
| 1,183 | 0.700 | 1.000 | 1.000 | 828 | 750 |

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data and results must be checked for conformity with the existing conditions and for plausibility!

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**Concrete edge failure in direction x+**

| | |
|---|---|
| $_{b} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ | ACI 318-14 Eq. (17.5.2.1a) |
| $V_{cb} \geq V_{ua}$ | ACI 318-14 Table 17.3.1.1 |
| $_{c}$ see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b) | |
| $_{c0} = 4.5\ c_{a1}^2$ | ACI 318-14 Eq. (17.5.2.1c) |
| $_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ | ACI 318-14 Eq. (17.5.2.6b) |
| $_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ | ACI 318-14 Eq. (17.5.2.8) |
| $= \left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right) \lambda_a\ \sqrt{f'_c}\ c_{a1}^{1.5}$ | ACI 318-14 Eq. (17.5.2.2a) |

**ıriables**

| $c_{a1}$ [in.] | $c_{a2}$ [in.] | $\psi_{c,V}$ | $h_a$ [in.] | $l_e$ [in.] |
|---|---|---|---|---|
| 1.750 | - | 1.000 | 6.000 | 1.180 |

| $\lambda_a$ | $d_a$ [in.] | $f'_c$ [psi] | $\psi_{parallel,V}$ |
|---|---|---|---|
| 1.000 | 0.250 | 3,000 | 2.000 |

**ılculations**

| $A_{Vc}$ [in.$^2$] | $A_{Vc0}$ [in.$^2$] | $\psi_{ed,V}$ | $\psi_{h,V}$ | $V_b$ [lb] |
|---|---|---|---|---|
| 13.78 | 13.78 | 1.000 | 1.000 | 605 |

**ɘsults**

| $V_{cb}$ [lb] | $\phi_{concrete}$ | $\phi_{seismic}$ | $\phi_{nonductile}$ | $\phi\ V_{cb}$ [lb] | $V_{ua}$ [lb] |
|---|---|---|---|---|---|
| 1,211 | 0.700 | 1.000 | 1.000 | 847 | 750 |

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## Warnings

'he anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered – the nchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculat ne minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. Tl roof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for greement with the existing conditions and for plausibility!

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

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

'or additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 7.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the ase, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The onnection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).

Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductil ielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5. o) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the nchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design trength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increas y $\omega_0$.

Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-14, Section 17.8.1.

**Fastening meets the design criteria!**

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## Installation data

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) 1/4 (1 5/8)

ofile: no profile

Item number: 423473 KH-EZ 1/4"x1 7/8"

le diameter in the fixture: $d_f$ = 0.375 in.

Maximum installation torque: 216 in.lb

ate thickness (input): 0.118 in.

Hole diameter in the base material: 0.250 in.

commended plate thickness: not calculated

Hole depth in the base material: 1.836 in.

illing method: Hammer drilled

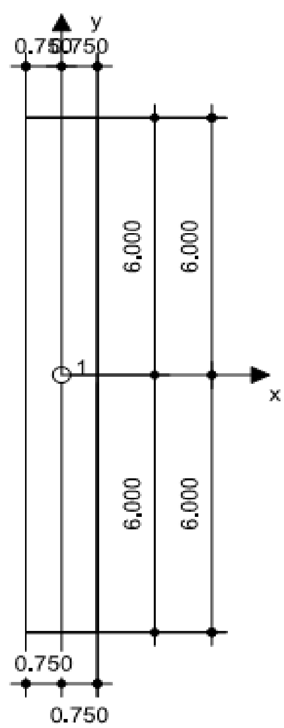
Minimum thickness of the base material: 3.250 in.

eaning: Manual cleaning of the drilled hole according to instructions for use is quired.

ti KH-EZ screw anchor with 1.625 in embedment, 1/4 (1 5/8), Carbon steel, installation per ESR-3027

### Recommended accessories

| lling | Cleaning | Setting |
|---|---|---|
| uitable Rotary Hammer<br>'roperly sized drill bit | • Manual blow-out pump | • Torque wrench |




ordinates Anchor [in.]

| Anchor | x | y | $c_{-x}$ | $c_{+x}$ | $c_{-y}$ | $c_{+y}$ |
|---|---|---|---|---|---|---|
| 1 | 0.000 | 0.000 | - | 1.750 | - | - |



data and results must be checked for conformity with the existing conditions and for plausibility!

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| tening point: | WALL TO FLOOR | | |

## Remarks; Your Cooperation Duties

ıny and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas ar ecurity regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly omplied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using ıe relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. 'herefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. ıoreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to ompliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norn ınd permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific pplication.

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

# TESTING REPORT OR LARR

## CITY OF LOS ANGELES

CALIFORNIA



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.

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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
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Attachment: Span Chart (4 page)



RR 24921

# LARR 24921 TABLE 1

Sheet Number 147 of 147

| PANELS | | | | | TOPS/CEILINGS | | | | | | | | | | WALLS | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| FINISH | SKIN THICKNESS | PANEL THICKNESS | DEAD LOAD PSF | R VALUE | INDOOR Defl L/180 | OUTDOOR Includes 2 PSF for Membrane or Standing Seam Roof Design based on deflection criteria: L/240 LIVE LOADS – PSF | | | | | | | | | 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 | | | | | | | | |
| | | | | | | | | | | | | | | | | 85 MPH 110 MPH | 90 MPH 115 MPH | 95 MPH 120 MPH | 100 MPH 130 MPH | 110 MPH 140 MPH | 115 MPH 150 MPH | 125 MPH 160 MPH | 140 MPH 180 MPH | 155 MPH 200 MPH |
| | | | | | 10 | 20 | 30 | 40 | 50 | 60 | 70 | 80 | 90 | 100 | 5 | 13.9 | 15.5 | 17.4 | 19.5 | 23.2 | 25.9 | 29.9 | 37.6 | 46.3 |
| WOOD FRAME | | | | | | | | | | | | | | | 2.2-POUND DENSITY | | | | | | | | | Fc=18psi |
| ALUM | 0.032" | 2½" | 2.0 | 20 | 12'-6" | Do not use 2 1/2" panels outdoors | | | | | | | | | 15'-2" | 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" | 20'-2" | 13'-11" | 13'-4" | 12'-8" | 12'-0" | 11'-1" | 10'-6" | 9'-9" | 8'-7" | 7'-7" |
| | | 4" | 3.5 | 32 | 17'-7" | 14'-4" | 11'-9" | 10'-2" | 9'-1" | 8'-3" | 7'-7" | 7'-1" | 6'-8" | 6'-4" | 22'-6" | 15'-0" | 14'-4" | 13'-8" | 13'-0" | 12'-4" | 11'-9" | 11'-0" | 9'-11" | 8'-11" |
| | | 5" | 4.0 | 40 | 20'-9" | 14'-10" | 13'-2" | 12'-1" | 11'-3" | 10'-7" | 10'-1" | 9'-7" | 9'-3" | 8'-7" | 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 | 13'-8" | Do not use 2 1/2" panels outdoors | | | | | | | | | 16'-7" | 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" | 22'-10" | 15'-5" | 14'-9" | 14'-1" | 13'-6" | 12'-8" | 12'-1" | 11'-5" | 10'-6" | 9'-8" |
| | | 4" | 4.0 | 32 | 19'-3" | 13'-8" | 12'-1" | 10'-11" | 10'-0" | 9'-3" | 8'-7" | 8'-0" | 7'-6" | 7'-1" | 23'-6" | 16'-3" | 15'-8" | 15'-0" | 14'-5" | 13'-6" | 13'-0" | 12'-4" | 11'-4" | 10'-7" |
| | | 5" | 4.5 | 40 | 22'-9" | 16'-5" | 14'-6" | 13'-2" | 12'-0" | 11'-2" | 10'-4" | 9'-8" | 9'-1" | 8'-7" | 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"* | 18'-6" | 15'-8" | 13'-11" | 12'-8" | 11'-9" | 11'-0" | 10'-5" | 9'-11" | 9'-5" | 26'-0"* | 26'-0" | 24'-11" | 22'-7" | 20'-0" | 15'-11" | 15'-9" | 15'-7" | 15'-2" | 14'-8" |
| INSULFRAME/HIGH DENSITY RAIL (HDR) | | | | | | | | | | | | | | | 2.2-POUND DENSITY | | | | | | | | | Fc=18psi |
| 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" | 19'-5" | 13'-2" | 12'-6" | 11'-10" | 11'-3" | 10'-3" | 10'-1" | 9'-4" | 8'-3" | 7'-3" |
| | | 4" | 3.5 | 32 | 16'-2" | 13'-5" | 10'-9" | 9'-1" | 8'-0" | 7'-3" | 6'-7" | 6'-1" | 5'-9" | 5'-4" | 19'-9" | 13'-3" | 12'-8" | 12'-1" | 11'-7" | 10'-10" | 10'-4" | 9'-9" | 8'-11" | 8'-3" |
| | | 5" | 4.0 | 40 | 19'-4" | 13'-10" | 12'-2" | 11'-0" | 10'-1" | 9'-5" | 8'-11" | 7'-10" | 7'-6" | 7'-2" | 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 | 16'-7" | 12'-8" | 10'-10" | 9'-6" | 8'-5" | 7'-7" | 6'-10" | 6'-2" | 5'-7" | 5'-1" | 22'-0" | 14'-11" | 14'-2" | 13'-6" | 12'-10" | 11'-11" | 11'-4" | 10'-8" | 9'-7" | 8'-9" |
| | | 4" | 4.0 | 32 | 16'-1" | 12'-6" | 10'-11" | 9'-9" | 8'-9" | 8'-0" | 7'-4" | 6'-10" | 6'-4" | 5'-10" | 22'-3" | 16'-1" | 15'-7" | 14'-11" | 14'-4" | 13'-5" | 12'-11" | 12'-2" | 10'-11" | 9'-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" | 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 | 23'-6" | 17'-11" | 15'-4" | 13'-5" | 12'-3" | 11'-1" | 10'-2" | 9'-4" | 8'-7" | 8'-0" | 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 |

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8-3-17

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# 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 |

# DOWEL BEARING STRENGTH

## DOWEL BEARING STRENGTH IN WOOD - NDS

| *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* |
| $K_{\vartheta}$ | 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 | | $\frac{\sqrt{R_e + 2R_e^2(1 + R_t + R_t^2) + R_t^2 R_e^3} - R_e(1 + R_t)}{(1 + R_e)}$ |
| $k_2$ | 0.662695038 | | $-1 + \sqrt{2(1 + R_e) + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em} l_m^2}}$ |
| $k_3$ | 21.98259487 | | $-1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}(2 + R_e)D^2}{3F_{em} l_s^2}}$ |

| *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* |

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| 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 |

| REFERENCE DESIGN VALUE (LBF) |
|---|
| 2.0 |

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## HILL PHOENIX REPORT

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**

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

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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 |





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