

#20035

Structural Calculations For:
FOO RESIDENCE

AT

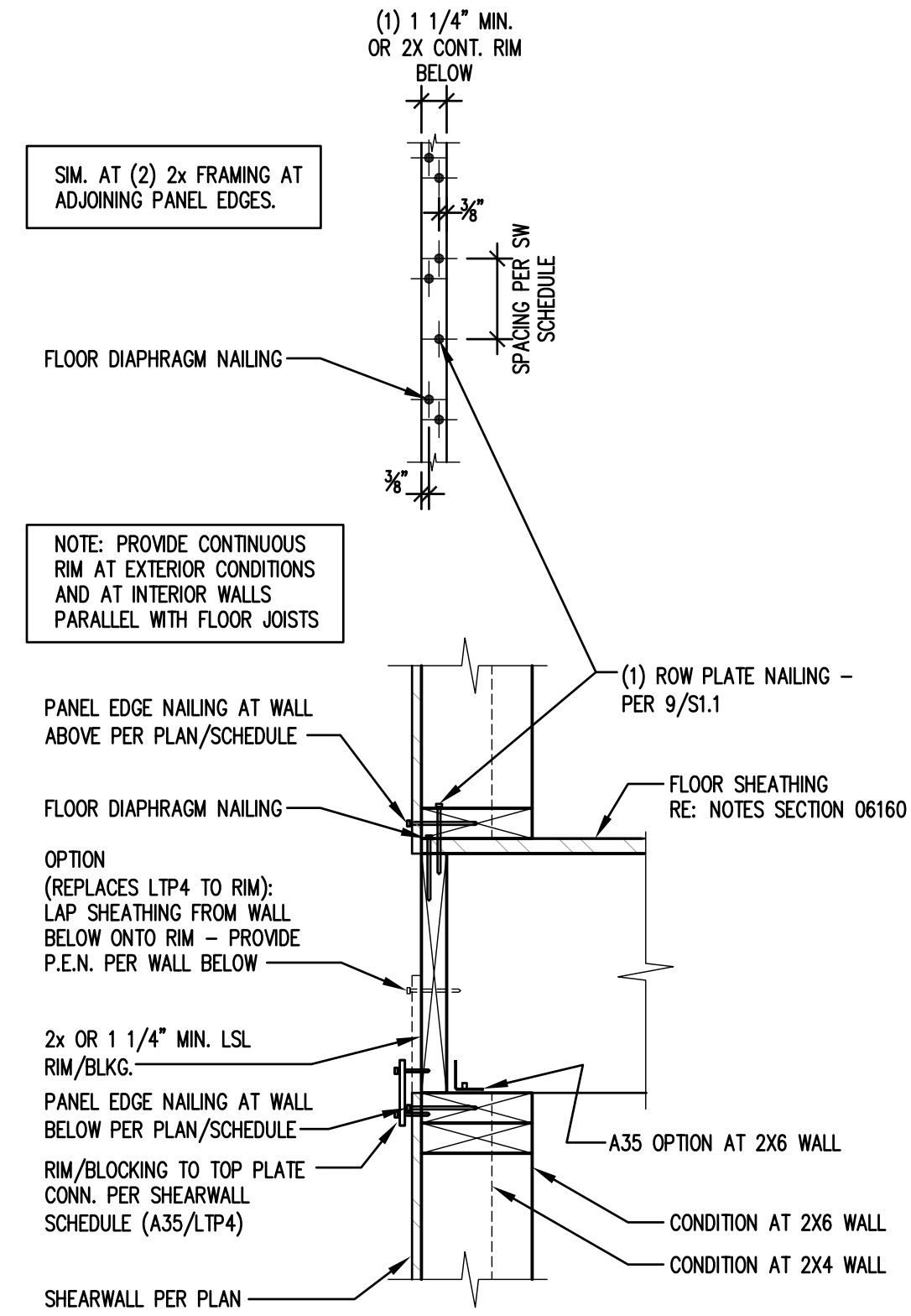
3453 74th Ave SE
Mercer Island, WA 98040



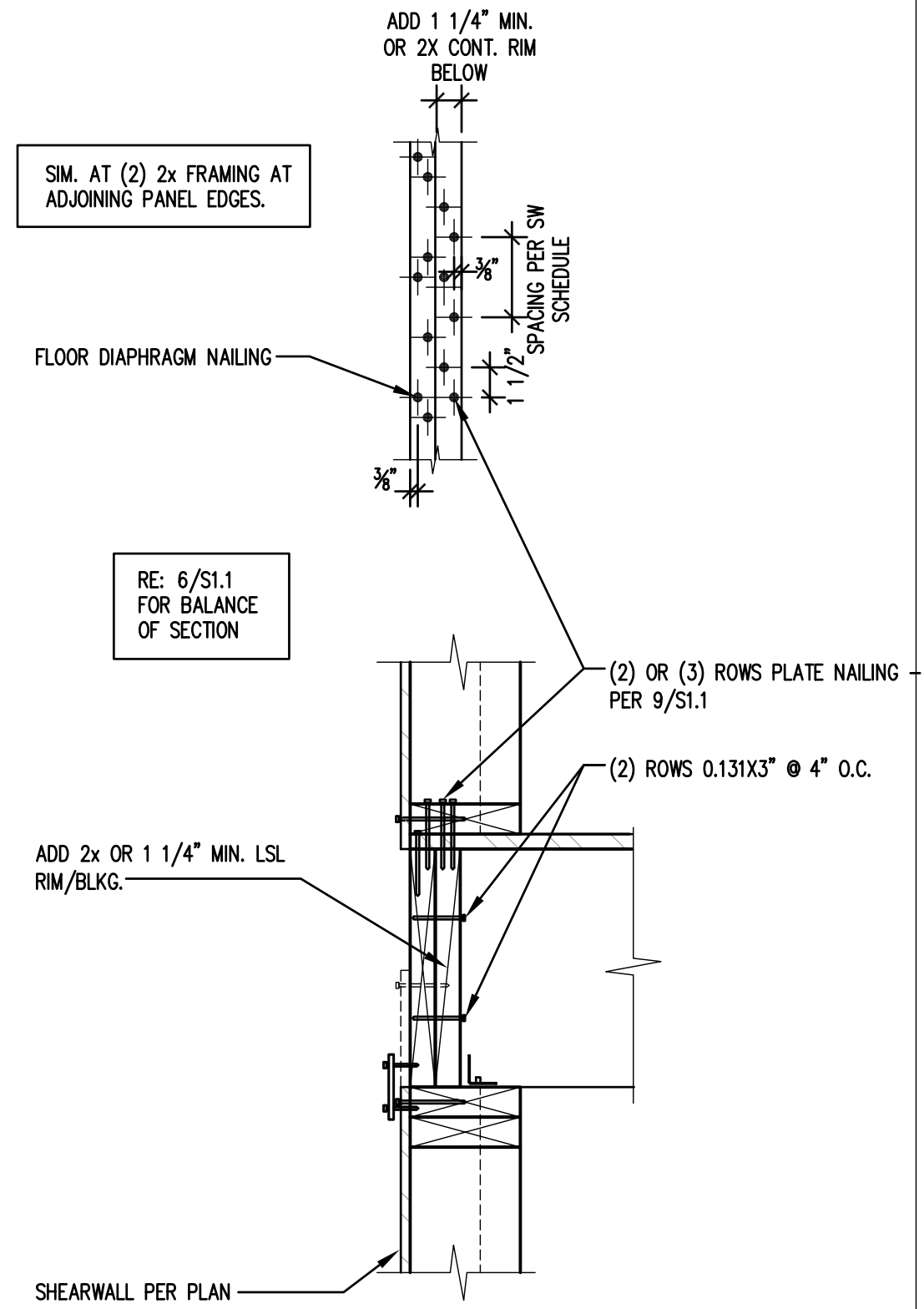
03/16/2021

Client: Jimmy and Shannon Foo
2820 29th Ave. W
Seattle, WA 98199

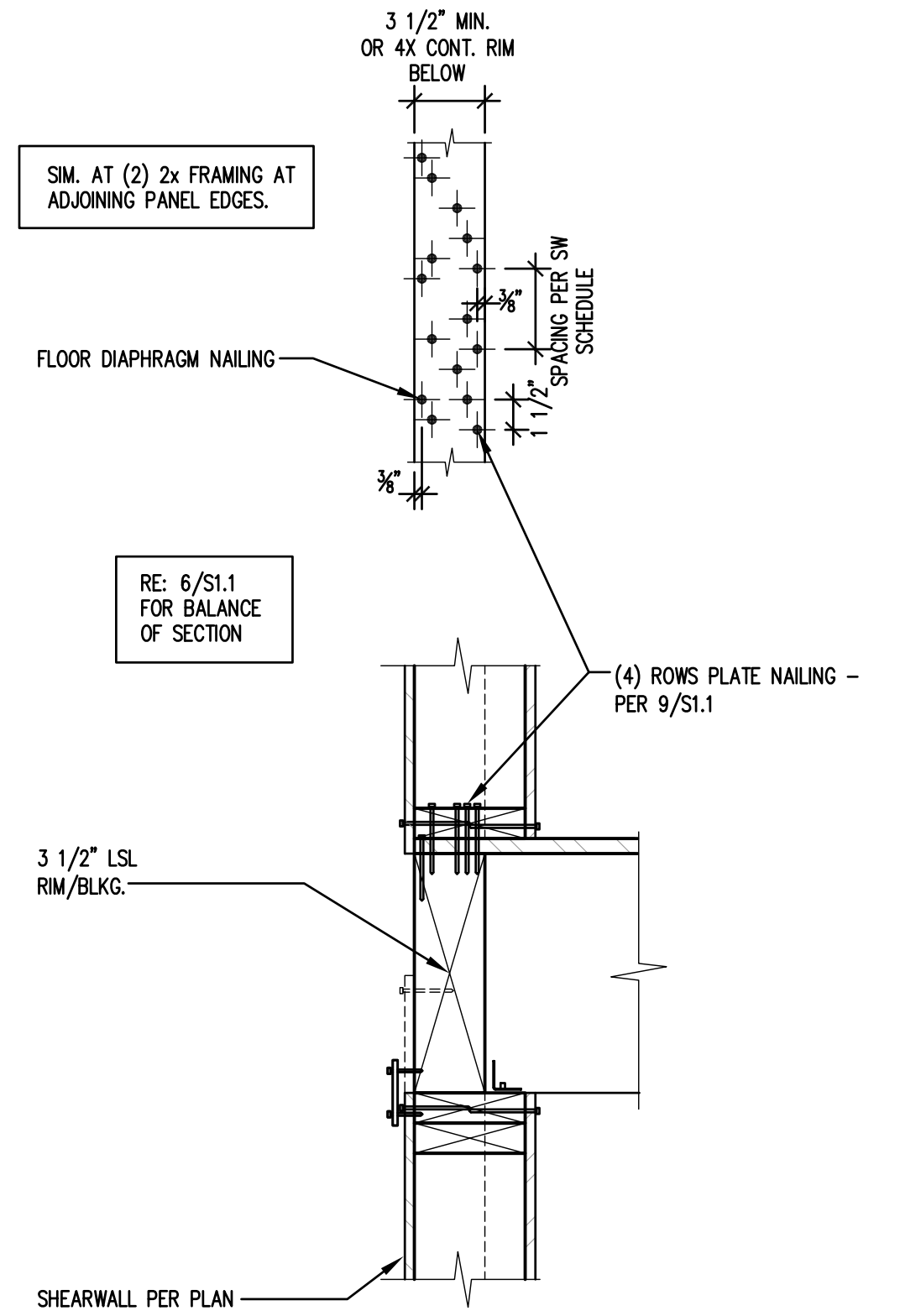
LATERAL SUPPLEMENTAL CALCULATIONS



SCALE: 1 1/2" = 1'-0"
6 P6TN & P6
 TYPICAL SHEARWALL CONNECTION



SCALE: 1 1/2" = 1'-0"
7 P4, P3, & P2
 TYPICAL SHEARWALL CONNECTION

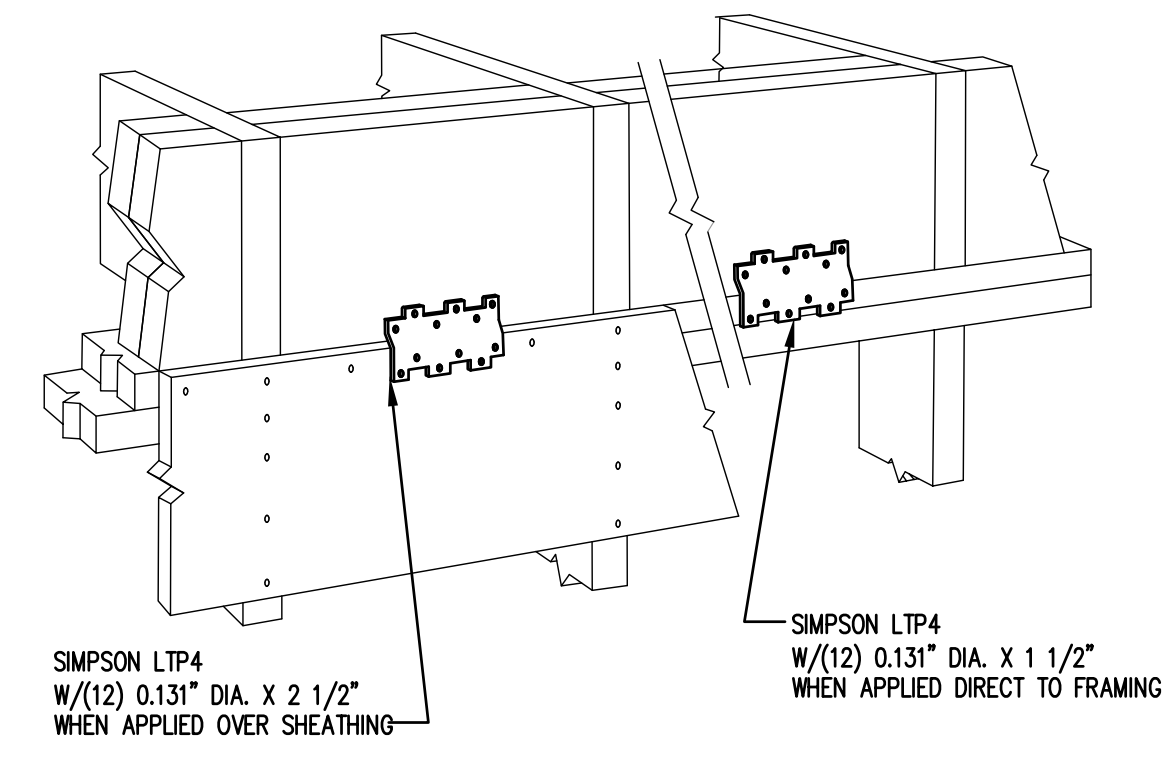


SCALE: 1 1/2" = 1'-0"
8 2P4, 2P3, & 2P2
 TYPICAL SHEARWALL CONNECTION

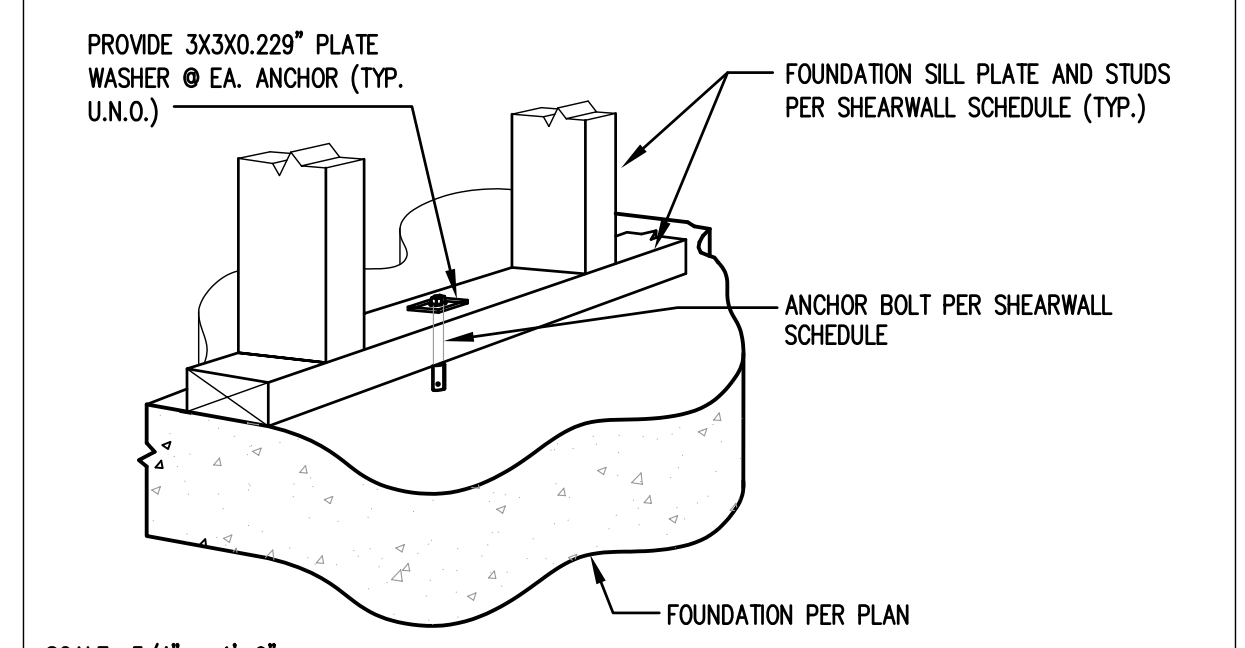
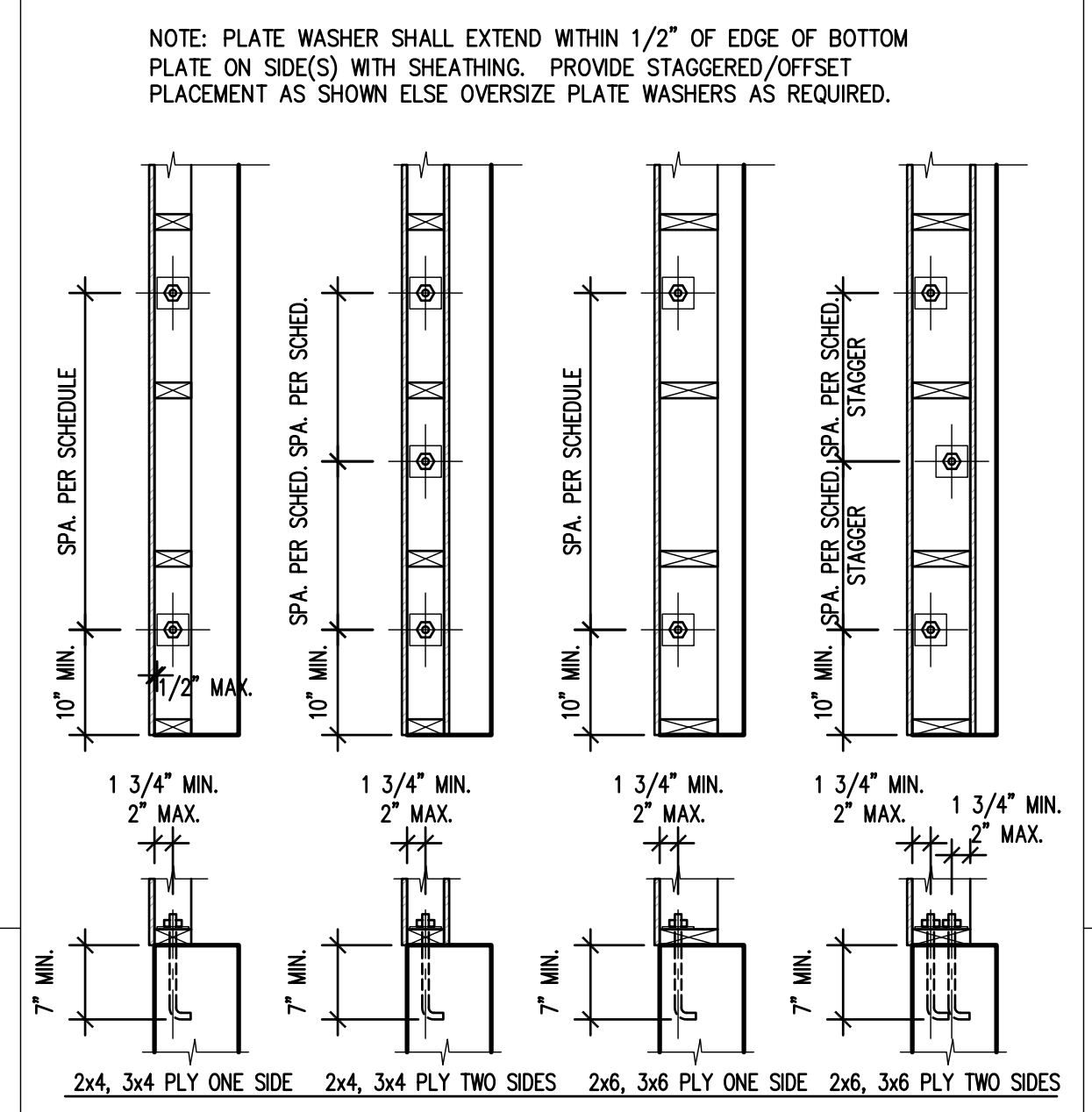
| SHEARWALL TYPE | SHEATHING | PANEL EDGE NAILING | FIELD NAILING | BOTTOM PLATE NAILING | | RIM OR BLOCKING TO TOP PLATE CONNECTION | | FRAMING AT ADJOINING PANEL EDGES | FOUNDATION SILL PLATE TO CONCRETE | 5/8" DIA. ANCHOR BOLT SPACING |
|----------------|----------------------|--------------------|---------------|----------------------|---------|---|------------------------------|----------------------------------|-----------------------------------|-------------------------------|
| | | | | ROWS | SPACING | 0.148"x3.25" TOENAIL | LTP4 (OR A35 OPTION) SPACING | | | |
| P6TN | 3/4" MIN. ONE SIDE | 6" O.C. | 12" O.C. | (1) | 8" O.C. | 4" O.C. | N/A | 2X | 2X | 48" O.C. |
| P6 | 3/4" MIN. ONE SIDE | 6" O.C. | 12" O.C. | (1) | 6" O.C. | N/A | 24" O.C. | 2X | 2X | 48" O.C. |
| P4 | 3/4" MIN. ONE SIDE | 4" O.C. | 12" O.C. | (2) | 6" O.C. | N/A | 16" O.C. | (2) 2X OR 3X | 2X | 36" O.C. |
| P3 | 3/4" MIN. ONE SIDE | 3" O.C. | 12" O.C. | (2) | 5" O.C. | N/A | 12" O.C. | (2) 2X OR 3X | 2X | 30" O.C. |
| P2 | 3/4" MIN. ONE SIDE | 2" O.C. | 12" O.C. | (3) | 6" O.C. | N/A | 10" O.C. | (2) 2X OR 3X | 2X | 24" O.C. |
| 2P4 | 3/4" MIN. BOTH SIDES | 4" O.C. | 12" O.C. | (4) | 6" O.C. | N/A | 16" O.C. EA. SIDE | (2) 2X OR 3X | 2X | 18" O.C. |
| 2P3 | 3/4" MIN. BOTH SIDES | 3" O.C. | 12" O.C. | (4) | 5" O.C. | N/A | 12" O.C. EA. SIDE | (2) 2X OR 3X | 2X | 14" O.C. |
| 2P2 | 3/4" MIN. BOTH SIDES | 2" O.C. | 12" O.C. | (4) | 4" O.C. | N/A | 10" O.C. EA. SIDE | (2) 2X OR 3X | 2X | 10" O.C. |

- SHEARWALL SCHEDULE NOTES:**
- STUDS SHALL NOT BE SPACED MORE THAN 16" O.C.
 - RE: S1.0 SECTION 06100 "ROUGH FRAMING" FOR REQUIRED WALL STUD AND PLATE SPECIES AND GRADE.
 - RE: S1.0 SECTION 06160 "WOOD SHEATHING" FOR REQUIRED SHEAR WALL SHEATHING, THICKNESS AND GRADE. ALL SHEAR WALL PANELS SHALL BE APPLIED DIRECTLY TO FRAMING.
 - SHEATHING PANELS MAY BE INSTALLED EITHER HORIZONTALLY OR VERTICALLY WITH ALL PANEL EDGES BACKED/BLOCKED WITH 2" NOMINAL OR WIDER FRAMING. SEE NOTE 5.
 - FRAMING MEMBERS RECEIVING EDGE NAILING FROM ADJUTING PANELS SHALL NOT BE LESS THAN 3" NOMINAL AND NAILS SHALL BE STAGGERED FOR ALL SHEARWALL MARKS EXCEPT "P6 AND P6TN".
 - WHERE PANELS ARE APPLIED ON BOTH FACES OF A WALL AND NAIL SPACING IS LESS THAN 6" O.C. ON EITHER SIDE, PANEL JOINTS SHALL BE OFFSET TO FALL ON DIFFERENT FRAMING MEMBERS OR FRAMING SHALL BE 3" NOMINAL OR THICKER AND NAILS SHALL BE STAGGERED.
 - NAILS FOR PLYWOOD AND OSB PANEL EDGE AND FIELD NAILING SHALL BE 8d (0.131" X 2 1/4").
 - NAILS FOR BOTTOM PLATE FRAMING SHALL BE 12d COMMON (0.148" X 3.25").
 - FLOOR DIAPHRAGM NAILING SHALL BE PLACED BETWEEN THE SPACING CALLED OUT FOR BOTTOM PLATE NAILING. DO NOT OVER NAIL THE BLOCKING.
 - ANCHOR BOLTS SHALL BE GALVANIZED 5/8" DIAMETER A-307 AND SHALL BE SECURED IN PLACE PRIOR TO CONCRETE POUR.
 - GALVANIZED PLATE WASHERS PER STRUCTURAL NOTES ARE REQUIRED AT EACH ANCHOR BOLT - SEE 17 THIS SHEET FOR PLACEMENT REQUIREMENTS. DO NOT RECESS PLATE WASHERS INTO PLATES.
 - LTP4 FRAMING PLATES SHALL BE INSTALLED WITH 12-8d X 1 1/2" (0.131" X 2 1/2") NAILS. SEE DETAIL 11.
 - A35 FRAMING ANCHORS SHALL BE INSTALLED WITH 12-8d X 1 1/2" (0.131" X 1 1/2") NAILS.
 - ALL NAILS INTO PRESSURE TREATED WOOD SHALL BE HOT-DIPPED GALVANIZED CONFORMING TO ASTM 153 OR STAINLESS STEEL.
 - WHERE BOTTOM PLATE NAILING SPECIFIES A SPACING OF 4 INCHES OR LESS NAILS SHALL BE INSTALLED IN TWO ROWS OFFSET 1/2 INCH AND STAGGERED.
 - GALVANIZED EXPANSION ANCHORS OF SIMILAR DIAMETER AND EMBEDMENT ALLOWED AT INTERIOR BEARING AND PARTY WALLS.
 - 2-2x5'S IN LIEU OF 3x'S AT PANEL EDGES ACCEPTABLE PROVIDED STUDS ARE FACE NAILED TOGETHER W/2) 0.131" DIA. X 3" NAILS AT 6" O.C..
 - WHERE BUILDING OFFICIALS ALLOW, OSB SHEATHING MAY BE APPLIED OVER 1/2" OR 3/8" GYPSUM WALL BOARD PROVIDED SHEATHING IS NAILED WITH 10d NAILS (0.148" DIA X 3" LONG)

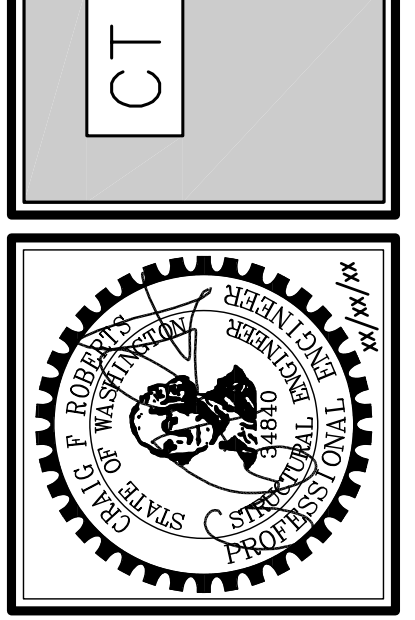
SCALE: NONE
9 SHEARWALL SCHEDULE



SCALE: NONE
11 TYPICAL SIMPSON LTP4



SCALE: 3/4" = 1'-0"
17 ANCHOR BOLT PLACEMENT DETAILS



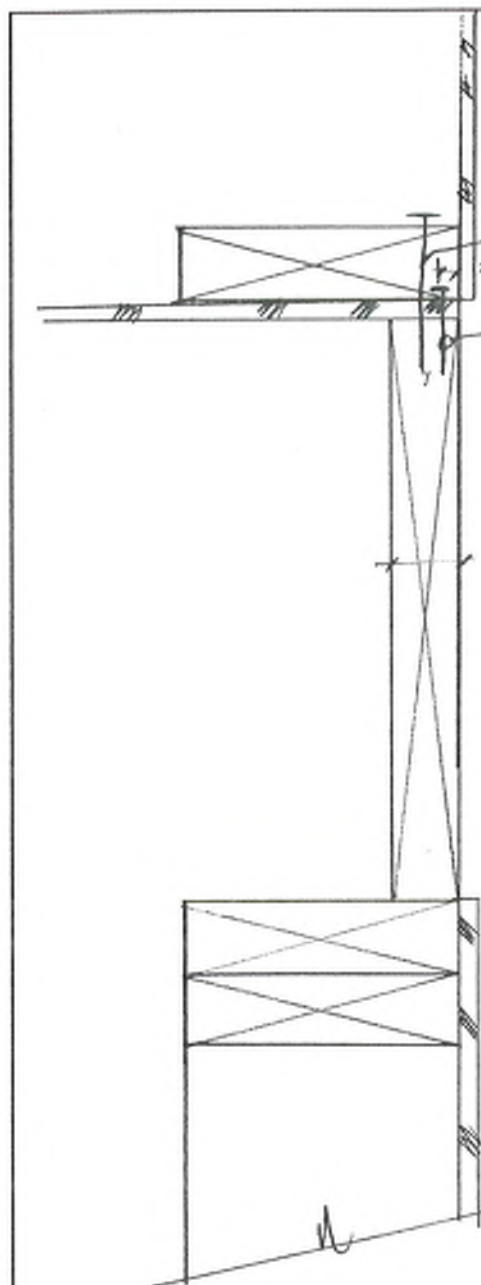
| | | | | | | |
|-----|------|----------|------|----|------------------|---------|
| NO. | DATE | REVISION | DATE | BY | FILE NO. | JOB NO. |
| | | | | | xx/xx/xx | xxx |
| | | | | | PERMIT SUBMITTAL | |
| | | | | | CADD/DWH | |
| | | | | | PLOT: xx/xx/xx | |
| | | | | | ENGFR: xxx | |

Project: SHOAR WALL PLATE NAILING

Date: MAY 2018

Client: _____

Page Number: _____



w/ 3/4" LONG NAIL - CHECK
PENETRATION -

$$\begin{array}{r} 3/4" \\ - 1/12" \text{ PLATE} \\ \hline 1 3/4" \text{ :OK} \\ \hline \end{array}$$

PLATE NAIL
3/8"
DIA. NAIL
AT C.O.D.

1/4" MIN

IF DO NOT INCLUDE 3/4" PLT'D;
ONLY 1" PENETRATION.

∴ MUST USE REDUCTION -

$$1/1.5" = 0.67$$

FOR HOM FIX -

$$163^*/\text{NAIL} \times 0.67 = \underline{\underline{109^*/\text{NAIL}}}$$

FOR DDBL FIX -

$$189^*/\text{NAIL} \times 0.67 = 127^*/\text{NAIL}$$

$$(188.8) \left(\frac{1}{1.5} \right) = 127.5 \checkmark$$

~~$$C_0 = (127.5)(1.6) =$$~~

Fastener Spacing in Weyerhaeuser Engineered Lumber Products

This technical bulletin provides fastener spacing and placement information for Weyerhaeuser engineered lumber products. It is intended to supplement what is included in Weyerhaeuser's code evaluation reports and product literature. Specifically, the document provides recommended on-center spacing and minimum end distances for fasteners in continuous patterns (i.e. diaphragm nailing). These guidelines do not apply to conditions including joist hangers, straps, and nailing of TJI® joists at bearing locations or other localized nailing applications. As with any connection in wood or wood based material, avoiding unacceptable splitting often dictates fastener spacing and placement. The recommendations given in the following tables are based on preventing splitting that propagates from fastener to fastener within the connection. Splitting can be reduced by installing nails at slight angles and by using staggered or offset patterns.

The following tables provide general guidelines for fastener spacing as well as information relevant to determining the capacity of fastener connections. For additional information regarding TJI® joists, reference *Specifier's Guide for TJI® Joists* (TJ-4000 \ Canada-East (s-Series): TJ-4510 \ Canada-West: TJ-4500). For additional information regarding structural composite lumber, reference *Specifier's Guide for Trus Joist® Beams, Headers, and Columns* (TJ-9000 \ Pacific Coast and Northwest: TJ-9020 \ Canada-East: TJ-9500 \ Canada-West: TJ-9505). For other applications, see *Attaching Fire Sprinkler Components to Weyerhaeuser Engineered Wood Products* (TB-203). If closer on-center spacing patterns are required, please consult your Weyerhaeuser representative.

TABLE 1: TJI® JOISTS, RIM JOISTS, AND BLOCKING PANELS⁽¹⁾⁽²⁾⁽³⁾⁽⁴⁾

| Nail Size ⁽⁵⁾ | | Nails into Wide Face of Flange ⁽⁶⁾ | | | | Nails into Narrow Edge of Flange ⁽⁷⁾ | | | |
|---|----------|---|-------------------|---------------------------------------|-------------------|---|-------------------|---------------------------------------|-------------------|
| | | TJI® Joist Series | | | | TJI® Joist Series | | | |
| | | 110 \ 210 \ 230 | | 360 \ 560 \ 560D \ s31 \ s33 \ s47 | | 110 \ 210 \ 230 | | 360 \ 560 \ 560D \ s31 \ s33 \ s47 | |
| | | On-Center Spacing | Min. End Distance | On-Center Spacing | Min. End Distance | On-Center Spacing | Min. End Distance | On-Center Spacing | Min. End Distance |
| 6d (2") common & 8d (2 1/2") box | (0.113") | 4" | 2 1/2" | 3" | 2" | 6" | 6" | 3" | 4" |
| 8d (2 1/2") common | (0.131") | 4" | 2 1/2" | 3" | 2" | 6" | 6" | 6" | 6" |
| 8d (1 1/2") N8 or NA11 ⁽⁸⁾ | (0.131") | 3" | 2 1/2" | 3" | 2" | 6" | 6" | 6" | 6" |
| 10d (3") box | (0.128") | 4" | 3" | 3" | 2" | 6" | 6" | 5" | 5" |
| 12d (3 1/4") box | (0.128") | 4" | 3" | 3" | 2" | 6" | 6" | 5" | 5" |
| 10d (3") common | (0.148") | 4" | 4 1/2" | 4" | 3" | 6" | 6" | 6" | 6" |
| 12d (3 1/4") common | (0.148") | 4" | 4 1/2" | 4" | 3" | 6" | 6" | 6" | 6" |
| 10d (1 1/2") N10 or NA9D ⁽⁸⁾ | (0.148") | 3" | 4 1/2" | 3" | 3" | 6" | 6" | 6" | 6" |
| 16d (3 1/2") box | (0.135") | 4" | 4 1/2" | 4" | 3" | 6" | 6" | 6" | 6" |
| 16d (3 1/4") sinker | (0.148") | 4" | 4 1/2" | 4" | 3" | 6" | 6" | 6" | 6" |
| 16d (3 1/2") common | (0.162") | 6" | 6" | 6" | 4" | Not Recommended | | | |
| Framing Angles: A34, A35, LTP4, LTP5, MP34, MPA1, MPA1F, and MP4F | | N/A | | N/A | | Not Recommended | | | |

[1] Fastener spacings in this table may be used for wood screws provided the lengths and root diameters are less than or equal to the nail sizes listed in the table. Always use screws intended for structural assembly of wood structures. Drywall screws should never be used since they tend to be brittle and may easily break.

[2] Includes attachment of the bottom flange of TJI® rim joists and blocking panels to the wall plate below.

[3] Recommended edge distance is 1/2" for TJI® 110 joists and 3/4" for all other TJI® joist series (does not apply to diaphragm construction, see note 6).

[4] Maximum spacing of nails should not exceed lateral stability requirements. See applicable literature.

[5] Length of nail shown in parentheses () ; diameter of nail shown in brackets [].

[6] One row of nails permitted (two at abutting panel edges) for diaphragms. Stagger nails when using 4" on-center spacing or less and maintain 1/4" joist and panel edge distance. For other applications, multiple rows of fasteners are permitted if the rows are offset at least 1/4" and staggered.

[7] One (1) row of nails only.

[8] Nail spacing values shown are intended for use with only light-gauge steel straps. Multiple rows of nails must be offset at least 1/4" and staggered.

TABLE 2: STRUCTURAL COMPOSITE LUMBER⁽¹⁾

| Nail Size | | Nails Into Wide Face (Perpendicular to Strands) | | Nails Into Narrow Edge (Parallel to Strands) | | | | | | Min. End Distance |
|---|----------|--|--|--|------------------|---|-------------------|-------------------|-------------------|----------------------|
| | | Microllam® LVL, Parallam® PSL | TimberStrand® LSL, TJ® Rim Board | Microllam® LVL | Parallam® PSL | TimberStrand® LSL ⁽²⁾ , TJ® Rim Board | | | | |
| | | | | | | 1.3 1 1/2" | 1.55 1 1/4" | 1 1/2" | 1.55 1 3/4" | |
| | | On-Center Spacing | | On-Center Spacing | | | | | | |
| 8d (2") common & 8d (2 1/2") box | [0.113"] | 2" | 2" | 3" | 3" | 6" | 4" | 3" | 3" | 2 1/2" |
| 8d (2 1/2") common | [0.131"] | 2" | 2" | 4" | 4" | 6" | 4" | 3" | 3" | 2 3/4" |
| 8d (1 1/2") N8 or NA11 | [0.131"] | 2" | 2" | 4" | 4" | 6" | 4" | 3" | 3" | 2 3/4" |
| 10d (3") box | [0.128"] | 2" | 2" | 4" | 4" | 6" | 4" | 3" | 3" | 2 3/4" |
| 12d (3 1/4") box | [0.128"] | 2" | 2" | 4" | 4" | 6" | 4" | 3" | 3" | 2 3/4" |
| 10d (3") common | [0.148"] | 3" | 2 1/2" | 5" | 4" | 6" | 4" | 3" | 3" | 3" |
| 12d (3 1/4") common | [0.148"] | 3" | 2 1/2" | 5" | 4" | 6" | 4" | 3" | 3" | 3" |
| 10d (1 1/2") N10 or NA9D | [0.148"] | 3" | 2 1/2" | 5" | 4" | 6" | 4" | 3" | 3" | 3" |
| 16d (3 1/2") box | [0.135"] | 3" | 2 1/2" | 5" | 4" | 16" ⁽⁷⁾ | 4" | 3" | 3" | 2 3/4" |
| 16d (3 1/4") sinker | [0.148"] | 3" | 2 1/2" | 5" | 4" | 16" ⁽⁷⁾ | 4" | 3" | 3" | 3" |
| 16d (3 1/2") common | [0.162"] | 4" | 3" | 8" ⁽⁸⁾ | 6" | 16" ⁽⁸⁾ | 6" ⁽⁸⁾ | 6" ⁽⁸⁾ | 6" ⁽⁸⁾ | 3 1/4" |
| Proprietary Wood Screws ^(9,10) | [0.250"] | - (11) | | - (10) | - (10) | N/A | - (10) | | - (11) | - (11) |
| Framing Angles: A34, A35, LTP4, LTP5, MP34, MPA1, MPA1F, and MP4F | | OK | | N/A | | N/A | N/A | | N/A | 2 3/4" |

(1) See General Notes on page 3.

(2) Closest on-center edge nailing for StrandGuard® TimberStrand® LSL sill plates is one (1) row at 4" o.c. for 2x (1 1/2" thick).

(3) Can be reduced to 5" o.c. with maximum nail penetration of 1 1/2" into narrow edge (e.g. nails that connect sole plate above to block or rim).

(4) Can be reduced to 4" o.c. with maximum nail penetration of 1 1/4" into narrow edge (e.g. nails that connect sole plate above to block or rim).

(5) Can be reduced to 3 1/2" o.c. with maximum nail penetration of 1 1/4" into narrow edge (e.g. nails that connect sole plate above to block or rim).

(6) Proprietary wood screws are Simpson Strong-Tie® SDS and SDW, USP® WS, and FastenMaster® TrussLok® structural wood screws; nominal fastener shank diameter = 0.25".

(7) 6" long USP® WS structural wood screws are not recommended for TimberStrand® LSL or Parallam® PSL.

(8) Space proprietary wood screws at 6" o.c. minimum, into the narrow edge.

(9) Two (2) staggered rows of proprietary wood screws are permitted in the narrow edge of Parallam® PSL for members 3 1/2" thick. Three (3) staggered rows of proprietary wood screws are permitted in the narrow edge of Parallam® PSL for members greater than or equal to 5 1/4" thick. For multiple rows, edge distance is a minimum of 1" and spacing between staggered rows is a minimum of 1 1/2".

(10) One (1) row of proprietary wood screws is permitted in the narrow edge of TimberStrand® LSL for members 1 1/2", 1 1/4", and 1 1/2" thick. Two (2) staggered rows of proprietary wood screws are permitted in the narrow edge of TimberStrand® LSL for members 3 1/2" thick. For multiple rows, edge distance is a minimum of 1" and spacing between staggered rows is a minimum of 1 1/2".

(11) See screw manufacturer's recommendations for spacing and capacity of connections. End distances, edge distances, and capacity of the screws must be sufficient to minimize splitting.

Fastener spacing not applicable for shear wall applications. See appropriate code report for grade specific TimberStrand® LSL nailing requirements.

General Notes for Table 2

- Fastener spacings in this table may be used for wood screws provided the lengths and root diameters are less than or equal to the nail sizes listed in the table. Always use screws intended for structural assembly of wood structures. Drywall screws should never be used since they tend to be brittle and may easily break.
- Maximum permissible rows are two (2) for 1 3/4" and 1 1/2" thicknesses, three (3) for 1 3/4" thickness, and six (6) for thicknesses greater than or equal to 3 1/2".
- To minimize splitting, member edge distance and spacing between rows shall be the greater of (2.5 x nail diameter) or 3/8". Where multiple rows are used, fasteners in adjacent rows must be staggered and the rows must be equally spaced from the centerline of the narrow face axis.
- Slant sheathing nails to maintain minimum required structural composite lumber edge distance.
- Length of nail shown in parenthesis (). Diameter of nail shown in brackets [].
- To determine connection capacities for applications such as TimberStrand® LSL shearwalls, reference Table 3 below.

TABLE 3: EQUIVALENT SPECIFIC GRAVITY FOR CONNECTION DESIGN^[1]

| Product | Lateral | | Withdrawal | | Shearwalls ^{[2][3]} |
|-------------------------------|---------|---------------------|------------|---------------|------------------------------|
| | Face | Edge | Face | Edge | |
| MicroIam® LVL \ Parallam® PSL | 0.50 | 0.50 | 0.50 | 0.50 | Not Evaluated |
| 1.3E TimberStrand® LSL | 0.50 | 0.50 ^[4] | 0.50 | 0.42 | 0.42 ^[5] |
| 1.5E TimberStrand® LSL | 0.50 | 0.50 ^[4] | 0.50 | 0.42 | 0.42 |
| 1.55E TimberStrand® LSL | 0.50 | 0.50 ^[4] | 0.50 | 0.42 | 0.42 |
| 1.6E TimberStrand® LSL | 0.50 | 0.50 ^[4] | 0.50 | 0.42 | 0.50 |
| 1 1/2" TJ® Rim Board | 0.50 | Not Evaluated | 0.42 | Not Evaluated | Not Evaluated |

[1] Specific gravity of 0.55 is equivalent to Southern Pine; 0.50 is equivalent to Douglas-Fir; 0.42 is equivalent to Spruce-Pine-Fir.

[2] For US, design shearwall applications per ANSI/AWC SDPWS-2015. If equivalent specific gravity is equal to 0.42, multiply values in Table 4.3A by 0.92.

- For Canada, design shearwall applications per CCMC Report No. 12627-B.

[3] When StrandGuard® TimberStrand® LSL sill plates are used in shearwall construction, use the specific gravity of the studs (up to 0.50) when determining the allowable shear.

[4] Specific gravity for proprietary wood screws installed into the edge of TimberStrand® LSL for lateral connections is 0.42.

[5] As per ICC-ES ESR-1387 and CCMC Report No. 12627-B, minimum boundary nail spacing must be 6" o.c. Studs at boundary locations, where two panels abut, are allowed two (2) rows @ 6" o.c.

TABLE 4: COMMON ADJUSTMENT FACTORS FOR CONNECTIONS^[1]

| Property | United States | | | Canada | | |
|--------------------------------------|---------------|---------|------------|----------|---------|------------|
| | Notation | Lateral | Withdrawal | Notation | Lateral | Withdrawal |
| Duration of Load (Live Load) | C_d | 1.00 | 1.00 | K_o | 1.00 | 1.00 |
| Duration of Load (Snow Load) | C_d | 1.15 | 1.15 | K_o | 1.00 | 1.00 |
| Duration of Load (Construction Load) | C_d | 1.25 | 1.25 | K_o | 1.15 | 1.15 |
| Duration of Load (Wind/Seismic Load) | C_d | 1.60 | 1.60 | K_o | 1.15 | 1.15 |
| End Grain Factor | C_{eg} | 0.67 | N/A | J_E | 0.67 | N/A |
| Diaphragm Factor | C_{di} | 1.10 | N/A | J_D | 1.30 | N/A |
| Toe-nail Factor | C_{tn} | 0.83 | 0.67 | J_A | 0.83 | 0.67 |

Project: SHOARMAR PLATE

Date: MAY 2018

Client: _____

Page Number: _____

ANCHOR BOLT SPACING:

TABLE 11E NDS -

1 1/2" SIDE PLATES (2x)

DF 930# / BOLT 2 // * 1.6 = 1488#

HF 860# / BOLT 2 // * 1.6 = 1376#

WBS CM

DURATION CD

| | <u>DF</u> | <u>HTM F12</u> |
|-----|-----------|----------------|
| P0 | 48" | 48" |
| P4 | 38" | 38" |
| P3 | 30" | |
| P2 | 20" | |
| 2P4 | 20" | |
| 213 | 14" | |
| 2P2 | 10" | |

SHEET TITLE: IBC SHEARWALLS: PER ANS/A/P&PA SDPWS Table 4.3A
 CT PROJECT #: TYPICAL

SHEATHING THICKNESS: $t_{sheathing} = 15/32"$ (7/16")
 NAIL SIZE: nail size = 0.131" dia. X 2.5" long
 STUD SPECIES: SPECIES = H-F or SPF
 SPECIFIC GRAVITY: S.G. = 0.43
 ANCOR BOLT DIAMETER: Anc. Bolt dia. = 0.625
 ASD F.O.S. = 2.0

0.148X3.25_{plate nail} = 110.00

5/8" DIA. anchor bolt = 1360.00 1695.00

LTP4 550.00
 575 A35

SHEARWALL TYPE: 7/16" w8d common
 (15/32" values per footnote 2)

Table 4.3A
 $V_{seismic}$
 (SDPWS)

Seismic
 V_s allowable
 modify per S. G.
 (divide by 2.0 FOS)
 (for ASD)

Table 4.3A
 V_{wind}
 (SDPWS)

Wind
 V_w allowable
 modify per S. G.
 (divide by 2.0 FOS)
 (for ASD)

One Row Spacing Two Row Spacing Three Row Spacing Four Row Spacing
 PLATE NAIL PLATE NAIL PLATE NAIL PLATE NAIL

2X plate max AB spacing
 3X plate max AB spacing

LTP4 spacing

| | 0 | 1 | 0 | 1 | One Row Spacing PLATE NAIL | Two Row Spacing PLATE NAIL | Three Row Spacing PLATE NAIL | Four Row Spacing PLATE NAIL | 2X plate max AB spacing | 3X plate max AB spacing | LTP4 spacing |
|------|-------|------|-------|------|----------------------------|----------------------------|------------------------------|-----------------------------|-------------------------|-------------------------|--------------|
| P6TN | 150 | 150 | 150 | 150 | 8.80 | 17.60 | 26.40 | 35.20 | 108.8 | 135.7 | 44.0 |
| P6 | 520 | 242 | 730 | 339 | 5.48 | 10.92 | 16.38 | 21.84 | 67.5 | 84.2 | 27.3 |
| P4 | 760 | 353 | 1065 | 496 | 3.74 | 7.47 | 11.21 | 14.94 | 46.2 | 57.6 | 18.7 |
| P3 | 980 | 456 | 1370 | 637 | 2.90 | 5.79 | 8.69 | 11.59 | 35.8 | 44.7 | 14.5 |
| P2 | 1280 | 595 | 1790 | 832 | 2.22 | 4.44 | 6.65 | 8.87 | 27.4 | 34.2 | 11.1 |
| 2P4 | 1520 | 707 | 2130 | 990 | 1.87 | 3.74 | 5.60 | 7.47 | 23.1 | 28.8 | 9.3 |
| 2P3 | 1960 | 911 | 2740 | 1274 | 1.45 | 2.90 | 4.34 | 5.79 | 17.9 | 22.3 | 7.2 |
| 2P2 | 2560 | 1190 | 3580 | 1665 | 1.11 | 2.22 | 3.33 | 4.44 | 13.7 | 17.1 | 5.5 |
| N.G. | 10000 | 4650 | 10000 | 4650 | | | | | | | |

SHEATHING THICKNESS: $t_{sheathing} = 15/32"$ (7/16")
 NAIL SIZE: nail size = 0.131" dia. X 2.5" long
 STUD SPECIES: SPECIES = DF
 SPECIFIC GRAVITY: S.G. = 0.50
 ANCOR BOLT DIAMETER: Anc. Bolt dia. = 0.625
 ASD F.O.S. = 2.0

0.148X3.25_{plate nail} = 110.00

5/8" DIA. anchor bolt = 1488.00 1888.00

LTP4 670.00

SHEARWALL TYPE: 7/16" w8d common
 (15/32" values per footnote 2)

Table 4.3A
 $V_{seismic}$
 (SDPWS)

Seismic
 V_s allowable
 modify per S. G.
 (divide by 2.0 FOS)
 (for ASD)

Table 4.3A
 V_{wind}
 (SDPWS)

Wind
 V_w allowable
 modify per S. G.
 (divide by 2.0 FOS)
 (for ASD)

One Row Spacing Two Row Spacing Three Row Spacing Four Row Spacing
 PLATE NAIL PLATE NAIL PLATE NAIL PLATE NAIL

2X plate max AB spacing
 3X plate max AB spacing

LTP4 spacing

| | 0 | 1 | 0 | 1 | One Row Spacing PLATE NAIL | Two Row Spacing PLATE NAIL | Three Row Spacing PLATE NAIL | Four Row Spacing PLATE NAIL | 2X plate max AB spacing | 3X plate max AB spacing | LTP4 spacing |
|------|-------|------|-------|------|----------------------------|----------------------------|------------------------------|-----------------------------|-------------------------|-------------------------|--------------|
| P6TN | 150 | 150 | 150 | 150 | 8.80 | 17.60 | 26.40 | 35.20 | 119.0 | 151.0 | 53.8 |
| P6 | 520 | 260 | 730 | 365 | 5.08 | 10.15 | 15.23 | 20.31 | 68.7 | 87.1 | 30.9 |
| P4 | 760 | 380 | 1065 | 533 | 3.47 | 6.95 | 10.42 | 13.89 | 47.0 | 59.6 | 21.2 |
| P3 | 980 | 490 | 1370 | 685 | 2.69 | 5.39 | 8.08 | 10.78 | 35.4 | 46.2 | 16.4 |
| P2 | 1280 | 640 | 1790 | 895 | 2.06 | 4.13 | 6.19 | 8.25 | 27.9 | 35.4 | 12.6 |
| 2P4 | 1520 | 760 | 2130 | 1065 | 1.74 | 3.47 | 5.21 | 6.95 | 23.5 | 29.8 | 10.6 |
| 2P3 | 1960 | 980 | 2740 | 1370 | 1.35 | 2.69 | 4.04 | 5.39 | 18.2 | 23.1 | 8.2 |
| 2P2 | 2560 | 1280 | 3580 | 1790 | 1.03 | 2.06 | 3.09 | 4.13 | 14.0 | 17.7 | 6.3 |
| N.G. | 10000 | 5000 | 10000 | 5000 | | | | | | | |

SHEATHING THICKNESS: $t_{sheathing} = 15/32"$ (7/16")
 NAIL SIZE: nail size = 0.148" dia. X 2.5" long
 STUD SPECIES: SPECIES = DF
 SPECIFIC GRAVITY: S.G. = 0.50
 ANCOR BOLT DIAMETER: Anc. Bolt dia. = 0.625
 ASD F.O.S. = 2.0

0.148X3.25_{plate nail} = 127.50

5/8" DIA. anchor bolt = 1488.00 1888.00

LTP4 670.00 ✓

SHEARWALL TYPE: 7/16" w10d common
 (15/32" values per footnote 2)

Table 4.3A
 $V_{seismic}$
 (SDPWS)

Seismic
 V_s allowable
 modify per S. G.
 (divide by 2.0 FOS)
 (for ASD)

Table 4.3A
 V_{wind}
 (SDPWS)

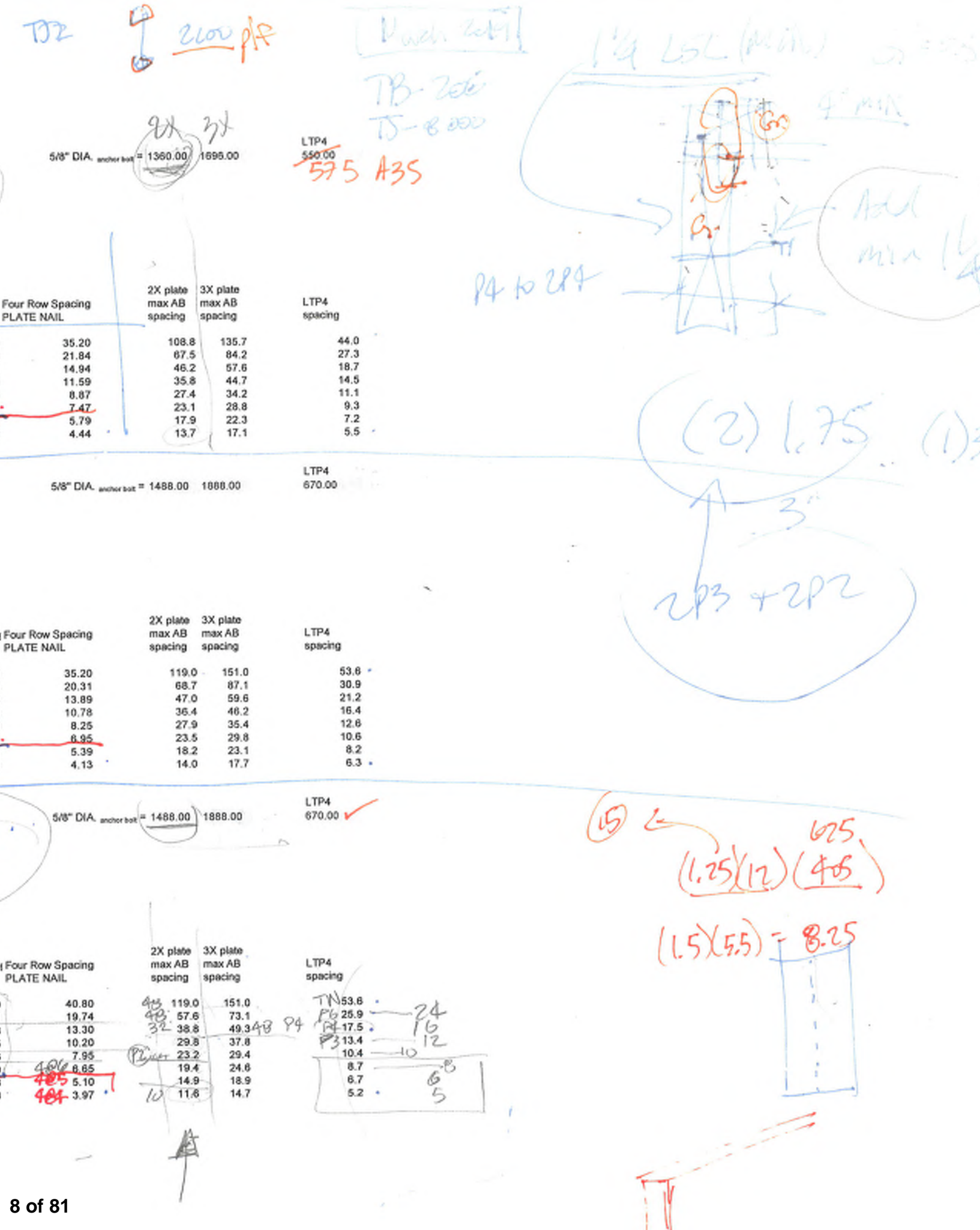
Wind
 V_w allowable
 modify per S. G.
 (divide by 2.0 FOS)
 (for ASD)

One Row Spacing Two Row Spacing Three Row Spacing Four Row Spacing
 PLATE NAIL PLATE NAIL PLATE NAIL PLATE NAIL

2X plate max AB spacing
 3X plate max AB spacing

LTP4 spacing

| | 0 | 1 | 0 | 1 | One Row Spacing PLATE NAIL | Two Row Spacing PLATE NAIL | Three Row Spacing PLATE NAIL | Four Row Spacing PLATE NAIL | 2X plate max AB spacing | 3X plate max AB spacing | LTP4 spacing |
|------|-------|------|-------|------|----------------------------|----------------------------|------------------------------|-----------------------------|-------------------------|-------------------------|--------------|
| P6TN | 150 | 150 | 150 | 150 | 10.20 | 20.40 | 30.60 | 40.80 | 119.0 | 151.0 | 53.8 |
| P6 | 620 | 310 | 870 | 435 | 4.94 | 9.87 | 14.81 | 19.74 | 68.7 | 87.1 | 30.9 |
| P4 | 920 | 460 | 1290 | 645 | 3.33 | 6.65 | 9.98 | 13.30 | 47.0 | 59.6 | 21.2 |
| P3 | 1200 | 600 | 1680 | 840 | 2.55 | 5.10 | 7.65 | 10.20 | 35.4 | 46.2 | 16.4 |
| P2 | 1540 | 770 | 2155 | 1078 | 1.99 | 3.97 | 5.95 | 7.95 | 27.9 | 35.4 | 12.6 |
| 2P4 | 1840 | 920 | 2580 | 1290 | 1.66 | 3.33 | 4.99 | 6.65 | 23.5 | 29.8 | 10.6 |
| 2P3 | 2400 | 1200 | 3360 | 1680 | 1.28 | 2.55 | 3.83 | 5.10 | 18.2 | 23.1 | 8.2 |
| 2P2 | 3080 | 1540 | 4310 | 2155 | 0.99 | 1.99 | 2.98 | 3.97 | 14.0 | 17.7 | 6.3 |
| N.G. | 10000 | 5000 | 10000 | 5000 | | | | | | | |



Project: Foo RESIDENCE

Date:

Client:

Page Number:

TOP PLATE LOAD CALC

WIND GOVERNS → LOOK AT WIND LOADS

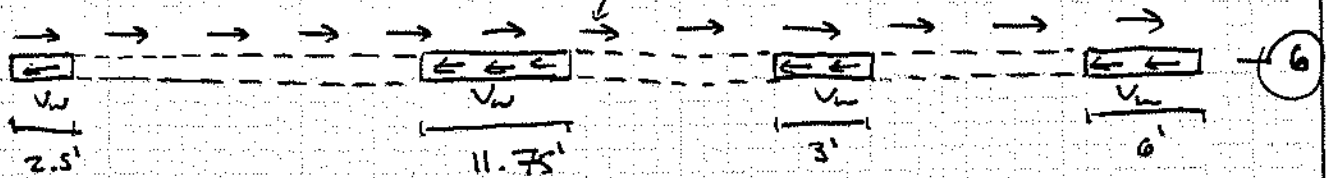
• MAX FORCE ALONG D GRID 6

TOTAL LOAD ALONG GRID = 10.66 K

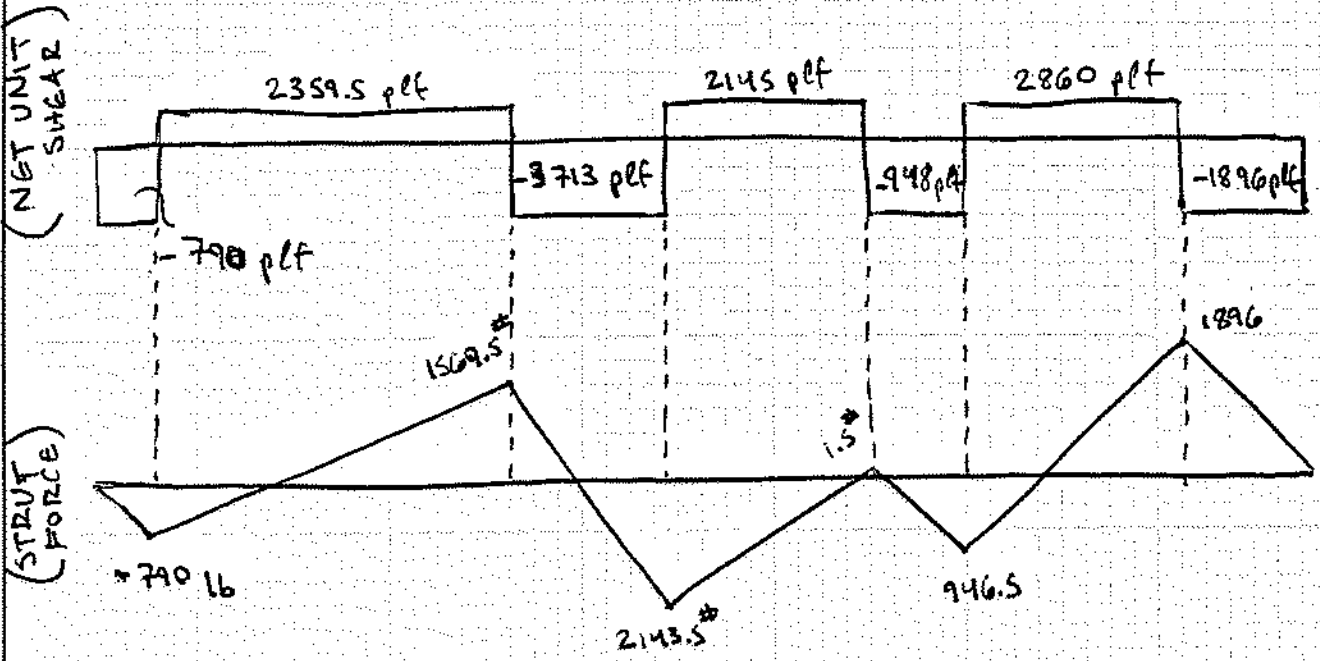
TOTAL LENGTH WALL = ~~74.75'~~ 74.75'

TOTAL LENGTH SHEAR WALL = 23.25'

$$V_T = \frac{10.66^k}{74.75'} = 143 \text{ plf}$$



$$V_w = \frac{10.66^k}{23.25} = 459 \text{ plf}$$



MAX STRUT FORCE = 2144#

(20) 16d common capacity = (20)(122#) = 2440#

DETAIL OK

Wood Beam

File: beam check.ec6
Software copyright ENERCALC, INC. 1983-2020, Build:12.20.8.24
C.T. ENGINEERING

Lic. #: KW-06002997

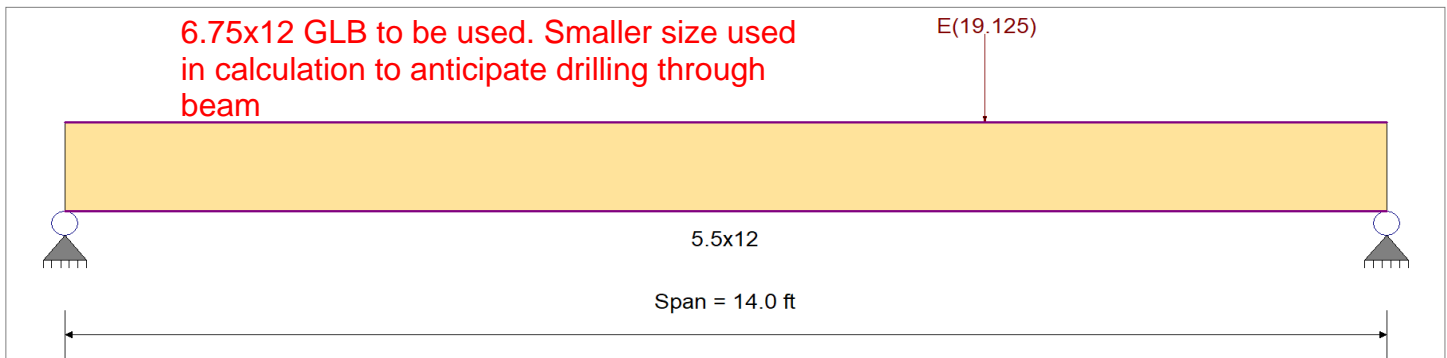
DESCRIPTION: Transfer Beam w/ HDU8

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

| | | | |
|--|-----------|----------|---------------------------|
| Analysis Method : Allowable Stress Design | Fb + | 2400 psi | E : Modulus of Elasticity |
| Load Combination : ASCE 7-16 | Fb - | 1850 psi | Ebend- xx |
| | Fc - Prll | 1650 psi | Eminbend - xx |
| Wood Species : DF/DF | Fc - Perp | 650 psi | Ebend- yy |
| Wood Grade : 24F-V4 | Fv | 265 psi | Eminbend - yy |
| | Ft | 1100 psi | Density |
| Beam Bracing : Beam is Fully Braced against lateral-torsional buckling | | | 31.21 pcf |



Applied Loads

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

Point Load : E = 19.125 k @ 9.750 ft, (HDU8)

DESIGN SUMMARY

Design N.G.

| | | | | | |
|-----------------------------------|----------------|-----|-------------------------------|----------------|------|
| Maximum Bending Stress Ratio = | 0.936 | 1 | Maximum Shear Stress Ratio = | 0.500 | : 1 |
| Section used for this span | 5.5x12 | | Section used for this span | 5.5x12 | |
| fb: Actual = | 3,594.50 | psi | fv: Actual = | 211.90 | psi |
| Fb: Allowable = | 3,840.00 | psi | Fv: Allowable = | 424.00 | psi |
| Load Combination = | +D+0.70E+0.60H | | Load Combination = | +D+0.70E+0.60H | |
| Location of maximum on span = | 9.759 ft | | Location of maximum on span = | 9.759 ft | |
| Span # where maximum occurs = | Span # 1 | | Span # where maximum occurs = | Span # 1 | |
| Maximum Deflection | | | | | |
| Max Downward Transient Deflection | 1.077 | in | Ratio = | 156 | <360 |
| Max Upward Transient Deflection | 0.000 | in | Ratio = | 0 | <360 |
| Max Downward Total Deflection | 0.754 | in | Ratio = | 222 | <240 |
| Max Upward Total Deflection | 0.000 | in | Ratio = | 0 | <240 |

Maximum Forces & Stresses for Load Combinations

| Load Combination | Segment Length | Span # | Max Stress Ratios | | | | | | | | | Moment Values | | | Shear Values | | | | | | |
|----------------------|------------------|--------|-------------------|---|----------------|-----------------|----------------|----------------|----------------|----------------|----------------|---------------|------|------|--------------|---------|------|------|------|------|--------|
| | | | M | V | C _d | C _{FV} | C _i | C _r | C _m | C _t | C _L | M | fb | F'b | V | fv | F'v | | | | |
| +D+H | Length = 14.0 ft | 1 | | | 0.90 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 2160.00 | 0.00 | 0.00 | 0.00 | 0.00 | 238.50 |
| +D+L+H | Length = 14.0 ft | 1 | | | 1.00 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| +D+Lr+H | Length = 14.0 ft | 1 | | | 1.00 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 265.00 |
| +D+S+H | Length = 14.0 ft | 1 | | | 1.25 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 331.25 |
| +D+0.750Lr+0.750L+H | Length = 14.0 ft | 1 | | | 1.15 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 304.75 |
| +D+0.750Lr+0.750Lr+H | Length = 14.0 ft | 1 | | | 1.25 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 331.25 |
| +D+0.750Lr+0.750S+H | Length = 14.0 ft | 1 | | | 1.15 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 304.75 |
| +D+0.60W+H | Length = 14.0 ft | 1 | | | 1.60 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 424.00 |



Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

File: beam check.ec6
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C.T. ENGINEERING

Lic. # : KW-06002997

DESCRIPTION: **Transfer Beam w/ HDU8**

| Load Combination | Segment Length | Span # | Max Stress Ratios | | Moment Values | | | | | | | | | | Shear Values | | | | | | |
|----------------------------|------------------|--------|-------------------|-------|----------------|------------------|----------------|----------------|----------------|----------------|----------------|------|------|------|--------------|----------------|---------|------|--------|--------|--------|
| | | | M | V | C _d | C _{F/V} | C _i | C _r | C _m | C _t | C _L | M | fb | F'b | V | f _v | F'v | | | | |
| +D+0.750Lr+0.750L+0.450W+H | Length = 14.0 ft | 1 | | | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| +D+0.750L+0.750S+0.450W+H | Length = 14.0 ft | 1 | | | 1.60 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 3840.00 | 0.00 | 0.00 | 0.00 | 0.00 | 424.00 |
| +D+0.750L+0.750S+0.5250E+H | Length = 14.0 ft | 1 | | | 1.60 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 3840.00 | 0.00 | 0.00 | 0.00 | 0.00 | 424.00 |
| +0.60D+0.60W+0.60H | Length = 14.0 ft | 1 | | | 1.00 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| +D+0.70E+0.60H | Length = 14.0 ft | 1 | 0.936 | 0.500 | 1.60 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 39.54 | 3,594.50 | 3840.00 | 9.32 | 211.90 | 424.00 | 0.00 |
| +D+0.750L+0.750S+0.5250E+H | Length = 14.0 ft | 1 | 0.702 | 0.375 | 1.60 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 29.65 | 2,695.88 | 3840.00 | 6.99 | 158.92 | 424.00 | 0.00 |
| +0.60D+0.70E+H | Length = 14.0 ft | 1 | 0.936 | 0.500 | 1.60 | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 39.54 | 3,594.50 | 3840.00 | 9.32 | 211.90 | 424.00 | 0.00 |

Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|------------------|------|---------------|------------------|------------------|---------------|------------------|
| E Only | 1 | 1.0769 | 7.715 | | 0.0000 | 0.000 |

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

| Load Combination | Support 1 | Support 2 |
|----------------------------|-----------|-----------|
| Overall MAXimum | 5.806 | 13.319 |
| Overall MINimum | 5.806 | 13.319 |
| +D+0.70E+0.60H | 4.064 | 9.323 |
| +D+0.750L+0.750S+0.5250E+H | 3.048 | 6.993 |
| +0.60D+0.70E+H | 4.064 | 9.323 |
| E Only | 5.806 | 13.319 |
| H Only | | |

Wood Beam

Lic. # : KW-06002997

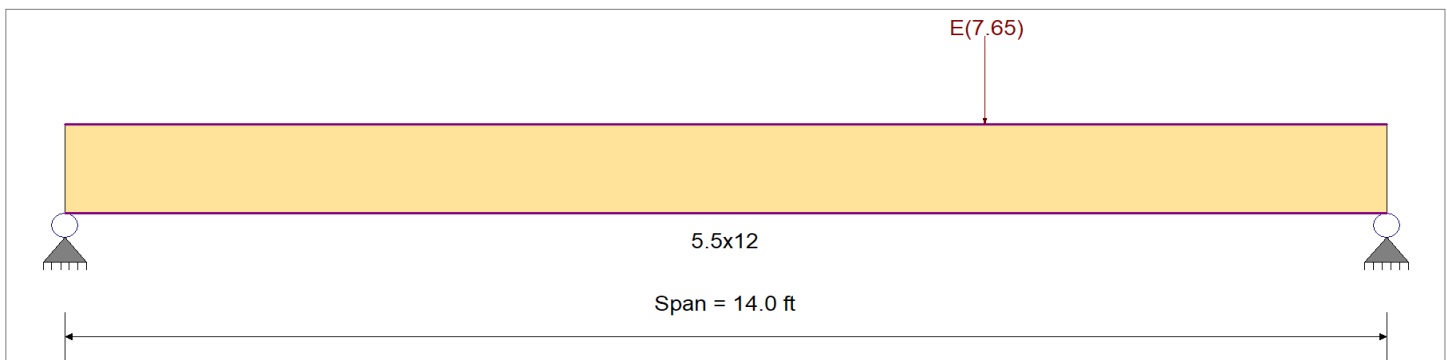
DESCRIPTION: Transfer Beam w/ HDU8 (Holdown calcs)

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
 Load Combination Set : ASCE 7-16

Material Properties

| | | | |
|--|-----------|-------------|---------------------------|
| Analysis Method : Allowable Stress Design | Fb + | 2,400.0 psi | E : Modulus of Elasticity |
| Load Combination : ASCE 7-16 | Fb - | 1,850.0 psi | Ebend- xx |
| | Fc - Prll | 1,650.0 psi | Eminbend - xx |
| Wood Species : DF/DF | Fc - Perp | 650.0 psi | Ebend- yy |
| Wood Grade : 24F-V4 | Fv | 265.0 psi | Eminbend - yy |
| | Ft | 1,100.0 psi | Density |
| Beam Bracing : Beam is Fully Braced against lateral-torsional buckling | | | 31.210pcf |



Applied Loads

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

Point Load : E = 7.650 k @ 9.750 ft, (HDU8)

DESIGN SUMMARY

Design OK

| | | | |
|-----------------------------------|----------------|------------------------------|------------------|
| Maximum Bending Stress Ratio = | 0.374 1 | Maximum Shear Stress Ratio = | 0.200 : 1 |
| Section used for this span | 5.5x12 | Section used for this span | 5.5x12 |
| fb: Actual = | 1,437.80psi | fv: Actual = | 84.76 psi |
| Fb: Allowable = | 3,840.00psi | Fv: Allowable = | 424.00 psi |
| Load Combination | +D+0.70E+0.60H | Load Combination | +D+0.70E+0.60H |
| Location of maximum on span | 9.759ft | Location of maximum on span | 9.759 ft |
| Span # where maximum occurs | Span # 1 | Span # where maximum occurs | Span # 1 |
| Maximum Deflection | | | |
| Max Downward Transient Deflection | 0.431 in | Ratio = | 390 >=360 |
| Max Upward Transient Deflection | 0.000 in | Ratio = | 0 <360 |
| Max Downward Total Deflection | 0.302 in | Ratio = | 557 >=240 |
| Max Upward Total Deflection | 0.000 in | Ratio = | 0 <240 |

Maximum Forces & Stresses for Load Combinations

| Load Combination | Segment Length | Span # | Max Stress Ratios | | Moment Values | | | | | | | | | | Shear Values | | | | | | | | | |
|----------------------|------------------|--------|-------------------|------|----------------|-----------------|----------------|----------------|----------------|----------------|----------------|------|------|------|--------------|------|---------|------|------|------|------|------|--------|--------|
| | | | M | V | C _d | C _{FV} | C _i | C _r | C _m | C _t | C _L | M | fb | F'b | V | fv | F'v | | | | | | | |
| +D+H | Length = 14.0 ft | 1 | 0.90 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 2160.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 238.50 | |
| +D+L+H | Length = 14.0 ft | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| +D+Lr+H | Length = 14.0 ft | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 2400.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 265.00 |
| +D+S+H | Length = 14.0 ft | 1 | 1.25 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 3000.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 331.25 |
| +D+0.750Lr+0.750L+H | Length = 14.0 ft | 1 | 1.15 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 2760.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 304.75 |
| +D+0.750Lr+0.750Lr+H | Length = 14.0 ft | 1 | 1.25 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 3000.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 331.25 |
| +D+0.750Lr+0.750S+H | Length = 14.0 ft | 1 | 1.15 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 2760.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 304.75 |
| +D+0.60W+H | Length = 14.0 ft | 1 | 1.60 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 3840.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 424.00 |



Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

File: beam check.ec6
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C.T. ENGINEERING

Lic. # : KW-06002997

DESCRIPTION: **Transfer Beam w/ HDU8 (Holdown calcs)**

| Load Combination | Segment Length | Span # | Max Stress Ratios | | C _d | C _{F/V} | C _i | C _r | C _m | C _t | C _L | Moment Values | | | Shear Values | | |
|----------------------------|------------------|--------|-------------------|-------|----------------|------------------|----------------|----------------|----------------|----------------|----------------|---------------|----------|---------|--------------|-------|--------|
| | | | M | V | | | | | | | | M | fb | F'b | V | fv | F'v |
| +D+0.750Lr+0.750L+0.450W+H | Length = 14.0 ft | 1 | | | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | | | 0.00 | 0.00 | 0.00 | 0.00 |
| +D+0.750L+0.750S+0.450W+H | Length = 14.0 ft | 1 | | | 1.60 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | | | 3840.00 | 0.00 | 0.00 | 424.00 |
| +D+0.750L+0.750S+0.5250E+H | Length = 14.0 ft | 1 | | | 1.60 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | | | 3840.00 | 0.00 | 0.00 | 424.00 |
| +0.60D+0.60W+0.60H | Length = 14.0 ft | 1 | | | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | | | 0.00 | 0.00 | 0.00 | 0.00 |
| +D+0.70E+0.60H | Length = 14.0 ft | 1 | 0.374 | 0.200 | 1.60 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 15.82 | 1,437.80 | 3840.00 | 3.73 | 84.76 | 424.00 |
| +D+0.750L+0.750S+0.5250E+H | Length = 14.0 ft | 1 | 0.281 | 0.150 | 1.60 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 11.86 | 1,078.35 | 3840.00 | 2.80 | 63.57 | 424.00 |
| +0.60D+0.70E+H | Length = 14.0 ft | 1 | 0.374 | 0.200 | 1.60 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 15.82 | 1,437.80 | 3840.00 | 3.73 | 84.76 | 424.00 |

Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|------------------|------|---------------|------------------|------------------|---------------|------------------|
| E Only | 1 | 0.4307 | 7.715 | | 0.0000 | 0.000 |

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

| Load Combination | Support 1 | Support 2 |
|----------------------------|-----------|-----------|
| Overall MAXimum | 2.322 | 5.328 |
| Overall MINimum | 2.322 | 5.328 |
| +D+0.70E+0.60H | 1.626 | 3.729 |
| +D+0.750L+0.750S+0.5250E+H | 1.219 | 2.797 |
| +0.60D+0.70E+H | 1.626 | 3.729 |
| E Only | 2.322 | 5.328 |
| H Only | | |

Project: Foo RESIDENCE

Date:

Client:

Page Number:

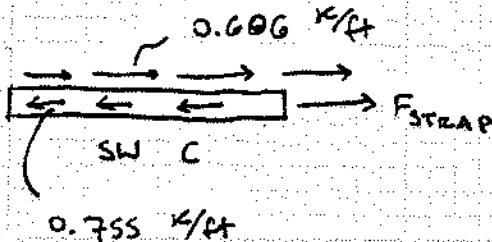
SHEAR WALL 'C' / DETAIL 17/SI.1

SW LENGTH = 11.25' w/ 755 plf

Total length of collector = 14'

TOTAL FORCE IN ~~CONCRETE~~
DIAPHRAGM = 8.49k

DIAPHRAGM UNIT SHEAR = $\frac{8.49k}{14'} = 0.606 \text{ k/ft}$



$$\begin{aligned} \sum F_x = 0 &\Rightarrow F_{STRAP} + (0.606 \text{ k/ft})(11.25') - (0.755 \text{ k/ft})(11.25') \\ &= 0 \Rightarrow F_{STRAP} = 1.67 \text{ kip} \end{aligned}$$

CAPACITY OF MST148 = 5070 lb (DF/SP)
4390 lb (SPF/HF)

DESIGN OK

A Ya Vyf'Dc]bhi@UXg'

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A Ya Vyf'8]glf]Vi hYX' @UXg'f6 @' % '8 YUX' @UXL

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A Ya Vyf'8]glf]Vi hYX' @UXg'f6 @' &: 'Gback' @UXL

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9bj YcdY>c]bhiFYUM]cbg

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| E-mail: | | | |

1. Project information

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

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General

Design method: ACI 318-14
Units: Imperial units

Anchor Information:

Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 0.875
Effective Embedment depth, h_{ef} (inch): 24.000
Anchor category: -
Anchor ductility: Yes
 h_{min} (inch): 25.63
 C_{min} (inch): 1.33
 S_{min} (inch): 3.50

Base Material

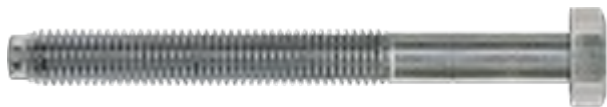
Concrete: Normal-weight
Concrete thickness, h (inch): 26.00
State: Cracked
Compressive strength, f_c (psi): 3000
 $\Psi_{c,v}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Base Plate

Length x Width x Thickness (inch): 11.50 x 10.28 x 1.00

Recommended Anchor

Anchor Name: Heavy Hex Bolt - 7/8"Ø Heavy Hex Bolt, F1554 Gr. 36





| | | | |
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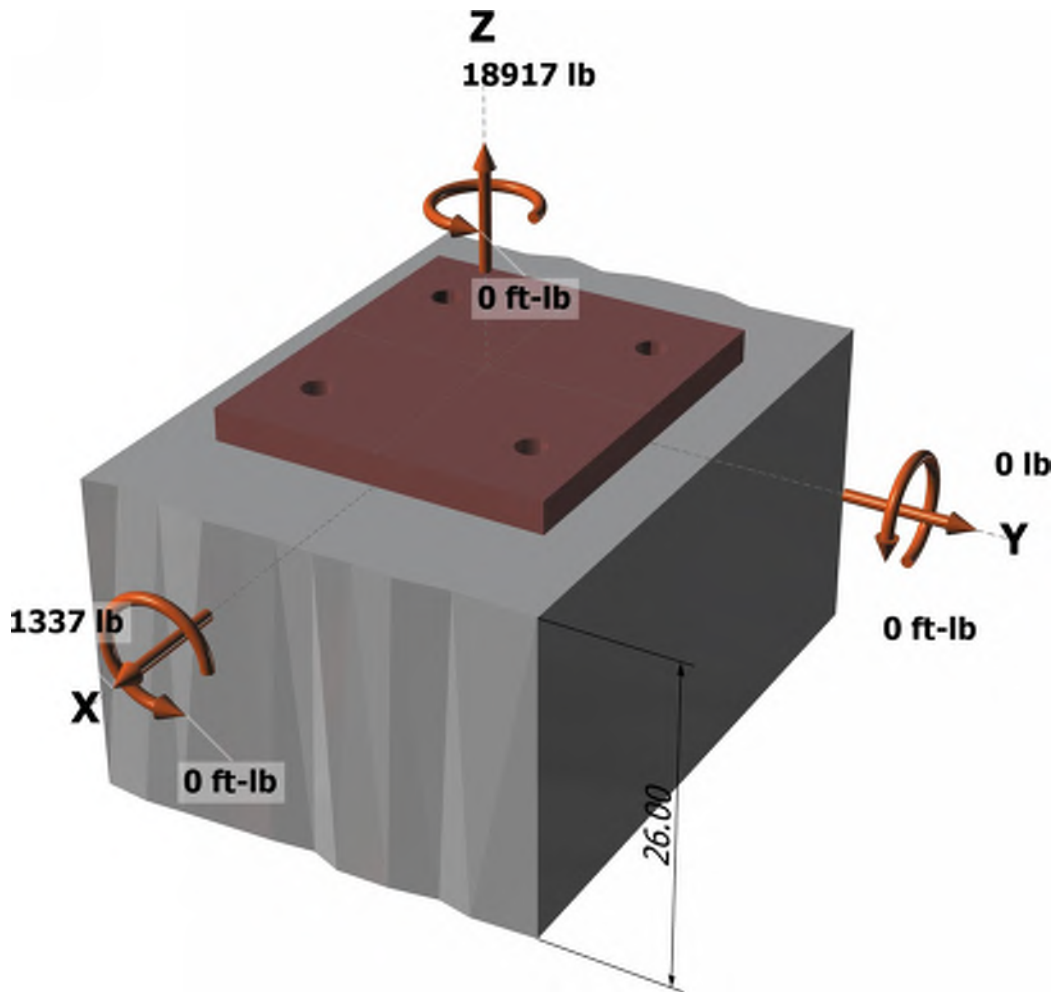
Load and Geometry

Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 18917
 V_{uax} [lb]: 1337
 V_{uay} [lb]: 0
 M_{ux} [ft-lb]: 0
 M_{uy} [ft-lb]: 0
 M_{uz} [ft-lb]: 0

<Figure 1>

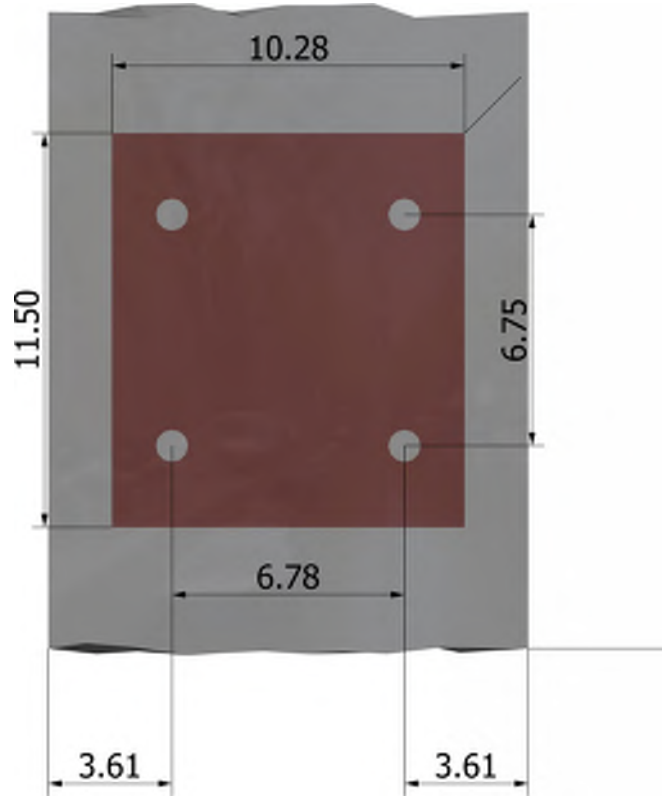


Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

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

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| E-mail: | | | |

<Figure 2>





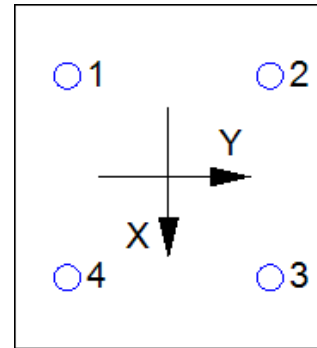
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| E-mail: | | | |

3. Resulting Anchor Forces

| Anchor | Tension load, N _{ua} (lb) | Shear load x, V _{uax} (lb) | Shear load y, V _{uay} (lb) | Shear load combined, $\sqrt{(V_{uax})^2 + (V_{uay})^2}$ (lb) |
|--------|---------------------------------------|--|--|---|
| 1 | 4729.3 | 334.3 | 0.0 | 334.3 |
| 2 | 4729.3 | 334.3 | 0.0 | 334.3 |
| 3 | 4729.3 | 334.3 | 0.0 | 334.3 |
| 4 | 4729.3 | 334.3 | 0.0 | 334.3 |
| Sum | 18917.0 | 1337.0 | 0.0 | 1337.0 |

Maximum concrete compression strain (%): 0.00
 Maximum concrete compression stress (psi): 0
 Resultant tension force (lb): 18917
 Resultant compression force (lb): 0
 Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
 Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
 Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
 Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



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

| N _{sa} (lb) | φ | φN _{sa} (lb) |
|----------------------|------|-----------------------|
| 26795 | 0.75 | 20096 |

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

$$N_b = 16\lambda_a \sqrt{f_c} h_{ef}^{5/3} \text{ (Eq. 17.4.2.2b)}$$

| λ _a | f _c (psi) | h _{ef} (in) | N _b (lb) |
|----------------|----------------------|----------------------|---------------------|
| 1.00 | 3000 | 24.000 | 174998 |

$$\phi N_{cbg} = \phi (A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \text{ (Sec. 17.3.1 \& Eq. 17.4.2.1b)}$$

| A _{Nc} (in ²) | A _{Nco} (in ²) | c _{a,min} (in) | Ψ _{ec,N} | Ψ _{ed,N} | Ψ _{c,N} | Ψ _{cp,N} | N _b (lb) | φ | φN _{cbg} (lb) |
|------------------------------------|-------------------------------------|-------------------------|-------------------|-------------------|------------------|-------------------|---------------------|------|------------------------|
| 1102.50 | 5184.00 | 3.61 | 1.000 | 0.730 | 1.00 | 1.000 | 174998 | 0.70 | 19020 |

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

$$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8A_{brg} f_c \text{ (Sec. 17.3.1, Eq. 17.4.3.1 \& 17.4.3.4)}$$

| Ψ _{c,P} | A _{brg} (in ²) | f _c (psi) | φ | φN _{pn} (lb) |
|------------------|-------------------------------------|----------------------|------|-----------------------|
| 1.0 | 1.19 | 3000 | 0.70 | 19958 |

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.



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7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.4.4)

$$\phi N_{sb} = \phi \left\{ (1 + c_{a2}/c_{a1})/4 \right\} (1 + s/6c_{a1}) N_{sb} = \phi \left\{ (1 + c_{a2}/c_{a1})/4 \right\} (1 + s/6c_{a1}) (160c_{a1} \sqrt{A_{brg}}) \lambda \sqrt{f_c} \quad (\text{Sec. 17.3.1, Eq. 17.4.4.1 \& 17.4.4.2})$$

| s (in) | c _{a1} (in) | c _{a2} (in) | A _{brg} (in ²) | λ _a | f' _c (psi) | φ | φN _{sb} (lb) |
|--------|----------------------|----------------------|-------------------------------------|----------------|-----------------------|------|-----------------------|
| 6.75 | 3.61 | 99999.00 | 1.19 | 1.00 | 3000 | 0.70 | 31660 |

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

| V _{sa} (lb) | φ _{grout} | φ | φ _{grout} φV _{sa} (lb) |
|----------------------|--------------------|------|--|
| 16080 | 1.0 | 0.65 | 10452 |

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

Shear parallel to edge in x-direction:

$$V_{by} = \min | 7(l_e/d_a)^{0.2} \sqrt{d_a} \lambda_a \sqrt{f_c} c_{a1}^{1.5}; 9 \lambda_a \sqrt{f_c} c_{a1}^{1.5} | \quad (\text{Eq. 17.5.2.2a \& Eq. 17.5.2.2b})$$

| l _e (in) | d _a (in) | λ _a | f' _c (psi) | c _{a1} (in) | V _{by} (lb) |
|---------------------|---------------------|----------------|-----------------------|----------------------|----------------------|
| 7.00 | 0.875 | 1.00 | 3000 | 3.61 | 3381 |

$$\phi V_{cbgx} = \phi (2)(A_{Vc}/A_{Vco}) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{by} \quad (\text{Sec. 17.3.1, 17.5.2.1(c) \& Eq. 17.5.2.1b})$$

| A _{Vc} (in ²) | A _{Vco} (in ²) | Ψ _{ec,V} | Ψ _{ed,V} | Ψ _{c,V} | Ψ _{h,V} | V _{by} (lb) | φ | φV _{cbgx} (lb) |
|------------------------------------|-------------------------------------|-------------------|-------------------|------------------|------------------|----------------------|------|-------------------------|
| 95.20 | 58.64 | 1.000 | 1.000 | 1.000 | 1.000 | 3381 | 0.70 | 7684 |

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

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

| k _{cp} | A _{Nc} (in ²) | A _{Nco} (in ²) | Ψ _{ec,N} | Ψ _{ed,N} | Ψ _{c,N} | Ψ _{cp,N} | N _b (lb) | φ | φV _{cp} (lb) |
|-----------------|------------------------------------|-------------------------------------|-------------------|-------------------|------------------|-------------------|---------------------|------|-----------------------|
| 2.0 | 1102.50 | 5184.00 | 1.000 | 0.730 | 1.000 | 1.000 | 174998 | 0.70 | 38041 |

11. Results

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

| Tension | Factored Load, N _{ua} (lb) | Design Strength, φN _n (lb) | Ratio | Status | |
|--------------------------------|-------------------------------------|---------------------------------------|----------------|-----------------------|--------|
| Steel | 4729 | 20096 | 0.24 | Pass | |
| Concrete breakout | 18917 | 19020 | 0.99 | Pass (Governs) | |
| Pullout | 4729 | 19958 | 0.24 | Pass | |
| Side-face blowout | 9459 | 31660 | 0.30 | Pass | |
| Shear | Factored Load, V _{ua} (lb) | Design Strength, φV _n (lb) | Ratio | Status | |
| Steel | 334 | 10452 | 0.03 | Pass | |
| Concrete breakout y+ | 669 | 7684 | 0.09 | Pass (Governs) | |
| Pryout | 1337 | 38041 | 0.04 | Pass | |
| Interaction check | N _{ua} /φN _n | V _{ua} /φV _n | Combined Ratio | Permissible | Status |
| Sec. 17.6..1 | 0.99 | 0.00 | 99.5% | 1.0 | Pass |

7/8"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 24.000 inch meets the selected design criteria.



Anchor Designer™
Software
Version 2.8.7094.0

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12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.
- Designer must exercise own judgement to determine if this design is suitable.

MOMENT FRAME CALCULATIONS

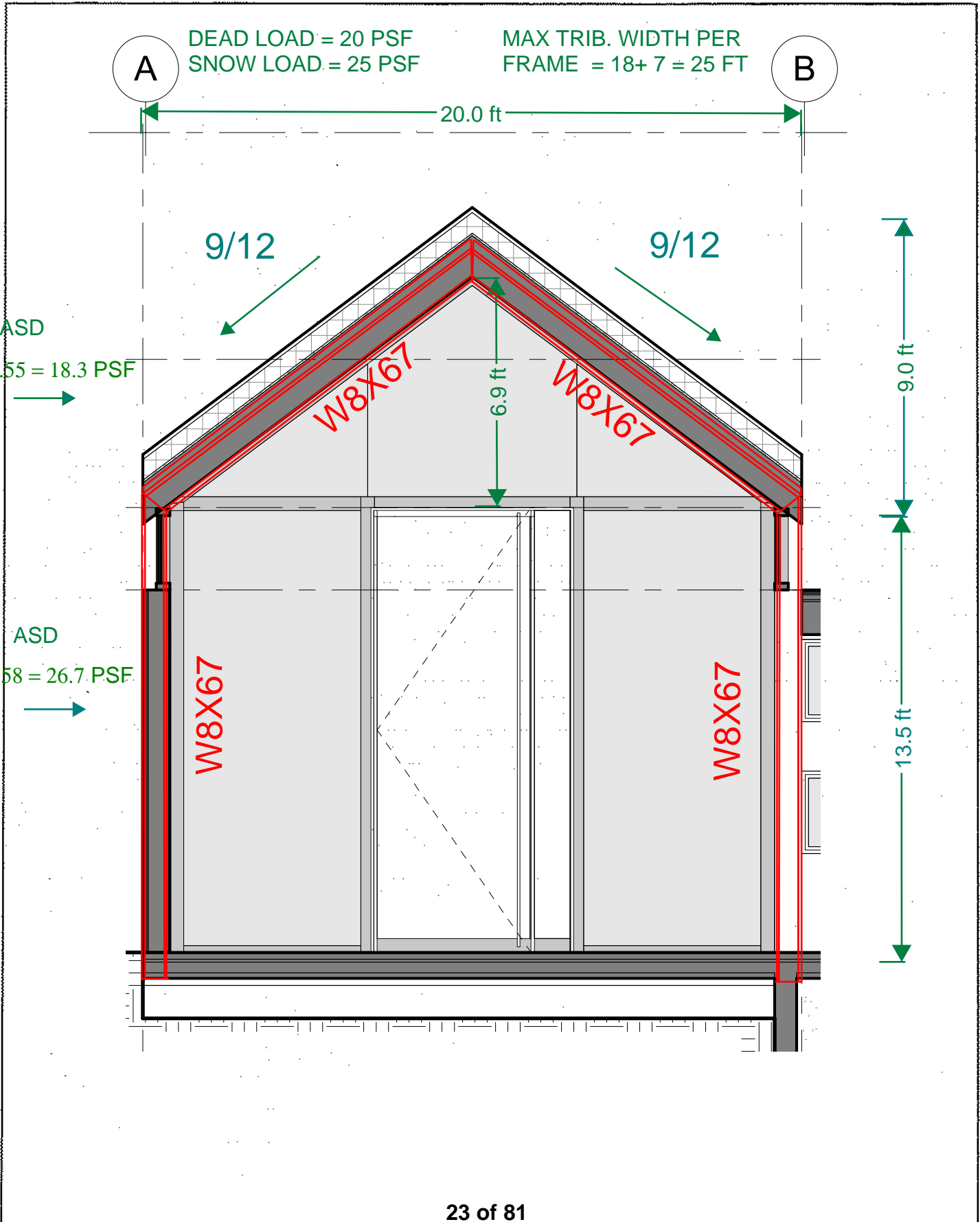
FOO FRAMES

Project:

Date:

Client:

Page Number:



Mercer Island Wind Exposure and Wind Speed-Up (Topographic Effect)



180 Nickerson St.
 Suite 302
 Seattle, WA
 98109
 (206) 285-4512
 FAX:
 (206) 285-0618

by Development Services Group (DSG), City of Mercer Island
 April 2009

COPY




WIND EXPOSURE CATEGORIES:

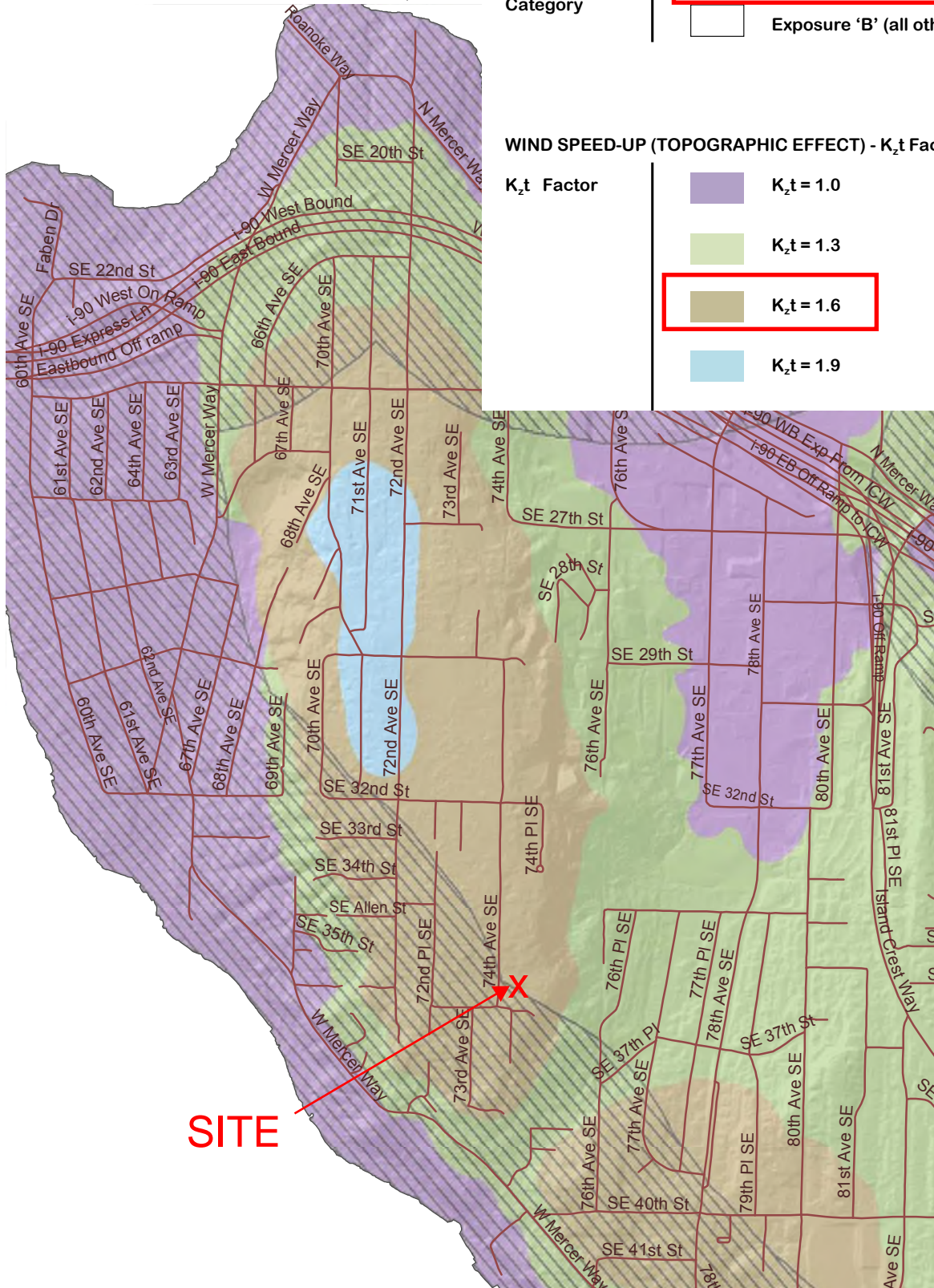
Wind Exposure Category

-  Exposure 'C' (1500 feet from Lake)
-  Exposure 'B' (all other areas)

WIND SPEED-UP (TOPOGRAPHIC EFFECT) - $K_z t$ Factor :

$K_z t$ Factor

-  $K_z t = 1.0$
-  $K_z t = 1.3$
-  $K_z t = 1.6$
-  $K_z t = 1.9$



SITE

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Date: 5-25-2020
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WIND LOADS - MAIN WIND FORCE RESISTING SYSTEM

Simplified Design Wind Pressure ASCE 7-10, Section 28.6

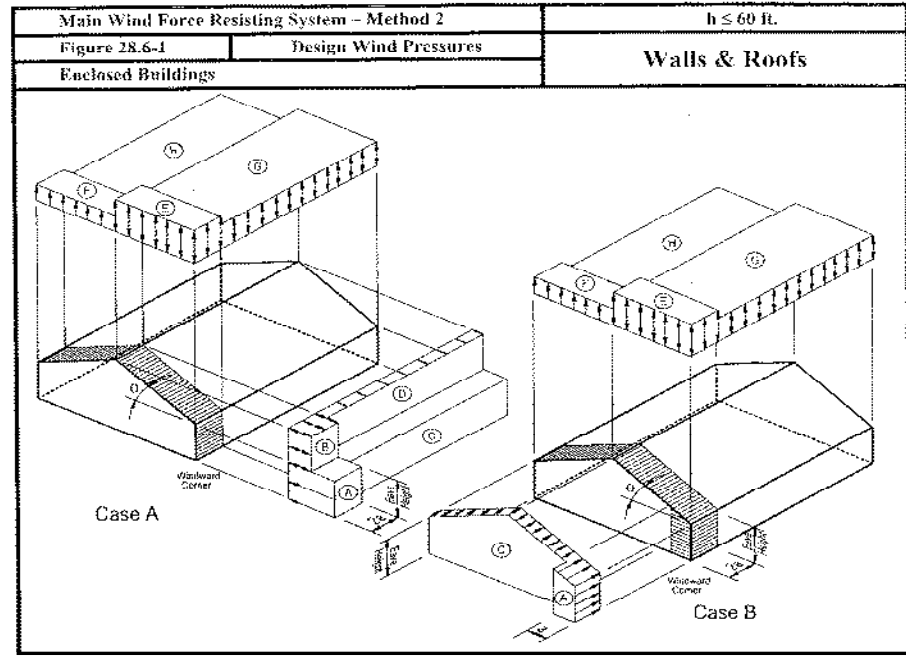
Wind Criteria:

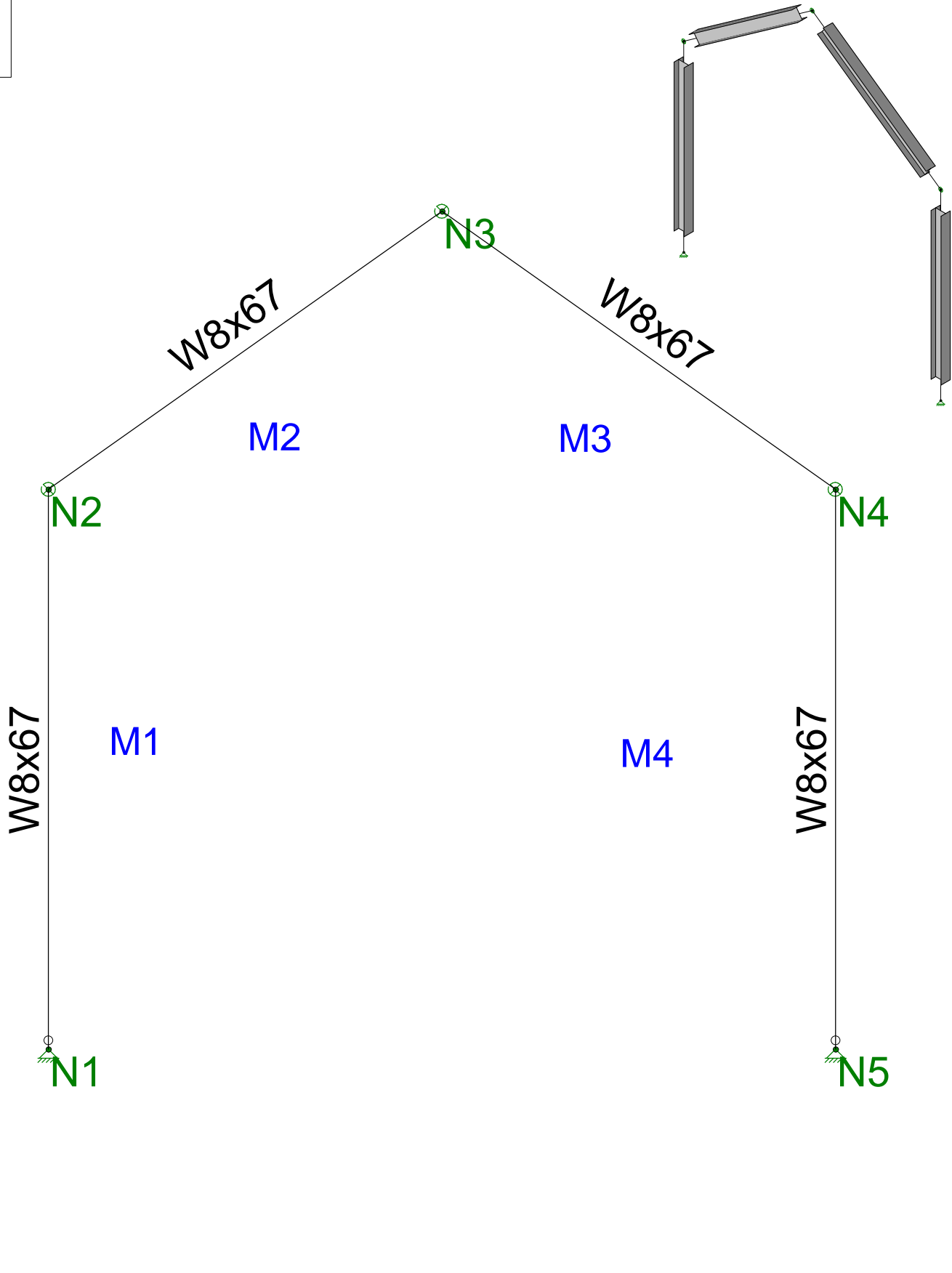
- Risk Category, I, II, III or IV = **II** Table 1.5-1 of ASCE 7-10
- Basic Wind Speed **110** MPH Figure 26.5-1A, B or C of ASCE 7-10
- Exposure **C**
- Average roof height **18.00** ft.
- Roof angle **30° to 45°**
- K_{zt} = **1.60** Section 26.8 and Figure 26.8-1 of ASCE 7-10
- λ = **1.29**
- Minimum wind pressure, p_s , Zones A and C = +16 PSF, Zones B and D = +8 PSF, with p_s for Zones E, F, G and H = 0 psf. (Section 26.6.4 of ASCE 7-10)
- $P_s = \lambda * K_{zt} * p_{s30}$

ASD

(0.6) 44.58 = 26.7 PSF →
 (0.6) 30.55 = 18.3 PSF →

| Zone | Load Case 1 | | | | Load Case 2 | | | |
|----------|-------------|-----|--------|-----|-------------|-----|--------|-----|
| | p_{s30} | | P_s | | p_{s30} | | P_s | |
| A | 21.60 | PSF | 44.58 | PSF | 21.60 | PSF | 44.58 | PSF |
| B | 14.80 | PSF | 30.55 | PSF | 14.80 | PSF | 30.55 | PSF |
| C | 17.20 | PSF | 35.50 | PSF | 17.20 | PSF | 35.50 | PSF |
| D | 11.80 | PSF | 24.36 | PSF | 11.80 | PSF | 24.36 | PSF |
| E | 1.70 | PSF | 3.51 | PSF | 8.30 | PSF | 17.13 | PSF |
| F | -13.10 | PSF | -27.04 | PSF | -6.50 | PSF | -13.42 | PSF |
| G | 0.60 | PSF | 1.24 | PSF | 7.20 | PSF | 14.86 | PSF |
| H | -11.30 | PSF | -23.32 | PSF | -4.60 | PSF | -9.49 | PSF |
| E_{OH} | -7.60 | PSF | -15.69 | PSF | -7.60 | PSF | -15.69 | PSF |
| G_{OH} | -8.70 | PSF | -17.96 | PSF | -8.70 | PSF | -17.96 | PSF |





CT Engineering

STG

20035

Foo Frames
max load middle frame

June 4, 2020 at 8:23 AM

Foo frames W8x67.r2d

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| FG | | | { á } ĞĚİİ | G | İİH | Fİ | ĚĚJİ | Fİ |
| FH | | G | { æ } ĪĚİ | FH | ĪĚFİ | Í | FĪĚFİ | Í |
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Project: **FOO FRAMES**

Date: _____

(206) 285-4512

Client: _____

Page Number: _____

FAX:
(206) 285-0618**SEISMIC LOAD ON FRAME NEAR CHIMNEY:**

$$\text{BUILDING WT} = .025 (26)(18) = 11.7\text{K}$$

$$\text{BUILDING SEISMIC} = 0.143(11.7) = 1.67 \text{ K } (0.7) = 1.17 \text{ ASD}$$

$$\text{CHIMNEY SEISMIC LOAD} = 26/36(5.86\text{K}) = 4.23\text{K ASD}$$

$$\text{TOTAL SEISMIC} = 1.17 + 4.23 = 5.4 \text{ K ASD} < \text{WIND} = 8.64 \text{ K ASD}$$

PLANTER BOX FRAMING CALCULATIONS

Project: FOO RESIDENCE

Date: _____

(206) 285-4512

Client: _____

Page Number: _____

FAX:

(206) 285-0618

PLANTER BOX SUPPORT (LEVEL 2 FLOOR FMG)

SIZE: 5.0' x 14.0' x 1.0 DEEP

SOIL: LIGHT WEIGHT SATURATED SOIL = 95 PCF

BOX WEIGHT: 5 PSF (ESTIMATE)

PLANTER WEIGHT: 5 PSF + 95 PCF (1.0) = 100 PSF

NORMAL DECK LOADS: DL = 34 PSF L = 60 PSF

BMX1
 L=13.0'
 TRIB=1.0'
 DL=34PSF
 LL=60PSF

$W = (34+60)1.0 = 94 \#/\text{ft}$
 $M = 1.986 \text{ K-ft} < 15.0 \text{ K-ft OK}$
 $V = 611 \# < 6630 \# \text{ OK}$
 $EI = 3.92 \times 10^4 < 8.10 \times 10^5 \text{ OK}$

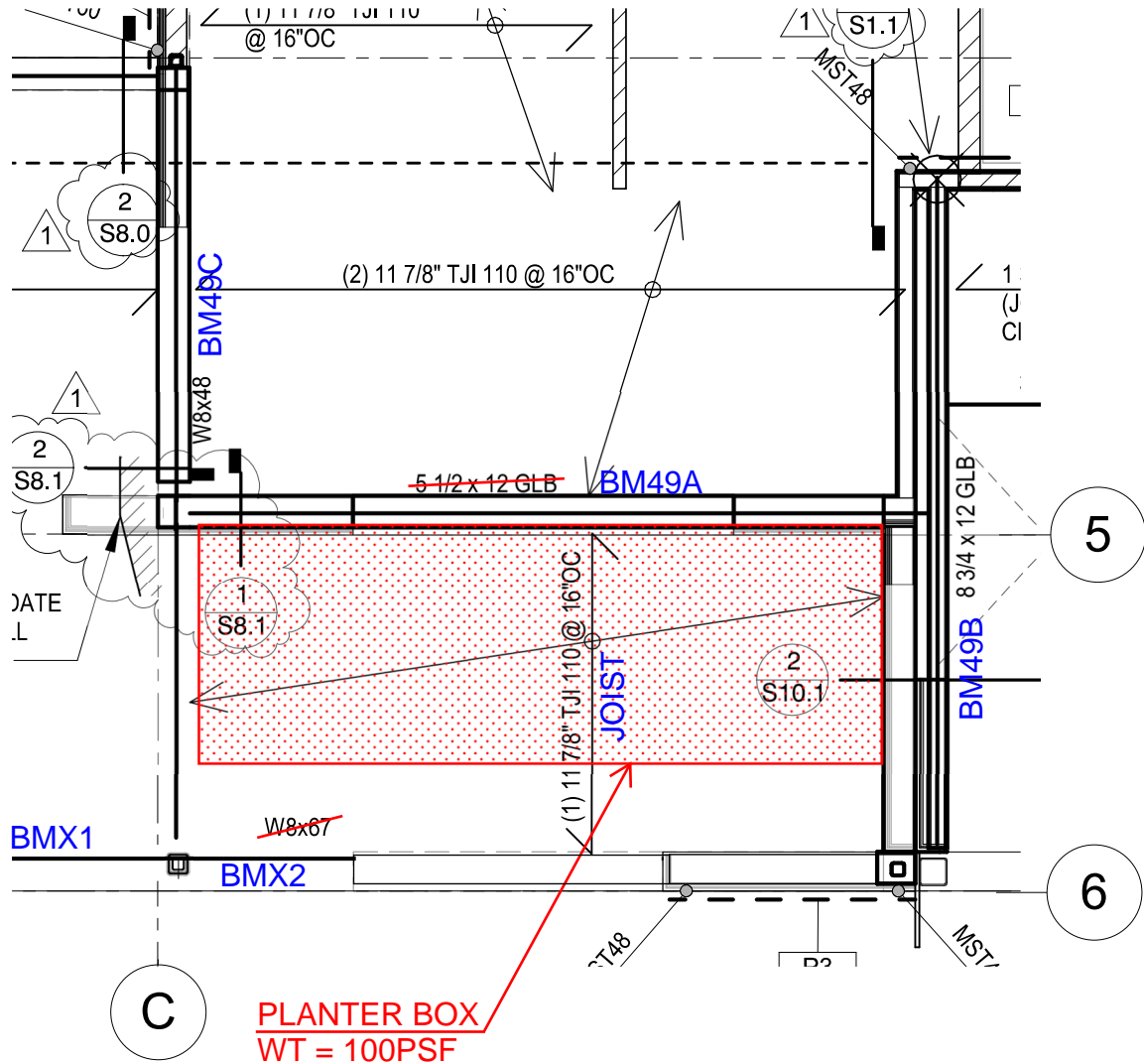
USE 3/2 x 12 GLB

BMX2

L=4.0'
 TRIB=4.0'
 DL=100PSF + 34PSF = 134PSF
 LL=25PSF

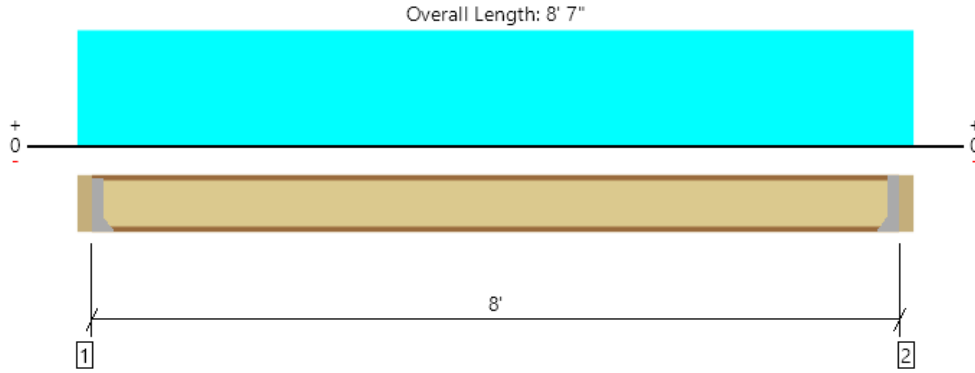
$W = (134+25) = 159 \#/\text{ft}$
 $M = 159(4) \frac{1}{6} = 318 \# \text{-ft}$
 $V = 318 \#$

USE 3/2 x 12 GLB



PARTIAL L2 FRAMING

Level 2, Floor: Joist @ planter box
 1 piece(s) 11 7/8" TJI @ 110 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

| Design Results | Actual @ Location | Allowed | Result | LDF | Load: Combination (Pattern) |
|-----------------------|-------------------|-------------|-----------------|------|-----------------------------|
| Member Reaction (lbs) | 715 @ 3 1/2" | 819 (1.75") | Passed (87%) | 0.90 | 1.0 D (All Spans) |
| Shear (lbs) | 715 @ 3 1/2" | 1404 | Passed (51%) | 0.90 | 1.0 D (All Spans) |
| Moment (Ft-lbs) | 1429 @ 4' 3 1/2" | 2844 | Passed (50%) | 0.90 | 1.0 D (All Spans) |
| Live Load Defl. (in) | 0.014 @ 4' 3 1/2" | 0.200 | Passed (L/999+) | -- | 1.0 D + 1.0 S (All Spans) |
| Total Load Defl. (in) | 0.089 @ 4' 3 1/2" | 0.400 | Passed (L/999+) | -- | 1.0 D + 1.0 S (All Spans) |
| TJ-Pro™ Rating | 64 | 40 | Passed | -- | -- |

System : Floor
 Member Type : Joist
 Building Use : Residential
 Building Code : IBC 2015
 Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

| Supports | Bearing Length | | | Loads to Supports (lbs) | | | Accessories |
|-------------------------------|----------------|---------------------|------------------------|-------------------------|------|-------|-----------------------|
| | Total | Available | Required | Dead | Snow | Total | |
| 1 - Hanger on 11 7/8" HF beam | 3.50" | Hanger ¹ | 1.75" / - ² | 767 | 143 | 910 | See note ¹ |
| 2 - Hanger on 11 7/8" HF beam | 3.50" | Hanger ¹ | 1.75" / - ² | 767 | 143 | 910 | See note ¹ |

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.
- ² Required Bearing Length / Required Bearing Length with Web Stiffeners

| Lateral Bracing | Bracing Intervals | Comments |
|------------------|-------------------|----------|
| Top Edge (Lu) | 4' 4" o/c | |
| Bottom Edge (Lu) | 8' o/c | |

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

| Connector: Simpson Strong-Tie | | | | | | |
|-------------------------------|---------------|-------------|---------------|----------------|------------------|-------------|
| Support | Model | Seat Length | Top Fasteners | Face Fasteners | Member Fasteners | Accessories |
| 1 - Face Mount Hanger | IUS1.81/11.88 | 2.00" | N/A | 10-10dx1.5 | 2-Strong-Grip | |
| 2 - Face Mount Hanger | IUS1.81/11.88 | 2.00" | N/A | 10-10dx1.5 | 2-Strong-Grip | |

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

| Vertical Load | Location (Side) | Spacing | Dead (0.90) | Snow (1.15) | Comments |
|-------------------|-----------------|---------|-------------|-------------|-------------|
| 1 - Uniform (PSF) | 0 to 8' 7" | 16" | 134.0 | 25.0 | planter box |

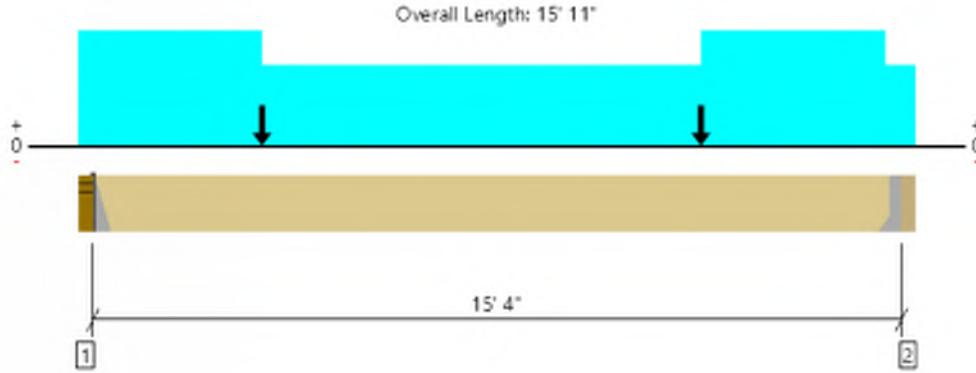
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 The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

| ForteWEB Software Operator | Job Notes |
|--|-----------|
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Level 2, BM49A(rev)

1 piece(s) 5 1/2" x 13 1/2" 24F-V4 DF Glulam



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

| Design Results | Actual @ Location | Allowed | Result | LDF | Load: Combination (Pattern) |
|-----------------------|----------------------|--------------|-----------------|------|-----------------------------|
| Member Reaction (lbs) | 7694 @ 3 1/2" | 7694 (2.15") | Passed (100%) | -- | 1.0 D + 1.0 S (All Spans) |
| Shear (lbs) | 6560 @ 14' 6" | 15085 | Passed (43%) | 1.15 | 1.0 D + 1.0 S (All Spans) |
| Pos Moment (Ft-lbs) | 26814 @ 8' 1" | 38424 | Passed (70%) | 1.15 | 1.0 D + 1.0 S (All Spans) |
| Live Load Defl. (in) | 0.142 @ 8' | 0.383 | Passed (L/999+) | -- | 1.0 D + 1.0 S (All Spans) |
| Total Load Defl. (in) | 0.576 @ 7' 11 11/16" | 0.767 | Passed (L/320) | -- | 1.0 D + 1.0 S (All Spans) |

System : Floor
 Member Type : Flush Beam
 Building Use : Residential
 Building Code : IBC 2015
 Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 15' 4".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

| Supports | Bearing Length | | | Loads to Supports (lbs) | | | | Accessories |
|-------------------------------|----------------|---------------------|----------|-------------------------|------------|------|-------|-----------------------|
| | Total | Available | Required | Dead | Floor Live | Snow | Total | |
| 1 - Hanger on HF studWall | 3.50" | Hanger ¹ | 2.15" | 5858 | 424 | 2127 | 8409 | See note ¹ |
| 2 - Hanger on 13 1/2" HF beam | 3.50" | Hanger ¹ | 2.13" | 5778 | 424 | 2030 | 8232 | See note ¹ |

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.

| Lateral Bracing | Bracing Intervals | Comments |
|------------------|-------------------|----------|
| Top Edge (Lu) | 15' 4" o/c | |
| Bottom Edge (Lu) | 15' 4" o/c | |

•Maximum allowable bracing intervals based on applied load.

| Connector: Simpson Strong-Tie | | | | | | |
|-------------------------------|---------------------|-------------|---------------|----------------|------------------|-------------|
| Support | Model | Seat Length | Top Fasteners | Face Fasteners | Member Fasteners | Accessories |
| 1 - Top Mount Hanger | Connector not found | N/A | N/A | N/A | N/A | |
| 2 - Face Mount Hanger | MGU5.62-SDS H=13.5 | 4.50" | N/A | 24-SDS25212 | 16-SDS25212 | |

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

| Vertical Loads | Location (Side) | Tributary Width | Dead (0.90) | Floor Live (1.00) | Snow (1.15) | Comments |
|-----------------------|---------------------------|-----------------|-------------|-------------------|-------------|----------------|
| 0 - Self Weight (PLF) | 3 1/2" to 15' 7 1/2" | N/A | 18.0 | -- | -- | |
| 1 - Uniform (PSF) | 0 to 15' 11" (Front) | 1' 4" | 34.0 | 40.0 | - | floor load |
| 2 - Uniform (PSF) | 0 to 15' 11" (Front) | 4' | 134.0 | - | 25.0 | planter load |
| 3 - Uniform (PSF) | 0 to 3' 6" (Front) | 7' | 20.0 | - | 25.0 | roof load |
| 4 - Uniform (PSF) | 11' 10" to 15' 4" (Front) | 7' | 20.0 | - | 25.0 | roof load |
| 5 - Point (lb) | 3' 6" (Front) | N/A | 563 | - | 670 | BM10 Support 1 |
| 6 - Point (lb) | 11' 10" (Front) | N/A | 563 | - | 670 | BM10 Support 2 |

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Member Notes

BM49A Planter Box

Weyerhaeuser Notes

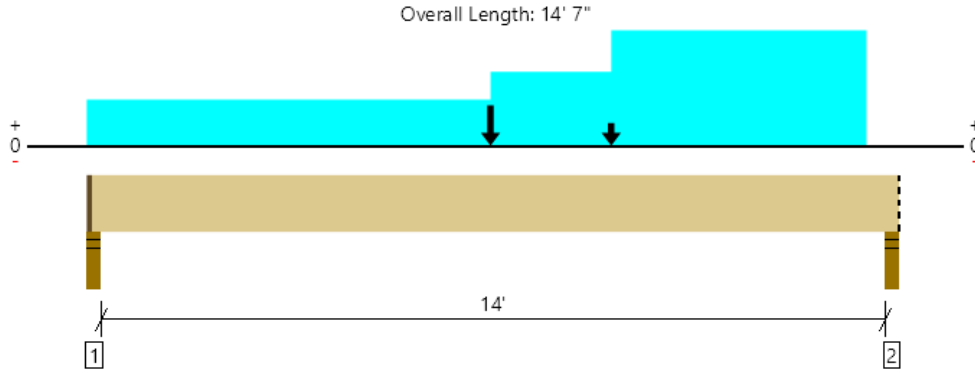
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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

| ForteWEB Software Operator | Job Notes |
|--|-----------|
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Level 2, BM49B(rev)
1 piece(s) 8 3/4" x 12" 24F-V4 DF Glulam



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

| Design Results | Actual @ Location | Allowed | Result | LDF | Load: Combination (Pattern) |
|-----------------------|---------------------|--------------|----------------|------|-------------------------------------|
| Member Reaction (lbs) | 7453 @ 2" | 7973 (2.25") | Passed (93%) | -- | 1.0 D + 0.75 L + 0.75 S (All Spans) |
| Shear (lbs) | 8589 @ 13' 3 1/2" | 18550 | Passed (46%) | 1.00 | 1.0 D + 1.0 L (All Spans) |
| Pos Moment (Ft-lbs) | 41462 @ 7' 3" | 41386 | Passed (100%) | 1.00 | 1.0 D + 1.0 L (All Spans) |
| Live Load Defl. (in) | 0.209 @ 7' 5 5/8" | 0.356 | Passed (L/818) | -- | 1.0 D + 0.75 L + 0.75 S (All Spans) |
| Total Load Defl. (in) | 0.626 @ 7' 4 15/16" | 0.712 | Passed (L/273) | -- | 1.0 D + 0.75 L + 0.75 S (All Spans) |

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 0.99 that was calculated using length L = 14' 3".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

| Supports | Bearing Length | | | Loads to Supports (lbs) | | | | Accessories |
|--------------------|----------------|-----------|----------|-------------------------|------------|------|-------|------------------|
| | Total | Available | Required | Dead | Floor Live | Snow | Total | |
| 1 - Stud wall - HF | 3.50" | 2.25" | 2.10" | 4930 | 2388 | 1021 | 8339 | 1 1/4" Rim Board |
| 2 - Stud wall - HF | 3.50" | 3.50" | 2.62" | 5823 | 3454 | 1009 | 10286 | Blocking |

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

| Lateral Bracing | Bracing Intervals | Comments |
|------------------|-------------------|----------|
| Top Edge (Lu) | 14' 6" o/c | |
| Bottom Edge (Lu) | 14' 6" o/c | |

- Maximum allowable bracing intervals based on applied load.

| Vertical Loads | Location (Side) | Tributary Width | Dead (0.90) | Floor Live (1.00) | Snow (1.15) | Comments |
|-----------------------|------------------------|-----------------|-------------|-------------------|-------------|--------------|
| 0 - Self Weight (PLF) | 1 1/4" to 14' 7" | N/A | 25.5 | -- | -- | |
| 1 - Uniform (PSF) | 0 to 7' 3" (Front) | 5' | 34.0 | 40.0 | - | Default Load |
| 2 - Uniform (PSF) | 7' 3" to 9' 5" (Front) | 8' | 34.0 | 40.0 | - | |
| 3 - Uniform (PSF) | 9' 5" to 14' (Front) | 12' 6" | 34.0 | 40.0 | - | |
| 4 - Point (lb) | 7' 3" (Front) | N/A | 5728 | 424 | 2030 | BM49A |
| 5 - Point (lb) | 9' 5" (Front) | N/A | 886 | 983 | - | BM49 |

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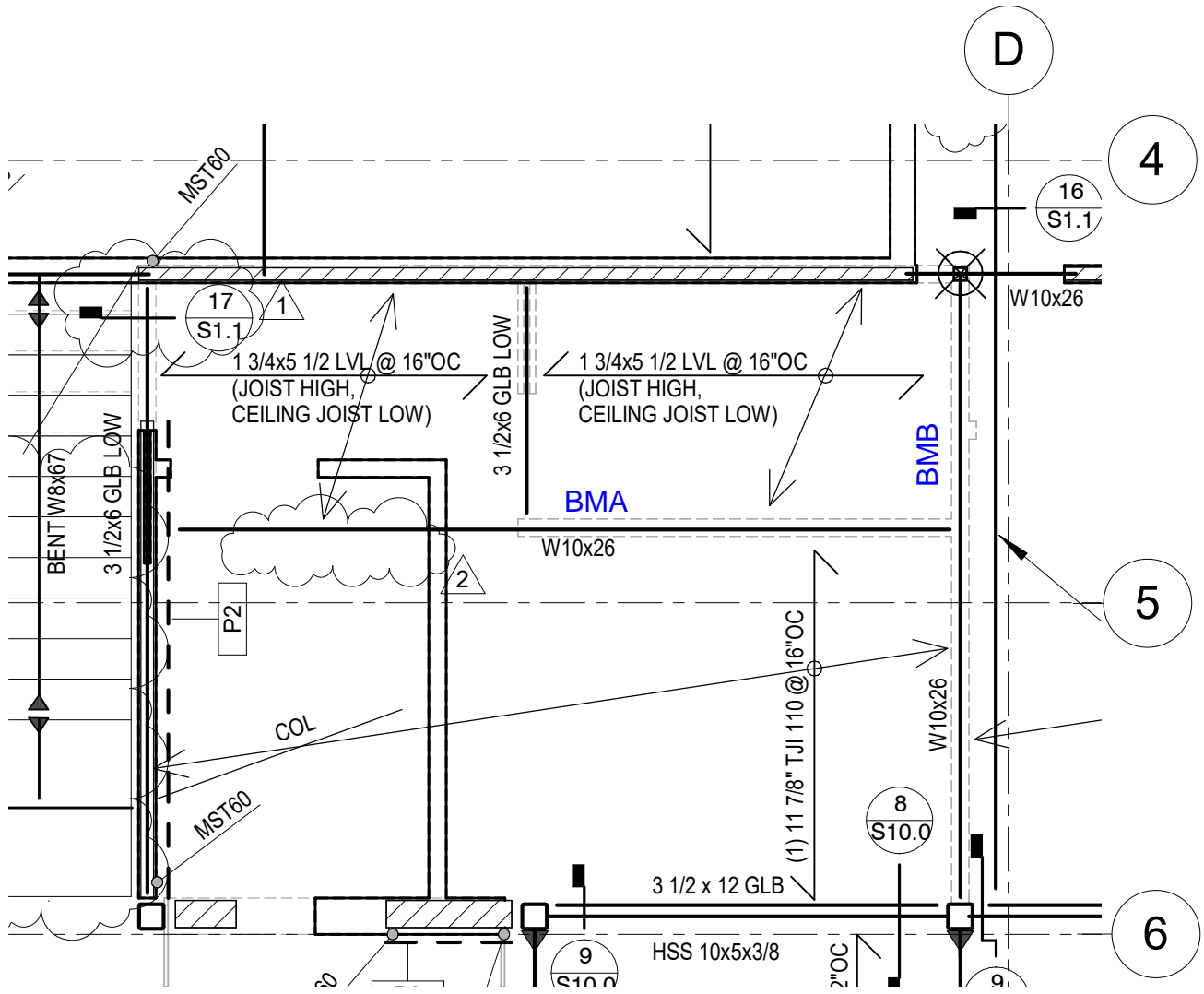
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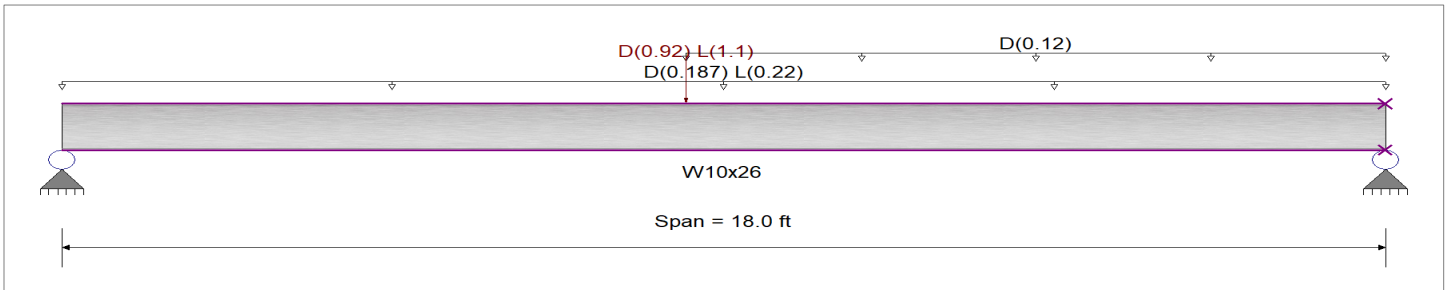
DESCRIPTION: BMA (over library)

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
 Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Strength Design
 Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
 Bending Axis : Major Axis Bending
 Fy : Steel Yield : 50.0 ksi
 E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading
 Loads on all spans...

Uniform Load on ALL spans : L = 0.040 ksf, Tributary Width = 0.0 ft

Uniform Load : D = 0.1870, L = 0.220 k/ft, Tributary Width = 1.0 ft, (floor)

Uniform Load : D = 0.120 k/ft, Extent = 8.50 --> 18.0 ft, Tributary Width = 1.0 ft, (wall)

Point Load : D = 0.920, L = 1.10 k @ 8.50 ft, (beam)

DESIGN SUMMARY

Design OK

| | | | |
|-----------------------------------|------------------|------------------------------|------------------|
| Maximum Bending Stress Ratio = | 0.372 : 1 | Maximum Shear Stress Ratio = | 0.106 : 1 |
| Section used for this span | W10x26 | Section used for this span | W10x26 |
| Ma : Applied | 29.081 k-ft | Va : Applied | 5.689 k |
| Mn / Omega : Allowable | 78.094 k-ft | Vn/Omega : Allowable | 53.560 k |
| Load Combination | +D+L+H | Load Combination | +D+L+H |
| Location of maximum on span | 8.537ft | Location of maximum on span | 18.000 ft |
| Span # where maximum occurs | Span # 1 | Span # where maximum occurs | Span # 1 |
| Maximum Deflection | | | |
| Max Downward Transient Deflection | 0.180 in | Ratio = | 1,197 >=360 |
| Max Upward Transient Deflection | 0.000 in | Ratio = | 0 <360 |
| Max Downward Total Deflection | 0.385 in | Ratio = | 562 >=240. |
| Max Upward Total Deflection | 0.000 in | Ratio = | 0 <240.0 |

Maximum Forces & Stresses for Load Combinations

| Load Combination | Segment Length | Span # | Max Stress Ratios | | Summary of Moment Values | | | | | | Summary of Shear Values | | | |
|------------------|--------------------|--------|-------------------|-------|--------------------------|--------|--------|--------|-----------|------|-------------------------|--------|-------|-----------|
| | | | M | V | Mmax + | Mmax - | Ma Max | Mnx | Mnx/Omega | Cb | Rm | Va Max | Vnx | Vnx/Omega |
| +D+L+H | Dsgn. L = 18.00 ft | 1 | 0.372 | 0.106 | 29.08 | | 29.08 | 130.42 | 78.09 | 1.00 | 1.00 | 5.69 | 80.34 | 53.56 |

Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|------------------|------|---------------|------------------|------------------|---------------|------------------|
| +D+L+H | 1 | 0.3846 | 9.051 | | 0.0000 | 0.000 |

Vertical Reactions

| Load Combination | Support notation : Far left is #1 | | Values in KIPS | |
|------------------|-----------------------------------|-----------|----------------|--|
| | Support 1 | Support 2 | | |
| Overall MAXimum | 5.263 | 5.689 | | |
| Overall MINimum | 2.561 | 2.499 | | |
| +D+L+H | 5.263 | 5.689 | | |
| D Only | 2.703 | 3.190 | | |
| L Only | 2.561 | 2.499 | | |

Title Block Line 1
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 Title Block Line 6

Project Title:
 Engineer:
 Project ID:
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Steel Beam

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File: 20035 Foo Residence.ec6
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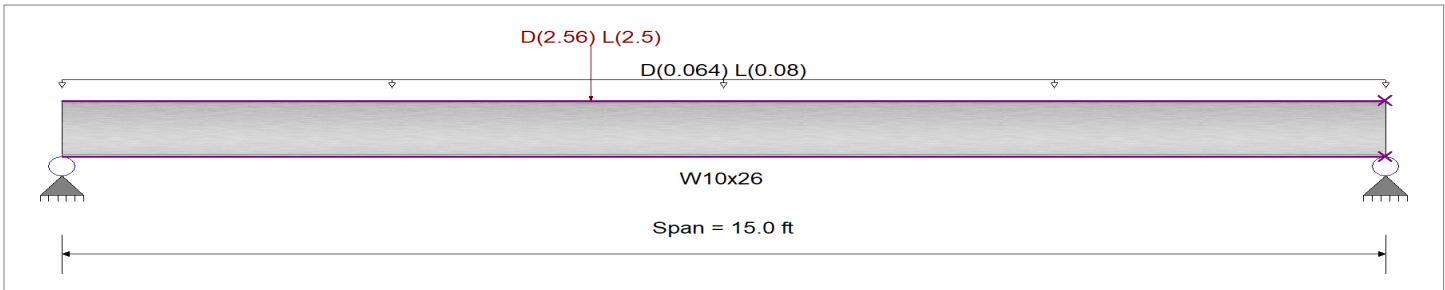
DESCRIPTION: BMB (over library)

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
 Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Strength Design
 Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
 Bending Axis : Major Axis Bending
 Fy : Steel Yield : 50.0 ksi
 E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading
 Uniform Load : D = 0.0640, L = 0.080 k/ft, Tributary Width = 1.0 ft, (floor)

Point Load : D = 2.560, L = 2.50 k @ 6.0 ft, (bma)

DESIGN SUMMARY

Design OK

| | | | |
|-----------------------------------|-------------|------------------------------|-------------|
| Maximum Bending Stress Ratio = | 0.292 : 1 | Maximum Shear Stress Ratio = | 0.080 : 1 |
| Section used for this span | W10x26 | Section used for this span | W10x26 |
| Ma : Applied | 22.803 k-ft | Va : Applied | 4.310 k |
| Mn / Omega : Allowable | 78.094 k-ft | Vn/Omega : Allowable | 53.560 k |
| Load Combination | +D+L+H | Load Combination | +D+L+H |
| Location of maximum on span | 6.000ft | 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 | 0.091 in | Ratio = | 1,974 >=360 |
| Max Upward Transient Deflection | 0.000 in | Ratio = | 0 <360 |
| Max Downward Total Deflection | 0.187 in | Ratio = | 964 >=240. |
| Max Upward Total Deflection | 0.000 in | Ratio = | 0 <240.0 |

Maximum Forces & Stresses for Load Combinations

| Load Combination | Segment Length | Span # | Max Stress Ratios | | Summary of Moment Values | | | | | | Summary of Shear Values | | | |
|------------------|--------------------|--------|-------------------|-------|--------------------------|--------|--------|--------|-----------|------|-------------------------|--------|-------|-----------|
| | | | M | V | Mmax + | Mmax - | Ma Max | Mnx | Mnx/Omega | Cb | Rm | Va Max | Vnx | Vnx/Omega |
| +D+L+H | Dsgn. L = 15.00 ft | 1 | 0.292 | 0.080 | 22.80 | | 22.80 | 130.42 | 78.09 | 1.00 | 1.00 | 4.31 | 80.34 | 53.56 |

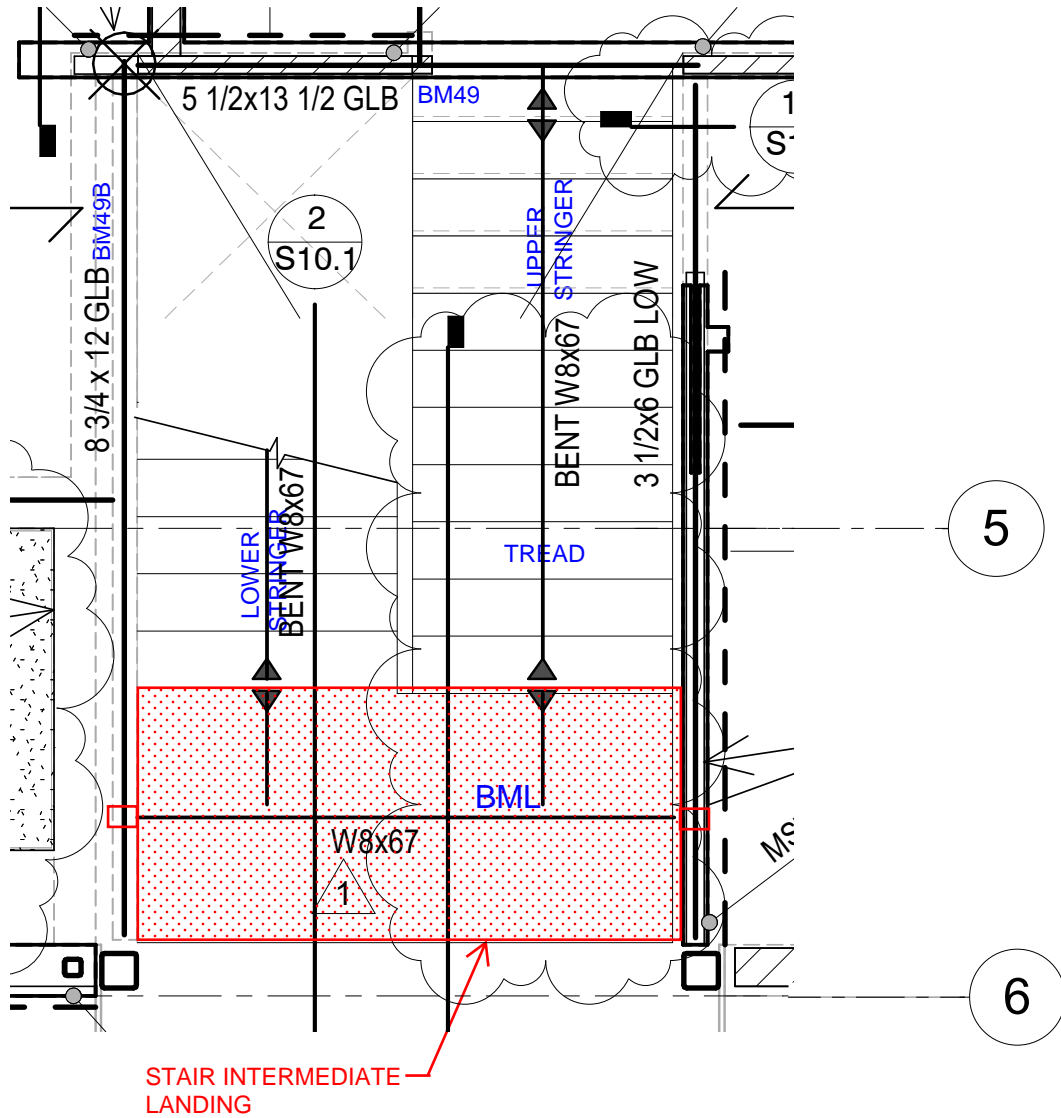
Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|------------------|------|---------------|------------------|------------------|---------------|------------------|
| +D+L+H | 1 | 0.1867 | 7.200 | | 0.0000 | 0.000 |

Vertical Reactions

| Load Combination | Support notation : Far left is #1 | | Values in KIPS | |
|------------------|-----------------------------------|-----------|----------------|--|
| | Support 1 | Support 2 | | |
| Overall MAXimum | 4.310 | 3.298 | | |
| Overall MINimum | 2.100 | 1.600 | | |
| +D+L+H | 4.310 | 3.298 | | |
| D Only | 2.210 | 1.698 | | |
| L Only | 2.100 | 1.600 | | |

STEEL STAIRS CALCULATIONS



STEEL STAIR FRAMING PLAN

Title Block Line 1
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 Title Block Line 6

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Printed: 7 MAR 2021, 10:03AM

Steel Beam

Lic. #: KW-06002997

File: 20035 Foo Residence.ec6
 Software copyright ENERCALC, INC. 1983-2020, Build:12.20.8.24
 C.T. ENGINEERING

DESCRIPTION: Stair Tread (Uniform Live Load)

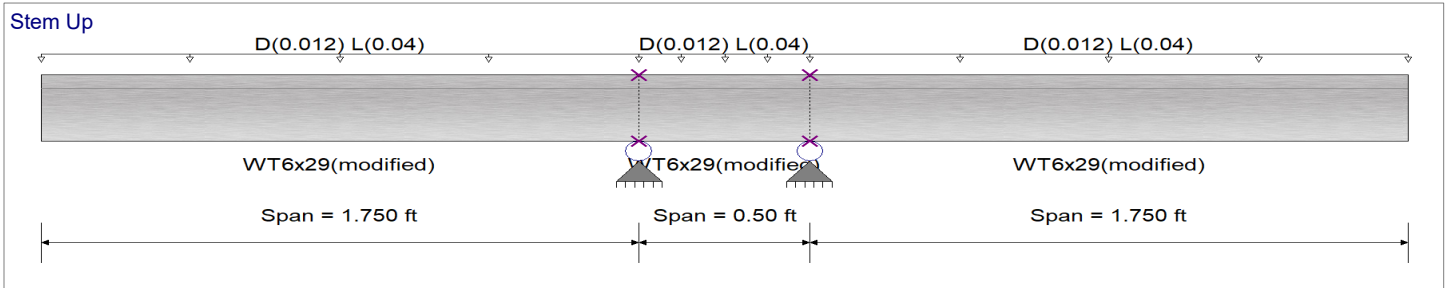
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
 Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Strength Design
 Beam Bracing : Completely Unbraced
 Bending Axis : Major Axis Bending

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



Applied Loads

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

Beam self weight calculated and added to loading

Load for Span Number 1

Uniform Load : D = 0.0120, L = 0.040 k/ft, Tributary Width = 1.0 ft

Load for Span Number 2

Uniform Load : D = 0.0120, L = 0.040 k/ft, Tributary Width = 1.0 ft

Load for Span Number 3

Uniform Load : D = 0.0120, L = 0.040 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

| | | | |
|-----------------------------------|---------------------------|------------------------------|---------------------------|
| Maximum Bending Stress Ratio = | 0.031 : 1 | Maximum Shear Stress Ratio = | 0.007 : 1 |
| Section used for this span | WT6x29(modified) | Section used for this span | WT6x29(modified) |
| Ma : Applied | 0.124 k-ft | Va : Applied | 0.1428 k |
| Mn / Omega : Allowable | 4.000 k-ft | Vn/Omega : Allowable | 20.307 k |
| Load Combination | +D+L+H, LL Comb Run (L*L) | Load Combination | +D+L+H, LL Comb Run (LL*) |
| Location of maximum on span | 0.000ft | Location of maximum on span | 0.000 ft |
| Span # where maximum occurs | Span # 2 | Span # where maximum occurs | Span # 2 |
| Maximum Deflection | | | |
| Max Downward Transient Deflection | 0.002 in | Ratio = | 25,104 >=360 |
| Max Upward Transient Deflection | 0.000 in | Ratio = | 0 <360 |
| Max Downward Total Deflection | 0.003 in | Ratio = | 12428 >=240. |
| Max Upward Total Deflection | -0.000 in | Ratio = | 68062 >=240. |

Maximum Forces & Stresses for Load Combinations

| Load Combination | Segment Length | Span # | Max Stress Ratios | | Summary of Moment Values | | | | | | Summary of Shear Values | | | |
|----------------------------|-------------------|--------|-------------------|-------|--------------------------|--------|--------|------|-----------|------|-------------------------|--------|-------|-----------|
| | | | M | V | Mmax + | Mmax - | Ma Max | Mnx | Mnx/Omega | Cb | Rm | Va Max | Vnx | Vnx/Omega |
| +D+L+H, LL Comb Run (**L) | Dsgn. L = 1.74 ft | 1 | 0.015 | 0.004 | | -0.06 | 0.06 | 6.68 | 4.00 | 1.00 | 1.00 | 0.07 | 33.91 | 20.31 |
| | Dsgn. L = 0.51 ft | 2 | 0.031 | 0.007 | -0.00 | -0.12 | 0.12 | 6.68 | 4.00 | 1.26 | 1.00 | 0.14 | 33.91 | 20.31 |
| | Dsgn. L = 1.75 ft | 3 | 0.031 | 0.007 | | -0.12 | 0.12 | 6.68 | 4.00 | 1.00 | 1.00 | 0.14 | 33.91 | 20.31 |
| +D+L+H, LL Comb Run (*L*) | Dsgn. L = 1.74 ft | 1 | 0.015 | 0.004 | | -0.06 | 0.06 | 6.68 | 4.00 | 1.00 | 1.00 | 0.07 | 33.91 | 20.31 |
| | Dsgn. L = 0.51 ft | 2 | 0.016 | 0.004 | -0.00 | -0.06 | 0.06 | 6.68 | 4.00 | 1.03 | 1.00 | 0.07 | 33.91 | 20.31 |
| | Dsgn. L = 1.75 ft | 3 | 0.016 | 0.004 | | -0.06 | 0.06 | 6.68 | 4.00 | 1.00 | 1.00 | 0.07 | 33.91 | 20.31 |
| +D+L+H, LL Comb Run (**LL) | Dsgn. L = 1.74 ft | 1 | 0.015 | 0.004 | | -0.06 | 0.06 | 6.68 | 4.00 | 1.00 | 1.00 | 0.07 | 33.91 | 20.31 |
| | Dsgn. L = 0.51 ft | 2 | 0.031 | 0.007 | -0.00 | -0.12 | 0.12 | 6.68 | 4.00 | 1.27 | 1.00 | 0.14 | 33.91 | 20.31 |
| | Dsgn. L = 1.75 ft | 3 | 0.031 | 0.007 | | -0.12 | 0.12 | 6.68 | 4.00 | 1.00 | 1.00 | 0.14 | 33.91 | 20.31 |
| +D+L+H, LL Comb Run (L**) | Dsgn. L = 1.74 ft | 1 | 0.031 | 0.007 | | -0.12 | 0.12 | 6.68 | 4.00 | 1.00 | 1.00 | 0.14 | 33.91 | 20.31 |
| | Dsgn. L = 0.51 ft | 2 | 0.031 | 0.007 | -0.00 | -0.12 | 0.12 | 6.68 | 4.00 | 1.25 | 1.00 | 0.14 | 33.91 | 20.31 |

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Title Block Line 1
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 Title Block Line 6

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 Engineer:
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Steel Beam

File: 20035 Foo Residence.ec6
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C.T. ENGINEERING

Lic. #: KW-06002997

DESCRIPTION: Stair Tread (Uniform Live Load)

| Load Combination | Segment Length | Span # | Max Stress Ratios | | Summary of Moment Values | | | | | | Summary of Shear Values | | | |
|----------------------------------|----------------|--------|-------------------|-------|--------------------------|--------|--------|------|-----------|------|-------------------------|--------|-------|-----------|
| | | | M | V | Mmax + | Mmax - | Ma Max | Mnx | Mnx/Omega | Cb | Rm | Va Max | Vnx | Vnx/Omega |
| Dsgn. L = 1.75 ft | | 3 | 0.016 | 0.004 | | -0.06 | 0.06 | 6.68 | 4.00 | 1.00 | 1.00 | 0.07 | 33.91 | 20.31 |
| +D+L+H, LL Comb Run (L*L) | | | | | | | | | | | | | | |
| Dsgn. L = 1.74 ft | | 1 | 0.031 | 0.007 | | -0.12 | 0.12 | 6.68 | 4.00 | 1.00 | 1.00 | 0.14 | 33.91 | 20.31 |
| Dsgn. L = 0.51 ft | | 2 | 0.031 | 0.007 | -0.00 | -0.12 | 0.12 | 6.68 | 4.00 | 1.01 | 1.00 | 0.14 | 33.91 | 20.31 |
| Dsgn. L = 1.75 ft | | 3 | 0.031 | 0.007 | | -0.12 | 0.12 | 6.68 | 4.00 | 1.00 | 1.00 | 0.14 | 33.91 | 20.31 |
| +D+L+H, LL Comb Run (LL*) | | | | | | | | | | | | | | |
| Dsgn. L = 1.74 ft | | 1 | 0.031 | 0.007 | | -0.12 | 0.12 | 6.68 | 4.00 | 1.00 | 1.00 | 0.14 | 33.91 | 20.31 |
| Dsgn. L = 0.51 ft | | 2 | 0.031 | 0.007 | -0.00 | -0.12 | 0.12 | 6.68 | 4.00 | 1.26 | 1.00 | 0.14 | 33.91 | 20.31 |
| Dsgn. L = 1.75 ft | | 3 | 0.016 | 0.004 | | -0.06 | 0.06 | 6.68 | 4.00 | 1.00 | 1.00 | 0.07 | 33.91 | 20.31 |
| +D+L+H, LL Comb Run (LLL) | | | | | | | | | | | | | | |
| Dsgn. L = 1.74 ft | | 1 | 0.031 | 0.007 | | -0.12 | 0.12 | 6.68 | 4.00 | 1.00 | 1.00 | 0.14 | 33.91 | 20.31 |
| Dsgn. L = 0.51 ft | | 2 | 0.031 | 0.007 | -0.00 | -0.12 | 0.12 | 6.68 | 4.00 | 1.01 | 1.00 | 0.14 | 33.91 | 20.31 |
| Dsgn. L = 1.75 ft | | 3 | 0.031 | 0.007 | | -0.12 | 0.12 | 6.68 | 4.00 | 1.00 | 1.00 | 0.14 | 33.91 | 20.31 |

Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|------------------|------|---------------|------------------|------------------|---------------|------------------|
| +D+L+H | 1 | 0.0034 | 0.000 | | 0.0000 | 0.000 |
| | 2 | 0.0000 | 0.000 | +D+L+H | -0.0001 | 0.250 |
| +D+L+H | 3 | 0.0034 | 1.750 | | 0.0000 | 0.250 |

Vertical Reactions

| Load Combination | Support notation : Far left is #1 | | | | Values in KIPS |
|---------------------------|-----------------------------------|-----------|-----------|-----------|----------------|
| | Support 1 | Support 2 | Support 3 | Support 4 | |
| Overall MAXimum | | 0.285 | 0.285 | | |
| Overall MINimum | | 0.010 | 0.010 | | |
| +D+L+H, LL Comb Run (**L) | | -0.040 | 0.275 | | |
| +D+L+H, LL Comb Run (*L*) | | 0.092 | 0.092 | | |
| +D+L+H, LL Comb Run (*LL) | | -0.030 | 0.285 | | |
| +D+L+H, LL Comb Run (L**) | | 0.275 | -0.041 | | |
| +D+L+H, LL Comb Run (L*L) | | 0.152 | 0.152 | | |
| +D+L+H, LL Comb Run (LL*) | | 0.285 | -0.031 | | |
| +D+L+H, LL Comb Run (LLL) | | 0.162 | 0.162 | | |
| D Only | | 0.082 | 0.082 | | |
| L Only, LL Comb Run (**L) | | -0.123 | 0.193 | | |
| L Only, LL Comb Run (*L*) | | 0.010 | 0.010 | | |
| L Only, LL Comb Run (*LL) | | -0.113 | 0.203 | | |
| L Only, LL Comb Run (L**) | | 0.193 | -0.123 | | |
| L Only, LL Comb Run (L*L) | | 0.070 | 0.070 | | |
| L Only, LL Comb Run (LL*) | | 0.203 | -0.113 | | |
| L Only, LL Comb Run (LLL) | | 0.080 | 0.080 | | |

Title Block Line 1
 You can change this area
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 and then using the "Printing &
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 Title Block Line 6

Project Title:
 Engineer:
 Project ID:
 Project Descr:

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Steel Beam

Lic. #: KW-06002997

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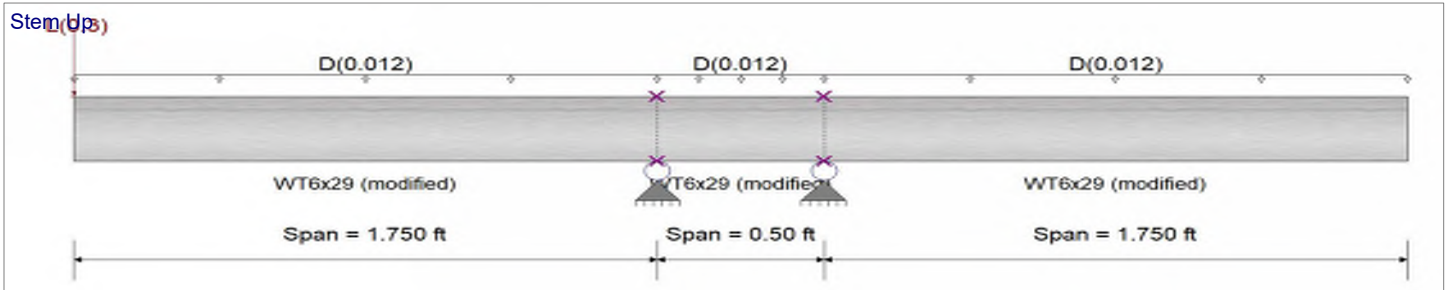
DESCRIPTION: Stair Tread (Concentrated Live Load)

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
 Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Strength Design
 Beam Bracing : Completely Unbraced
 Bending Axis : Major Axis Bending
 Fy : Steel Yield : 50.0 ksi
 E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading
 Load for Span Number 1
 Uniform Load : D = 0.0120 k/ft, Tributary Width = 1.0 ft

Point Load : L = 0.30 k @ 0.0 ft

Load for Span Number 2
 Uniform Load : D = 0.0120 k/ft, Tributary Width = 1.0 ft

Load for Span Number 3
 Uniform Load : D = 0.0120 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

| | | | |
|-----------------------------------|--------------------------|------------------------------|--------------------------|
| Maximum Bending Stress Ratio = | 0.147 : 1 | Maximum Shear Stress Ratio = | 0.052 : 1 |
| Section used for this span | WT6x29 (modified) | Section used for this span | WT6x29 (modified) |
| Ma : Applied | 0.588 k-ft | Va : Applied | 1.060 k |
| Mn / Omega : Allowable | 4.000 k-ft | Vn/Omega : Allowable | 20.307 k |
| Load Combination | +D+L+H | Load Combination | +D+L+H |
| Location of maximum on span | 0.000ft | Location of maximum on span | 0.000 ft |
| Span # where maximum occurs | Span # 2 | Span # where maximum occurs | Span # 2 |
| Maximum Deflection | | | |
| Max Downward Transient Deflection | 0.016 in | Ratio = | 2,681 >=360 |
| Max Upward Transient Deflection | 0.000 in | Ratio = | 0 <360 |
| Max Downward Total Deflection | 0.017 in | Ratio = | 2418 >=240. |
| Max Upward Total Deflection | -0.000 in | Ratio = | 25283 >=240. |

Maximum Forces & Stresses for Load Combinations

| Load Combination | Segment Length | Span # | Max Stress Ratios | | Summary of Moment Values | | | | | Summary of Shear Values | | | | |
|------------------|----------------|--------|-------------------|-------|--------------------------|--------|--------|------|-----------|-------------------------|------|--------|-------|-----------|
| | | | M | V | Mmax + | Mmax - | Ma Max | Mnx | Mnx/Omega | Cb | Rm | Va Max | Vnx | Vnx/Omega |
| +D+L+H | | | | | | | | | | | | | | |
| Dsgn. L = | 1.74 ft | 1 | 0.146 | 0.018 | | -0.58 | 0.58 | 6.68 | 4.00 | 1.00 | 1.00 | 0.37 | 33.91 | 20.31 |
| Dsgn. L = | 0.51 ft | 2 | 0.147 | 0.052 | -0.00 | -0.59 | 0.59 | 6.68 | 4.00 | 1.55 | 1.00 | 1.06 | 33.91 | 20.31 |
| Dsgn. L = | 1.75 ft | 3 | 0.016 | 0.004 | | -0.06 | 0.06 | 6.68 | 4.00 | 1.00 | 1.00 | 0.07 | 33.91 | 20.31 |

Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|------------------|------|---------------|------------------|------------------|---------------|------------------|
| +D+L+H | 1 | 0.0174 | 0.000 | | 0.0000 | 0.000 |
| | 2 | 0.0000 | 0.000 | +D+L+H | -0.0002 | 0.220 |
| | 3 | 0.0034 | 1.750 | | 0.0000 | 0.220 |

Title Block Line 1
You can change this area
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and then using the "Printing &
Title Block" selection.
Title Block Line 6

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 7 MAR 2021, 10:02AM

Steel Beam

File: 20035 Foo Residence.ec6

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Lic. #: KW-06002997

C.T. ENGINEERING

DESCRIPTION: Stair Tread (Concentrated Live Load)

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

| Load Combination | Support 1 | Support 2 | Support 3 | Support 4 |
|------------------|-----------|-----------|-----------|-----------|
| Overall MAXimum | | 1.432 | -1.050 | |
| Overall MINimum | | 0.082 | 0.082 | |
| +D+L+H | | 1.432 | -0.968 | |
| D Only | | 0.082 | 0.082 | |
| L Only | | 1.350 | -1.050 | |

Title Block Line 1
 You can change this area
 using the "Settings" menu item
 and then using the "Printing &
 Title Block" selection.
 Title Block Line 6

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Printed: 7 MAR 2021, 10:00AM

General Section Property Calculator

File: 20035 Foo Residence.ec6
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C.T. ENGINEERING

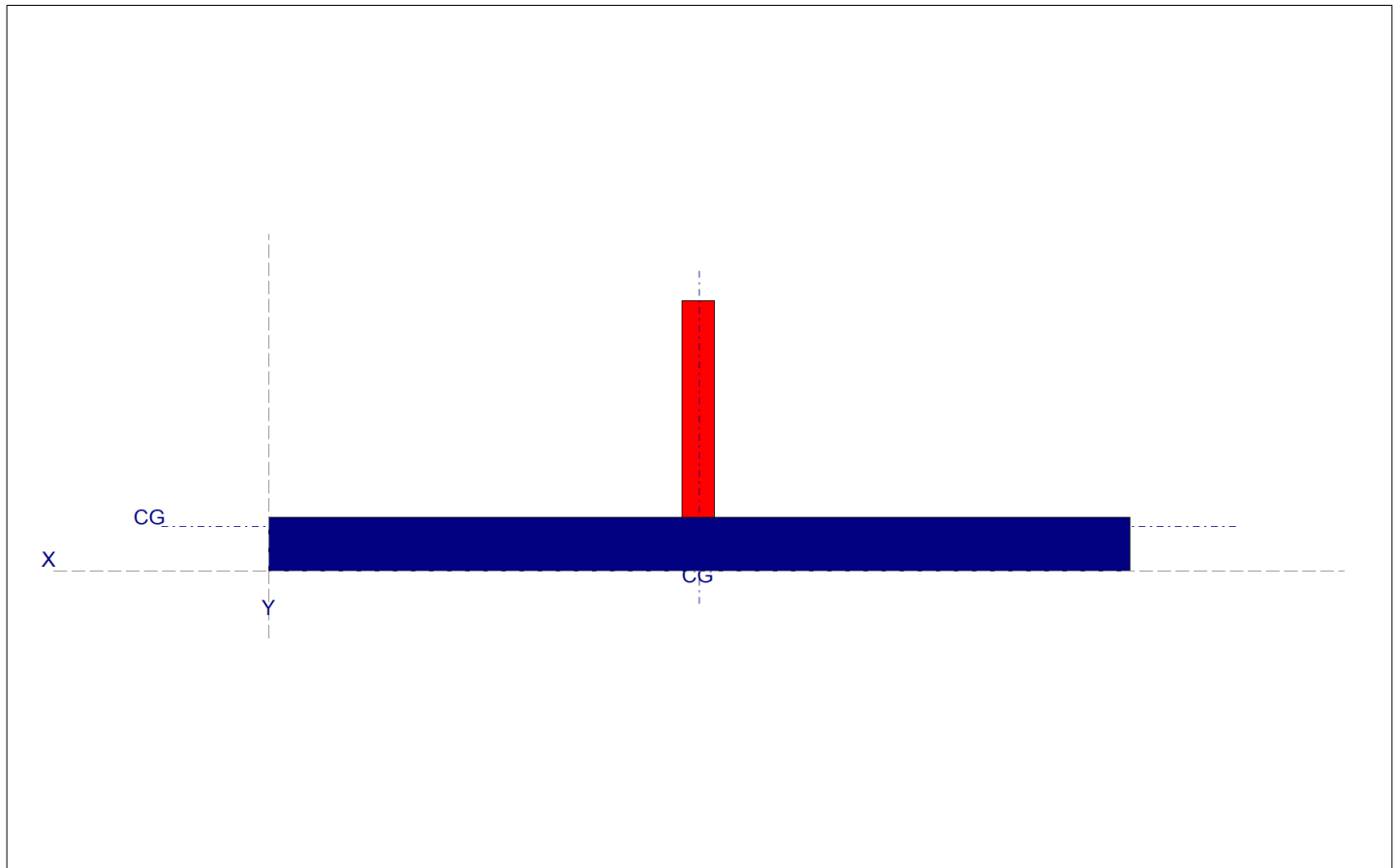
Lic. #: KW-06002997

DESCRIPTION: Tread Beam Section Property


Final Section Properties

| | | | | | |
|---|----------------------|-------|------------------------|-------------|------------------------|
| Total Area : | 7.30 in ² | lxx : | 2.632 in ⁴ | Sxx : - Y : | 5.125 in ³ |
| Calculated final C.G. distance from Datum : | | lyy : | 53.341 in ⁴ | Sxx : +Y : | 1.002 in ³ |
| X cg Dist. : | 5.0 in | Zxx : | 2.417 in ³ | Syy : - X : | 10.668 in ³ |
| Y cg Dist. : | 0.5136 in | Zyy : | 16.081 in ³ | Syy : +X : | 10.668 in ³ |
| Edge Distances from CG. : | | | | r xx : | 0.6005 in |
| +X : | 5.0 in | +Y : | 2.626 in | r yy : | 2.703 in |
| -X : | -5.0 in | -Y : | in | | |

Rotation of All Components @ Angle : 0.00 deg CCW



Rectangular & Circular Shapes

| | | | | | | | |
|--|-----------------------|----------|-----------------------|---------|-----------|------------|-----------|
|  | Rectangular Shape : 1 | Height = | 0.640 in | Width = | 10.000 in | Rotation = | 0 deg CCW |
| | | Area = | 6.400 in ² | Xcg = | 5.000 in | Ycg = | 0.320 in |

Title Block Line 1
You can change this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 7 MAR 2021, 10:00AM

General Section Property Calculator


File: 20035 Foo Residence.ec6

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C.T. ENGINEERING

DESCRIPTION: Tread Beam Section Property

| | | | | | | |
|--|----------|-----------------------|---------|----------|------------|-----------|
|  Rectangular Shape : 2 | Height = | 2.500 in | Width = | 0.360 in | Rotation = | 0 deg CCW |
| | Area = | 0.900 in ² | Xcg = | 5.000 in | Ycg = | 1.890 in |

Title Block Line 1
 You can change this area
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 and then using the "Printing &
 Title Block" selection.
 Title Block Line 6

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Printed: 7 MAR 2021, 10:04AM

Steel Beam

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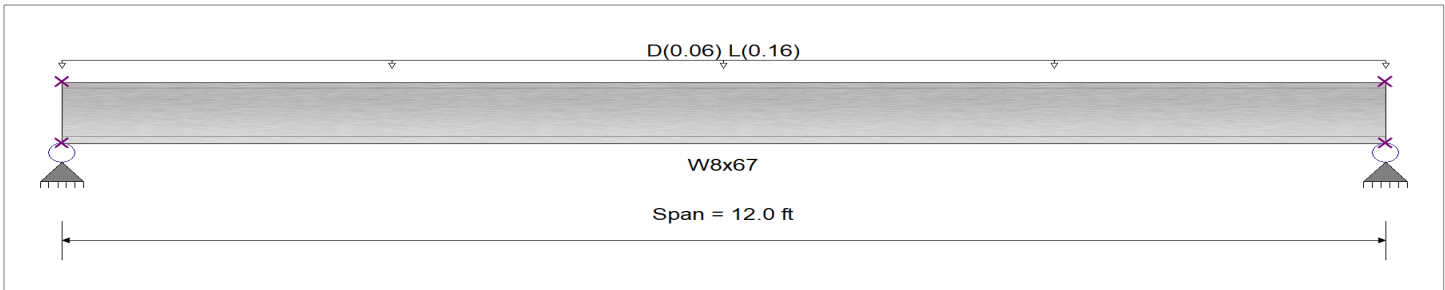
DESCRIPTION: Upper Stair Stringer

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
 Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Strength Design
 Beam Bracing : Completely Unbraced
 Bending Axis : Major Axis Bending
 Fy : Steel Yield : 50.0 ksi
 E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading
 Uniform Load : D = 0.060, L = 0.160 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

| | | | |
|-----------------------------------|------------------|------------------------------|------------------|
| Maximum Bending Stress Ratio = | 0.030 : 1 | Maximum Shear Stress Ratio = | 0.017 : 1 |
| Section used for this span | W8x67 | Section used for this span | W8x67 |
| Ma : Applied | 5.166 k-ft | Va : Applied | 1.722 k |
| Mn / Omega : Allowable | 174.900 k-ft | Vn/Omega : Allowable | 102.60 k |
| Load Combination | +D+L+H | Load Combination | +D+L+H |
| Location of maximum on span | 6.000ft | 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 | 0.010 in | Ratio = | 15,146 >=360 |
| Max Upward Transient Deflection | 0.000 in | Ratio = | 0 <360 |
| Max Downward Total Deflection | 0.017 in | Ratio = | 8444 >=240. |
| Max Upward Total Deflection | 0.000 in | Ratio = | 0 <240.0 |

Maximum Forces & Stresses for Load Combinations

| Load Combination | Segment Length | Span # | Max Stress Ratios | | Summary of Moment Values | | | | | | Summary of Shear Values | | | |
|------------------|--------------------|--------|-------------------|-------|--------------------------|--------|--------|--------|-----------|------|-------------------------|--------|--------|-----------|
| | | | M | V | Mmax + | Mmax - | Ma Max | Mnx | Mnx/Omega | Cb | Rm | Va Max | Vnx | Vnx/Omega |
| +D+L+H | Dsgn. L = 12.00 ft | 1 | 0.030 | 0.017 | 5.17 | | 5.17 | 292.08 | 174.90 | 1.14 | 1.00 | 1.72 | 153.90 | 102.60 |

Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|------------------|------|---------------|------------------|------------------|---------------|------------------|
| +D+L+H | 1 | 0.0171 | 6.034 | | 0.0000 | 0.000 |

Vertical Reactions

| Load Combination | Support notation : Far left is #1 | | Values in KIPS | |
|------------------|-----------------------------------|-----------|----------------|--|
| | Support 1 | Support 2 | | |
| Overall MAXimum | 1.722 | 1.722 | | |
| Overall MINimum | 0.762 | 0.762 | | |
| +D+L+H | 1.722 | 1.722 | | |
| D Only | 0.762 | 0.762 | | |
| L Only | 0.960 | 0.960 | | |

Title Block Line 1
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 Title Block Line 6

Project Title:
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 Project ID:
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Steel Beam

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File: 20035 Foo Residence.ec6
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 C.T. ENGINEERING

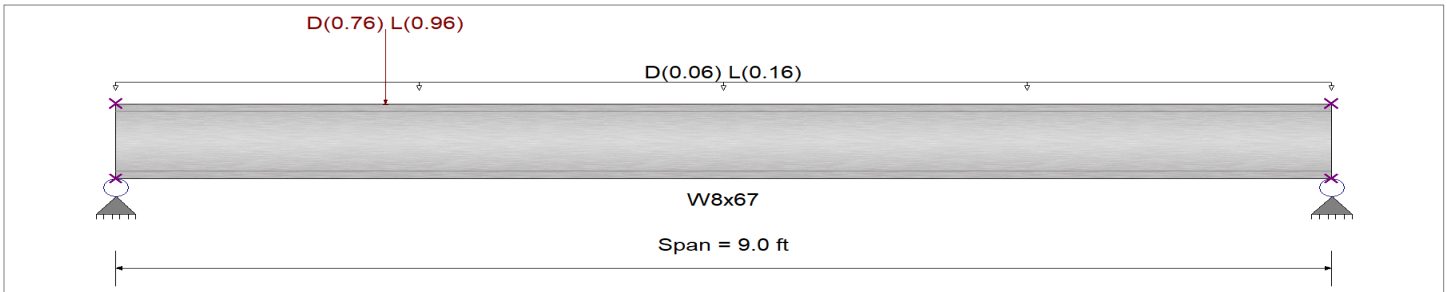
DESCRIPTION: Lower Stair Stringer

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
 Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Strength Design
 Beam Bracing : Completely Unbraced
 Bending Axis : Major Axis Bending
 Fy : Steel Yield : 50.0 ksi
 E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading
 Uniform Load : D = 0.060, L = 0.160 k/ft, Tributary Width = 1.0 ft

Point Load : D = 0.760, L = 0.960 k @ 2.0 ft, (upper stair stringer)

DESIGN SUMMARY

Design OK

| | | | |
|-----------------------------------|------------------|------------------------------|------------------|
| Maximum Bending Stress Ratio = | 0.028 : 1 | Maximum Shear Stress Ratio = | 0.026 : 1 |
| Section used for this span | W8x67 | Section used for this span | W8x67 |
| Ma : Applied | 4.880 k-ft | Va : Applied | 2.629 k |
| Mn / Omega : Allowable | 174.900 k-ft | Vn/Omega : Allowable | 102.60 k |
| Load Combination | +D+L+H | Load Combination | +D+L+H |
| Location of maximum on span | 3.163ft | 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 | 0.005 in | Ratio = | 21,503 >=360 |
| Max Upward Transient Deflection | 0.000 in | Ratio = | 0 <360 |
| Max Downward Total Deflection | 0.009 in | Ratio = | 11993 >=240. |
| Max Upward Total Deflection | 0.000 in | Ratio = | 0 <240.0 |

Maximum Forces & Stresses for Load Combinations

| Load Combination | Segment Length | Span # | Max Stress Ratios | | Summary of Moment Values | | | | | | Summary of Shear Values | | | |
|------------------|-------------------|--------|-------------------|-------|--------------------------|--------|--------|--------|-----------|------|-------------------------|--------|--------|-----------|
| | | | M | V | Mmax + | Mmax - | Ma Max | Mnx | Mnx/Omega | Cb | Rm | Va Max | Vnx | Vnx/Omega |
| +D+L+H | Dsgn. L = 9.00 ft | 1 | 0.028 | 0.026 | 4.88 | | 4.88 | 292.08 | 174.90 | 1.13 | 1.00 | 2.63 | 153.90 | 102.60 |

Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|------------------|------|---------------|------------------|------------------|---------------|------------------|
| +D+L+H | 1 | 0.0090 | 4.294 | | 0.0000 | 0.000 |

Vertical Reactions

| Load Combination | Support 1 | Support 2 |
|------------------|-----------|-----------|
| Overall MAXimum | 2.629 | 1.674 |
| Overall MINimum | 1.163 | 0.740 |
| +D+L+H | 2.629 | 1.674 |
| D Only | 1.163 | 0.740 |
| L Only | 1.467 | 0.933 |

Title Block Line 1
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 Title Block" selection.
 Title Block Line 6

Project Title:
 Engineer:
 Project ID:
 Project Descr:

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Steel Beam

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File: 20035 Foo Residence.ecb
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 C.T. ENGINEERING

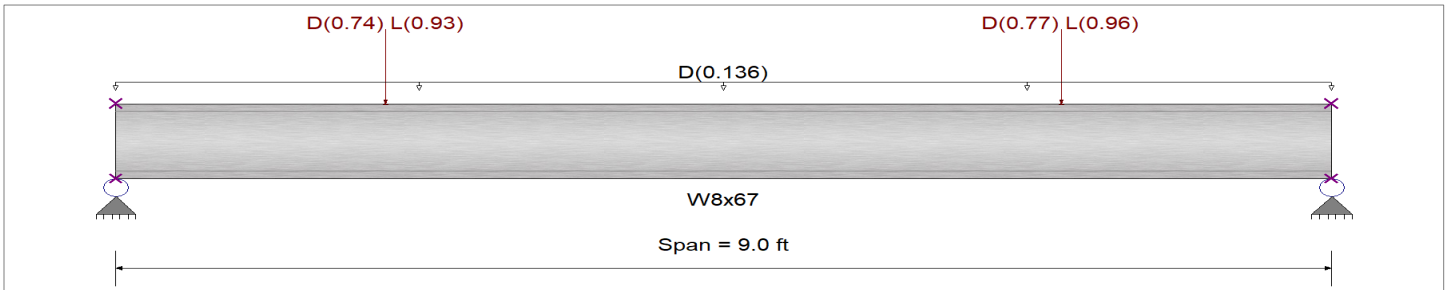
DESCRIPTION: BML

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
 Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Strength Design
 Beam Bracing : Completely Unbraced
 Bending Axis : Major Axis Bending
 Fy : Steel Yield : 50.0 ksi
 E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading
 Uniform Load : D = 0.1360 k/ft, Tributary Width = 1.0 ft

Point Load : D = 0.740, L = 0.930 k @ 2.0 ft, (lower stringer)

Point Load : D = 0.770, L = 0.960 k @ 7.0 ft, (upper stringer)

DESIGN SUMMARY

Design OK

| | | | |
|-----------------------------------|------------------|------------------------------|------------------|
| Maximum Bending Stress Ratio = | 0.031 : 1 | Maximum Shear Stress Ratio = | 0.026 : 1 |
| Section used for this span | W8x67 | Section used for this span | W8x67 |
| Ma : Applied | 5.456 k-ft | Va : Applied | 2.630 k |
| Mn / Omega : Allowable | 174.900 k-ft | Vn/Omega : Allowable | 102.60 k |
| Load Combination | +D+L+H | Load Combination | +D+L+H |
| Location of maximum on span | 4.577 ft | Location of maximum on span | 9.000 ft |
| Span # where maximum occurs | Span # 1 | Span # where maximum occurs | Span # 1 |
| Maximum Deflection | | | |
| Max Downward Transient Deflection | 0.004 in | Ratio = | 27,447 >=360 |
| Max Upward Transient Deflection | 0.000 in | Ratio = | 0 <360 |
| Max Downward Total Deflection | 0.011 in | Ratio = | 9912 >=240. |
| Max Upward Total Deflection | 0.000 in | Ratio = | 0 <240.0 |

Maximum Forces & Stresses for Load Combinations

| Load Combination | Segment Length | Span # | Max Stress Ratios | | Summary of Moment Values | | | | | | Summary of Shear Values | | | |
|------------------|-------------------|--------|-------------------|-------|--------------------------|--------|--------|--------|-----------|------|-------------------------|--------|--------|-----------|
| | | | M | V | Mmax + | Mmax - | Ma Max | Mnx | Mnx/Omega | Cb | Rm | Va Max | Vnx | Vnx/Omega |
| +D+L+H | Dsgn. L = 9.00 ft | 1 | 0.031 | 0.026 | 5.46 | | 5.46 | 292.08 | 174.90 | 1.05 | 1.00 | 2.63 | 153.90 | 102.60 |

Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|------------------|------|---------------|------------------|------------------|---------------|------------------|
| +D+L+H | 1 | 0.0109 | 4.526 | | 0.0000 | 0.000 |

Vertical Reactions

| Load Combination | Support notation : Far left is #1 | | Values in KIPS | |
|------------------|-----------------------------------|-----------|----------------|--|
| | Support 1 | Support 2 | | |
| Overall MAXimum | 2.597 | 2.630 | | |
| Overall MINimum | 0.937 | 0.953 | | |
| +D+L+H | 2.597 | 2.630 | | |
| D Only | 1.660 | 1.677 | | |
| L Only | 0.937 | 0.953 | | |

GUARDRAIL CALCULATIONS

Project: _____

Date: _____

Client: _____

Page Number: _____

GUARDRAIL DESIGN

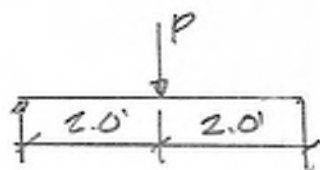
* 200# ANY DIRECTION C TOP RAIL & POST

* 50# IN A 12" x 12" AREA INTERMEDIATE

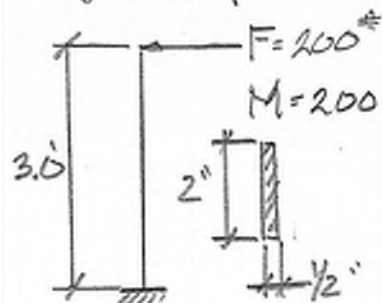
TOP RAIL: MAX SPAN = 4.0'

$$M = 200(4)/4 = 200 \#-1$$

$$M_n / S_x = 1.83 \#-1 \rightarrow M_n = 1.83 / 1.67 = 1100 \#-1$$

USE HSS 2x2x3/16

GUARD POST: (2x 1/2" FLAT PLATE)



$$M = 200(3) = 600 \#-1$$

$$A = 1/2(2) = 1.0 \text{ in}^2$$

$$S_x = 1/2(2)^2/6 = 0.33 \text{ in}^3$$

$$I_x = 1/2(2)^3/12 = 0.33 \text{ in}^4$$

$$r_x = \sqrt{I_x/A} = 0.57 \text{ in}$$

$$Z_x = 1/2(2)^2/4 = 0.50 \text{ in}^3$$

$$S_y = 2(1.5)^2/6 = 0.08 \text{ in}^3$$

$$I_y = 2(1.5)^3/12 = 0.02 \text{ in}^4$$

$$r_y = \sqrt{I_y/A} = 0.14 \text{ in}$$

$$L_{bd}/t \leq 0.80E/F_y$$

$$L_{bd}/t = 36(2)/5 = 14.4 < 644$$

$$0.80E/F_y = 0.80(29 \times 10^3)/36 = 644$$

$$M_n = M_p = F_y Z \leq 1.6 M_y$$

$$1.6 M_y = 1.6 S_x F_y = 1.6(36 \text{ ksi})(0.33) = 19.01 \text{ k-in}$$

$$M_p = F_y Z_x = 36 \text{ ksi}(0.50) = 18 \text{ k-in} < 19.01$$

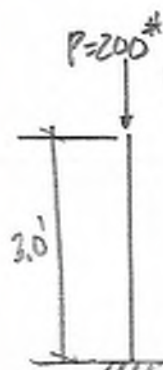
$$M_n = 18 \text{ k-in} \quad M_n / S_x = 18 / 1.67 = 10.78 \text{ k-in} = 895 \#-1 > 600 \#-1 \text{ OK}$$

$$\text{WEAK AXIS (PIN - FIX)} \quad K = 0.80 \quad K L / r_y = 0.8(36) / 0.14 = 205.7$$

$$\text{STRONG AXIS (FREE - FIX)} \quad K = 1.2 \quad K L / r_x = 1.2(36) / 0.57 = 75.4$$

$$4.71 \sqrt{E/F_y} = 4.71 \sqrt{29 \times 10^3 / 36} = 134 \rightarrow F_{cr} = \{ 0.658 F_y / r^2 \} F_y$$

$$F_c = \pi^2 E / (K L / r)^2 = \pi^2 (29 \times 10^3) / 75.4^2 = 50.32$$



Project: FOO RESIDENCE

Date: _____

Client: _____

Page Number: _____

$$F_{cr} = \frac{1}{2} (0.658)^{3/4} (50.32)^{3/4} (36) = 26.7 \text{ ksi}$$

$$P_{cap} = 26.7 (1.5) (2) = 27 \text{ k} \gg 200 \text{ k} \text{ ok}$$

BASE CONNECTION

$$M = 600 \text{ k-in} = 7200 \text{ k-ft}$$

$$T = 7200 / 3 = 2400 \text{ k} \div 2 = 1200 \text{ k}$$

$$V = 200 / 4 = 50 \text{ k}$$

$$R = \sqrt{50^2 + 1200^2} / 2 = 120 \text{ k}$$

$$3/8" \phi \text{ A307 THREADED BOLT CAP} = 20 \text{ ksi} (0.668) = 1360 \text{ k} > 120 \text{ k}$$

$$A_n = 0.668 \text{ in}^2$$

$$F_t = 20 \text{ ksi}$$

| |
|--|
| USE $3/8 \times 4 \times 0'-4"$ PLATE W/ (4) $3/8" \phi$ A307 WELD THREADED STUD |
|--|

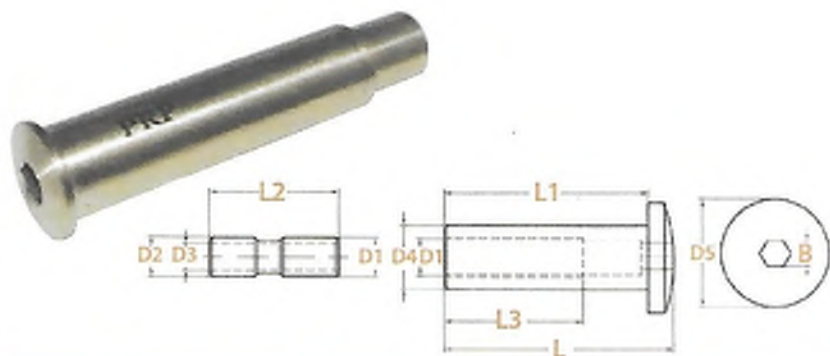
**1x19 STAINLESS STEEL
CABLE
TYPE 316, EXTRA BRIGHT
& SUPER CLEAN**



| ITEM CODE NO. | DIAMETER (IN.) | WEIGHT PER 100FT (LBS) | NOMINAL B. S (LBS) | SURFACE |
|---------------|----------------|------------------------|--------------------|----------------------|
| PRPC01191166K | 1/16 | 0.85 | 480 | Extra Bright & Clean |
| PRPC01193326K | 3/32 | 2.00 | 1,070 | Extra Bright & Clean |
| PRPC01191086K | 1/8 | 3.50 | 1,760 | Extra Bright & Clean |
| PRPC01195326K | 5/32 | 5.50 | 3,000 | Extra Bright & Clean |
| PRPC01193166K | 3/16 | 7.70 | 4,270 | Extra Bright & Clean |
| PRPC01197326K | 7/32 | 10.20 | 5,730 | Extra Bright & Clean |
| PRPC01191046K | 1/4 | 13.50 | 7,460 | Extra Bright & Clean |
| PRPC01199326K | 9/32 | 21.00 | 11,800 | Extra Bright & Clean |
| PRPC01195166K | 5/16 | 30.00 | 16,500 | Extra Bright & Clean |
| PRPC01193086K | 3/8 | 54.00 | 29,500 | Extra Bright & Clean |
| PRPC01197166K | 7/16 | 69.00 | 36,100 | Extra Bright & Clean |
| PRPC01195086K | 5/8 | 85.00 | 44,970 | Extra Bright & Clean |

Extra Bright and Perfectly Clean manufactured by KOS
Application: Sail Boat, Railing, Interior and others

**STAINLESS STEEL
STREAMLINE RECEIVER
(FLAT HEAD STYLE)
TYPE 316, MACHINED**



| ITEM CODE NO. | A (Cable) | D1 | L | L1 | L2 | L3 | D2 | D3 | D4 | D5 | B |
|--------------------|-----------|------------|------|-------|-------|-------|--------|--------|--------|-------|-------|
| S9246-A0803-L/R | 1/8" | 5/16"(UNF) | 1.55 | 1.65" | 1.00" | 1.10" | 0.218" | 0.140" | 0.437" | 0.625 | 0.19" |
| S9246-A0805-L/R | 3/16" | 5/16"(UNF) | 1.80 | 1.90" | 1.00" | 1.10" | 0.295" | 0.201" | 0.437" | 0.625 | 0.19" |
| S9246-A0805-XL-L/R | 3/16" | 5/16"(UNF) | 2.30 | 2.30" | 1.00" | 1.60" | 0.295" | 0.201" | 0.437" | 0.625 | 0.19" |
| S9246-A1106-L/R | 1/4" | 7/16"(UNF) | 2.00 | 2.00" | 1.25" | 1.38" | 0.425" | 0.265" | 0.526" | 0.75 | 0.22" |

CEILING JOIST CALCULATIONS

Project: **FOO RESIDENCE**

Date: _____

Client: _____ Page Number: _____

Ceiling Joist Maximum Span

Dead Load = 5 psf
 Light Storage = 10 psf
 Partition = 0 psf (added to live load)
 Spacing = 24 in

| Adjustment Factors | |
|--------------------|------|
| Size | 1.50 |
| Repetitive | 1.15 |
| Flat Use | 1.00 |
| Duration | 1.00 |

Member Size **2x4** b = 1.5 in.
 d = 3.5 in.

Section Properties A = 5.25 in²
 S = 3.063 in³
 I = 5.36 in⁴

Species Hem Fir No. 2

F_b = 850 psi (base) = 1275 psi
 F_v = 150 psi
 E = 1300 ksi

Load w_{DL} = 10 plf
 w_{LL} = 20 plf
 w_{TL} = 30 plf

Analysis

Bending $L = \{F_b \times S \times 8 / (12 \times w_{TL})\}^{0.5}$ L = 9.32 ft

Shear $L = \{(F_v \times A / 1.5 \times w_{TL}) + (d / 12)\} \times 2$ L = 35.58 ft

Deflection

Total Load $L = \{E \times I / (450 \times w_{TL})\}^{1/3}$ L = 8.00 ft

Live Load $L = \{E \times I / (675 \times w_{LL})\}^{1/3}$ L = 8.00 ft

2x4 Hem Fir No 2 @ 24" oc L_(MAX) = 8.0 ft

Project: **FOO RESIDENCE** Date: _____

Client: _____ Page Number: _____

Ceiling Joist Maximum Span

Dead Load = 5 psf
 Light Storage = 20 psf
 Partition = 0 psf (added to live load)
 Spacing = 24 in

| Adjustment Factors | |
|--------------------|------|
| Size | 1.50 |
| Repetitive | 1.15 |
| Flat Use | 1.00 |
| Duration | 1.00 |

Member Size **2x4** b = 1.5 in.
 d = 3.5 in.

Section Properties A = 5.25 in²
 S = 3.063 in³
 I = 5.36 in⁴

Species Hem Fir No. 2

F_b = 850 psi (base) = 1275 psi
 F_v = 150 psi
 E = 1300 ksi

Load w_{DL} = 10 plf
 w_{LL} = 40 plf
 w_{TL} = 50 plf

Analysis

Bending $L = \{F_b \times S \times 8 / (12 \times w_{TL})\}^{0.5}$ L = 7.22 ft

Shear $L = \{(F_v \times A / 1.5 \times w_{TL}) + (d / 12)\} \times 2$ L = 21.58 ft

Deflection

Total Load $L = \{E \times I / (450 \times w_{TL})\}^{1/3}$ L = 6.75 ft

Live Load $L = \{E \times I / (675 \times w_{LL})\}^{1/3}$ L = 6.35 ft

2x4 Hem Fir No 2 @ 24" oc L_(MAX) = 6.33 ft

Project: **FOO RESIDENCE** Date: _____

Client: _____ Page Number: _____

Ceiling Joist Maximum Span

Dead Load = 5 psf
 Light Storage = 10 psf
 Partition = 0 psf (added to live load)
 Spacing = 24 in

| Adjustment Factors | |
|--------------------|------|
| Size | 1.30 |
| Repetitive | 1.15 |
| Flat Use | 1.00 |
| Duration | 1.00 |

Member Size **2x6** b = 1.5 in.
 d = 5.5 in.

Section Properties A = 8.25 in²
 S = 7.563 in³
 I = 20.80 in⁴

Species Hem Fir No. 2

F_b = 850 psi (base) = 1105 psi
 F_v = 150 psi
 E = 1300 ksi

Load w_{DL} = 10 plf
 w_{LL} = 20 plf
 w_{TL} = 30 plf

Analysis

Bending $L = \{F_b \times S \times 8 / (12 \times w_{TL})\}^{0.5}$ L = 13.63 ft

Shear $L = \{(F_v \times A / 1.5 \times w_{TL}) + (d / 12)\} \times 2$ L = 55.92 ft

Deflection

Total Load $L = \{E \times I / (450 \times w_{TL})\}^{1/3}$ L = 12.57 ft

Live Load $L = \{E \times I / (675 \times w_{LL})\}^{1/3}$ L = 12.57 ft

2x6 Hem Fir No 2 @ 24" oc L_(MAX) = 12.6 ft

Project: **FOO RESIDENCE** Date: _____

Client: _____ Page Number: _____

Ceiling Joist Maximum Span

Dead Load = 5 psf
 Light Storage = 20 psf
 Partition = 0 psf (added to live load)
 Spacing = 24 in

| Adjustment Factors | |
|--------------------|------|
| Size | 1.30 |
| Repetitive | 1.15 |
| Flat Use | 1.00 |
| Duration | 1.00 |

Member Size **2x6** b = 1.5 in.
 d = 5.5 in.

Section Properties A = 8.25 in²
 S = 7.563 in³
 I = 20.80 in⁴

Species Hem Fir No. 2

F_b = 850 psi (base) = 1105 psi
 F_v = 150 psi
 E = 1300 ksi

Load w_{DL} = 10 plf
 w_{LL} = 40 plf
 w_{TL} = 50 plf

Analysis

Bending $L = \{F_b \times S \times 8 / (12 \times w_{TL})\}^{0.5}$ L = 10.56 ft

Shear $L = \{(F_v \times A / 1.5 \times w_{TL}) + (d/12)\} \times 2$ L = 33.92 ft

Deflection

Total Load $L = \{E \times I / (450 \times w_{TL})\}^{1/3}$ L = 10.61 ft

Live Load $L = \{E \times I / (675 \times w_{LL})\}^{1/3}$ L = 9.98 ft

2x6 Hem Fir No 2 @ 24" oc L_(MAX) = 10.0 ft

Project: **FOO RESIDENCE** Date: _____

Client: _____ Page Number: _____

Ceiling Joist Maximum Span

Dead Load = 5 psf
 Light Storage = 10 psf
 Partition = 0 psf (added to live load)
 Spacing = 24 in

| Adjustment Factors | |
|--------------------|------|
| Size | 1.20 |
| Repetitive | 1.15 |
| Flat Use | 1.00 |
| Duration | 1.00 |

Member Size **2x8** b = 1.5 in.
 d = 7.25 in.

Section Properties A = 10.875 in²
 S = 13.141 in³
 I = 47.63 in⁴

Species Hem Fir No. 2

F_b = 850 psi (base) = 1020 psi
 F_v = 150 psi
 E = 1300 ksi

Load w_{DL} = 10 plf
 w_{LL} = 20 plf
 w_{TL} = 30 plf

Analysis

Bending $L = \{F_b \times S \times 8 / (12 \times w_{TL})\}^{0.5}$ L = 17.26 ft

Shear $L = \{(F_v \times A / 1.5 \times w_{TL}) + (d / 12)\} \times 2$ L = 73.71 ft

Deflection

Total Load $L = \{E \times I / (450 \times w_{TL})\}^{1/3}$ L = 16.57 ft

Live Load $L = \{E \times I / (675 \times w_{LL})\}^{1/3}$ L = 16.57 ft

2x8 Hem Fir No 2 @ 24" oc L_(MAX) = 16.5 ft

Project: **FOO RESIDENCE** Date: _____

Client: _____ Page Number: _____

Ceiling Joist Maximum Span

Dead Load = 5 psf
 Light Storage = 20 psf
 Partition = 0 psf (added to live load)
 Spacing = 24 in

| Adjustment Factors | |
|--------------------|------|
| Size | 1.20 |
| Repetitive | 1.15 |
| Flat Use | 1.00 |
| Duration | 1.00 |

Member Size **2x8** b = 1.5 in.
 d = 7.25 in.

Section Properties A = 10.875 in²
 S = 13.141 in³
 I = 47.63 in⁴

Species Hem Fir No. 2

F_b = 850 psi (base) = 1020 psi
 F_v = 150 psi
 E = 1300 ksi

Load w_{DL} = 10 plf
 w_{LL} = 40 plf
 w_{TL} = 50 plf

Analysis

Bending $L = \{F_b \times S \times 8 / (12 \times w_{TL})\}^{0.5}$ L = 13.37 ft

Shear $L = \{(F_v \times A / 1.5 \times w_{TL}) + (d / 12)\} \times 2$ L = 44.71 ft

Deflection

Total Load $L = \{E \times I / (450 \times w_{TL})\}^{1/3}$ L = 13.98 ft

Live Load $L = \{E \times I / (675 \times w_{LL})\}^{1/3}$ L = 13.15 ft

2x8 Hem Fir No 2 @ 24" oc L_(MAX) = 13.0 ft

Project: **FOO RESIDENCE** Date: _____

Client: _____ Page Number: _____

Ceiling Joist Maximum Span

Dead Load = 5 psf
 Light Storage = 10 psf
 Partition = 0 psf (added to live load)
 Spacing = 24 in

| Adjustment Factors | |
|--------------------|------|
| Size | 1.10 |
| Repetitive | 1.15 |
| Flat Use | 1.00 |
| Duration | 1.00 |

Member Size **2x10** b = 1.5 in.
 d = 9.25 in.

Section Properties A = 13.875 in²
 S = 21.391 in³
 I = 98.93 in⁴

Species Hem Fir No. 2

F_b = 850 psi (base) = 935 psi
 F_v = 150 psi
 E = 1300 ksi

Load w_{DL} = 10 plf
 w_{LL} = 20 plf
 w_{TL} = 30 plf

Analysis

Bending $L = \{F_b \times S \times 8 / (12 \times w_{TL})\}^{0.5}$ L = 21.08 ft

Shear $L = \{(F_v \times A / 1.5 \times w_{TL}) + (d/12)\} \times 2$ L = 94.04 ft

Deflection

Total Load $L = \{ExI / (450 \times w_{TL})\}^{1/3}$ L = 21.13 ft

Live Load $L = \{ExI / (675 \times w_{LL})\}^{1/3}$ L = 21.13 ft

2x10 Hem Fir No 2 @ 24" oc L_(MAX) = 21.0 ft

Project: **FOO RESIDENCE** Date: _____

Client: _____ Page Number: _____

Ceiling Joist Maximum Span

Dead Load = 5 psf
 Light Storage = 20 psf
 Partition = 0 psf (added to live load)
 Spacing = 24 in

| Adjustment Factors | |
|--------------------|------|
| Size | 1.10 |
| Repetitive | 1.15 |
| Flat Use | 1.00 |
| Duration | 1.00 |

Member Size **2x10** b = 1.5 in.
 d = 9.25 in.

Section Properties A = 13.875 in²
 S = 21.391 in³
 I = 98.93 in⁴

Species Hem Fir No. 2

F_b = 850 psi (base) = 935 psi
 F_v = 150 psi
 E = 1300 ksi

Load w_{DL} = 10 plf
 w_{LL} = 40 plf
 w_{TL} = 50 plf

Analysis

Bending $L = \{F_b \times S \times 8 / (12 \times w_{TL})\}^{0.5}$ L = 16.33 ft

Shear $L = \{(F_v \times A / 1.5 \times w_{TL}) + (d/12)\} \times 2$ L = 57.04 ft

Deflection

Total Load $L = \{ExI / (450 \times w_{TL})\}^{1/3}$ L = 17.83 ft

Live Load $L = \{ExI / (675 \times w_{LL})\}^{1/3}$ L = 16.78 ft

2x10 Hem Fir No 2 @ 24" oc L_(MAX) = 16.33 ft

Project: **FOO RESIDENCE**

Date: _____

Client: _____ Page Number: _____

Ceiling Joist Maximum Span

Dead Load = 5 psf
 Light Storage = 10 psf
 Partition = 0 psf (added to live load)
 Spacing = 24 in

| Adjustment Factors | |
|--------------------|------|
| Size | 1.00 |
| Repetitive | 1.15 |
| Flat Use | 1.00 |
| Duration | 1.00 |

Member Size **2x12** b = 1.5 in.
 d = 11.25 in.

Section Properties A = 16.875 in²
 S = 31.641 in³
 I = 177.98 in⁴

Species Hem Fir No. 2

F_b = 850 psi (base) = 850 psi
 F_v = 150 psi
 E = 1300 ksi

Load w_{DL} = 10 plf
 w_{LL} = 20 plf
 w_{TL} = 30 plf

Analysis

Bending $L = \{F_b \times S \times 8 / (12 \times w_{TL})\}^{0.5}$ L = 24.45 ft

Shear $L = \{(F_v \times A / 1.5 \times w_{TL}) + (d / 12)\} \times 2$ L = 114.38 ft

Deflection

Total Load $L = \{E \times I / (450 \times w_{TL})\}^{1/3}$ L = 25.70 ft

Live Load $L = \{E \times I / (675 \times w_{LL})\}^{1/3}$ L = 25.70 ft

2x12 Hem Fir No 2 @ 24" oc L_(MAX) = 24.0 ft

Project: **FOO RESIDENCE** Date: _____

Client: _____ Page Number: _____

Ceiling Joist Maximum Span

Dead Load = 5 psf
 Light Storage = 20 psf
 Partition = 0 psf (added to live load)
 Spacing = 24 in

| Adjustment Factors | |
|--------------------|------|
| Size | 1.00 |
| Repetitive | 1.15 |
| Flat Use | 1.00 |
| Duration | 1.00 |

Member Size **2x12** b = 1.5 in.
 d = 11.25 in.

Section Properties A = 16.875 in²
 S = 31.641 in³
 I = 177.98 in⁴

Species Hem Fir No. 2

F_b = 850 psi (base) = 850 psi
 F_v = 150 psi
 E = 1300 ksi

Load w_{DL} = 10 plf
 w_{LL} = 40 plf
 w_{TL} = 50 plf

Analysis

Bending $L = \{F_b \times S \times 8 / (12 \times w_{TL})\}^{0.5}$ L = 18.94 ft

Shear $L = \{(F_v \times A / 1.5 \times w_{TL}) + (d/12)\} \times 2$ L = 69.38 ft

Deflection

Total Load $L = \{E \times I / (450 \times w_{TL})\}^{1/3}$ L = 21.68 ft

Live Load $L = \{E \times I / (675 \times w_{LL})\}^{1/3}$ L = 20.40 ft

2x12 Hem Fir No 2 @ 24" oc L_(MAX) = 19.0 ft

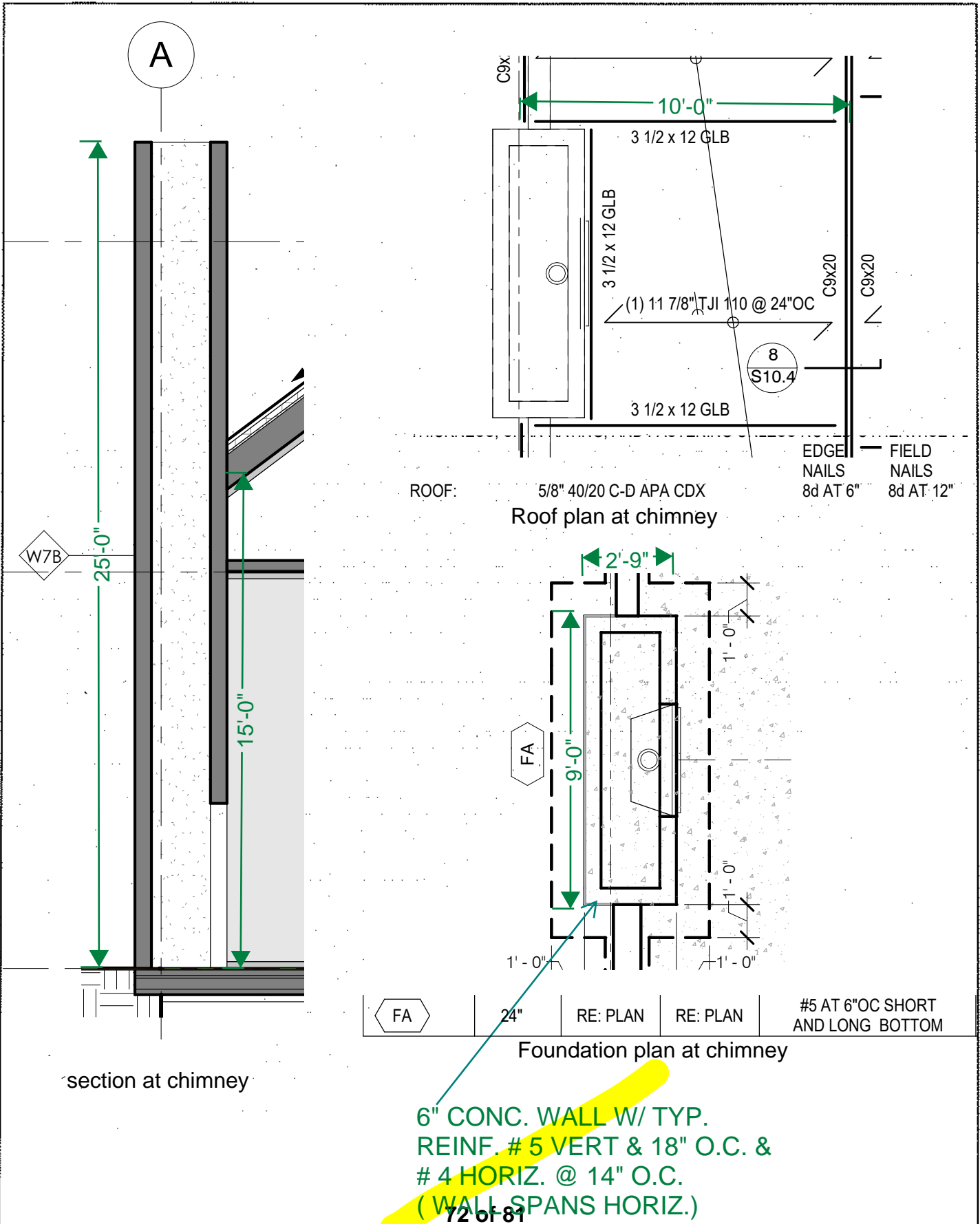
CHIMNEY CALCULATIONS

Project: Foo residence chimney

Date: _____

Client: _____

Page Number: _____



Project: Foo residence chimney

Date: _____

Client: _____

Page Number: _____

CHIMNEY WT = $150 (.5)(25)(23.5) = 44 \text{ K}$

SEISMIC = $0.309(44\text{K}) = 13.6 \text{ K ASD}$

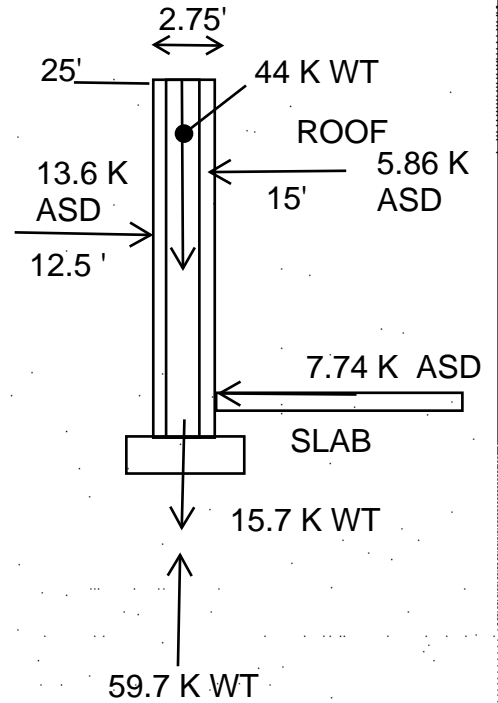
WIND = $23.69 (9)(25) = 5.33 \text{ ASD}$

CHIMNEY FOOTING WT = $150 (2)(4.75)(11) = 15.7 \text{ K}$

(2) TIES AT ROOF TO GLB $5.86 \text{ K}/2 = 2.93\text{K ASD}$
 < (3) 5/8" DIAM. TITEN HD (EMBED 4") = 5.76 K

LOAD INTO ROOF DIAPHRAGM = $2.93\text{K}/ 10 \text{ FT} = 293 \text{ LB/FT}$ < 360 LB/FT (BLOCK DIAPHRAGM)

SOIL BEARING = $59.7 \text{ K}/ (4.75*11) = 1143 \text{ PSF}$ < 3000 PSF



Project: **FOO RESIDENCE**

Date:

Client:

Page Number:

Seismic Load On Concrete Chimney Walls

Wall Components 6" conc.

Parapet Components 6" conc.

| | | | | | | |
|------|--------|-----------------------|------|--------|-----------------------|----------------|
| SDS= | 0.920 | | SDS= | 0.920 | | note: Z/H <= 1 |
| Rp= | 2.500 | | Rp= | 2.500 | | |
| Ap= | 1.000 | | Ap= | 1.000 | | |
| Ip= | 1.000 | | Ip= | 1.000 | | |
| Z = | 1.000 | Ht at attachment (FT) | Z = | 1.000 | Ht at attachment (FT) | |
| H= | 1.000 | Avg. roof ht (FT) | H= | 1.000 | Avg. roof ht (FT) | |
| Wp= | 75.000 | Component Wt (PSF) | Wp= | 75.000 | Component Wt (PSF) | |

Walls

| | | | LRFD | | ASD | |
|------------|-----|------------|-------------|--|------------|-----------|
| Design | Fp= | 0.442 *WP= | 33.12 PSF | | 0.309 *WP= | 19.87 PSF |
| Max. | Fp= | 1.472 *WP= | 110.40 PSF | | 1.030 *WP= | 66.24 PSF |
| Min. | Fp= | 0.276 *WP= | 20.70 PSF | | 0.193 *WP= | 12.42 PSF |
| USE | Fp= | 0.442 *WP= | 33.12 PSF | | 0.309 *WP= | 19.87 PSF |

Parapets

| | | | LRFD | | ASD | |
|------------|-----|------------|-------------|--|------------|-----------|
| Design | Fp= | 0.442 *WP= | 33.12 PSF | | 0.309 *WP= | 19.87 PSF |
| Max. | Fp= | 1.472 *WP= | 110.40 PSF | | 1.030 *WP= | 66.24 PSF |
| Min. | Fp= | 0.276 *WP= | 20.70 PSF | | 0.193 *WP= | 12.42 PSF |
| USE | Fp= | 0.442 *WP= | 33.12 PSF | | 0.309 *WP= | 19.87 PSF |

Project: Foo Chimnev
 Client: _____

Date: _____
 Page Number: _____

WIND LOAD ON COMPONENTS AND CLADDING

ASCE 7-10, Chapter 30, Part 2:Low-Rise Buildings (Simplified)

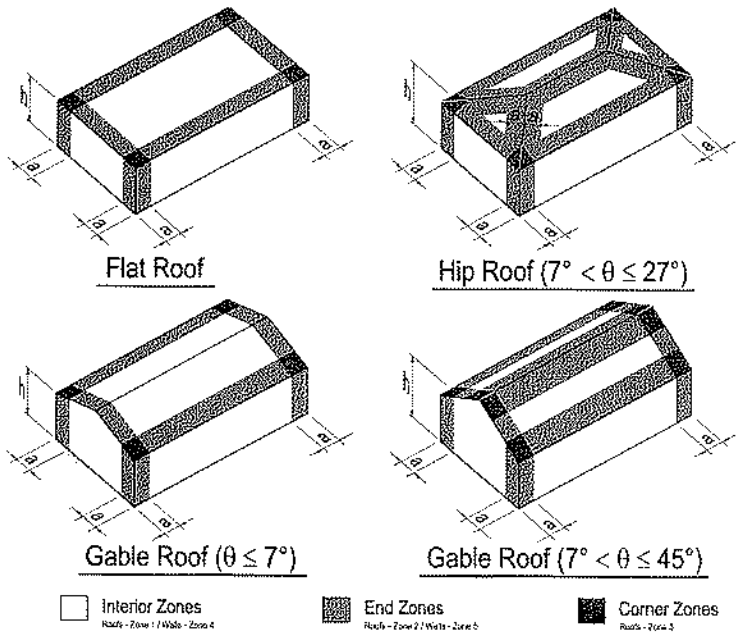
Wind Criteria:

- Risk Category, I, II, III or IV (Table 1.5-1) = **II**
- Basic Wind Speed **110** MPH (Figure 26.5-1A, B or C)
- Exposure **C**
- Average roof height **15.00** ft.
- Roof angle **0° to 7°**
- $K_{zt} =$ **1.60** (Sec. 26.8 and Figure 26.8-1 of ASCE 7-10)
- $\lambda =$ **1.21**

Note: The minimum design wind pressure is 16 PSF per Sec 30.2.2 .

$P_{net} = \lambda * K_{zt} * P_{net30}$

| Zone | Area | P_{net30} (PSF) | | P_{net} (PSF) |
|------|------|-------------------|--------|-----------------|
| 1 | 10 | 8.90 | -21.80 | 42.20 |
| 1 | 20 | 8.30 | -21.20 | 41.04 |
| 1 | 50 | 7.60 | -20.50 | 39.69 |
| 1 | 100 | 7.00 | -19.90 | 38.53 |
| 2 | 10 | 8.90 | -36.50 | 70.66 |
| 2 | 20 | 8.30 | -32.60 | 63.11 |
| 2 | 50 | 7.60 | -27.50 | 53.24 |
| 2 | 100 | 7.00 | -23.60 | 45.69 |
| 3 | 10 | 8.90 | -55.00 | 106.48 |
| 3 | 20 | 8.30 | -45.50 | 88.09 |
| 3 | 50 | 7.60 | -33.10 | 64.08 |
| 3 | 100 | 7.00 | -23.60 | 45.69 |
| 4 | 10 | 21.80 | -23.60 | 45.69 |
| 4 | 20 | 20.80 | -22.60 | 43.75 |
| 4 | 50 | 19.50 | -21.30 | 41.24 |
| 4 | 100 | 18.50 | -20.40 | 39.49 |
| 4 | 500 | 16.20 | -18.10 | 35.04 |
| 5 | 10 | 21.80 | -29.10 | 56.34 |
| 5 | 20 | 20.80 | -27.20 | 52.66 |
| 5 | 50 | 19.50 | -24.60 | 47.63 |
| 5 | 100 | 18.50 | -22.60 | 43.75 |
| 5 | 500 | 16.20 | -18.10 | 35.04 |



ASD = 0.6X 39.49 = 23.69 PSF

Interior Zones Roofs - Zone 1 / Walls - Zone 4
 End Zones Roofs - Zone 2 / Walls - Zone 5
 Corner Zones Roofs - Zone 3

JOIST

- D. L. to resist uplift _____ PSF
- Spacing _____ Ft.
- Span _____ Ft.
- Tributary Area **0.00** Sq. Ft.
- Wind Uplift _____ PSF
- Net Uplift **0.00** PSF (ASD)
- Specify → _____ PSF

beams

- D. L. to resist uplift _____ PSF
- Spacing (Trib. Width) _____ Ft.
- Span _____ Ft.
- Tributary Area **0.00** Sq. Ft.
- Wind Uplift _____ PSF
- Net Uplift **0.00** PSF (ASD)
- Specify → _____ PSF

Concrete Beam

Lic. # : KW-06002997

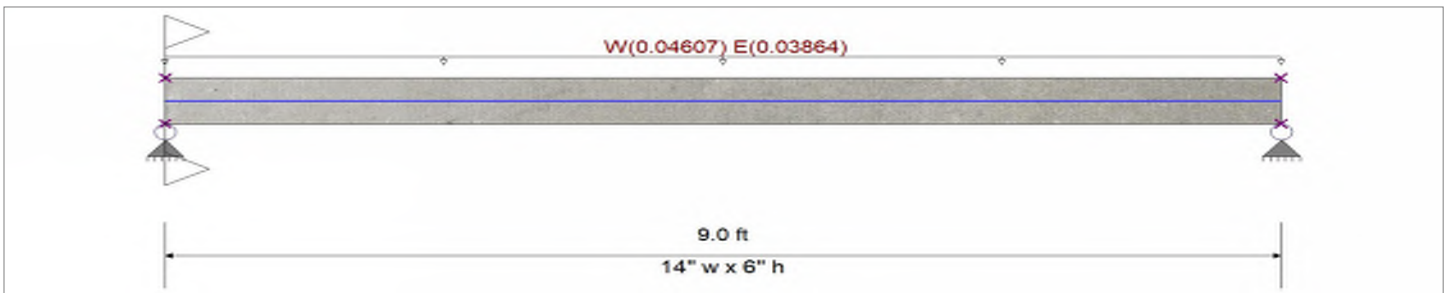
DESCRIPTION: chimney 6" conc. Walls span horiz.

CODE REFERENCES

Calculations per ACI 318-11, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

| | | | | | |
|---------------------------|---|--------------|--|-----------|--------------|
| f'_c | = | 2.50 ksi | ϕ Phi Values | Flexure : | 0.90 |
| $f_r = f'_c^{1/2} * 7.50$ | = | 375.0 psi | | Shear : | 0.750 |
| Ψ Density | = | 145.0 pcf | β_1 | = | 0.850 |
| λ LtWt Factor | = | 1.0 | | | |
| Elastic Modulus | = | 3,122.0 ksi | Fy - Stirrups | = | 40.0 ksi |
| fy - Main Rebar | = | 60.0 ksi | E - Stirrups | = | 29,000.0 ksi |
| E - Main Rebar | = | 29,000.0 ksi | Stirrup Bar Size # | = | 3 |
| | | | Number of Resisting Legs Per Stirrup = | = | 2 |



Cross Section & Reinforcing Details

Rectangular Section, Width = 14.0 in, Height = 6.0 in
Span #1 Reinforcing....
1-#4 at 3.0 in from Bottom, from 0.0 to 9.0 ft in this span

Load for Span Number 1
Uniform Load : W = 0.04607, E = 0.03864 k/ft, Tributary Width = 1.0 ft, (lateral per 14" width)

DESIGN SUMMARY

Design OK

| | | | |
|--------------------------------|------------------------|-----------------------------------|-------------------------------|
| Maximum Bending Stress Ratio = | 0.185 : 1 | Maximum Deflection | |
| Section used for this span | Typical Section | Max Downward Transient Deflection | 0.009 in Ratio = 12497 >=360. |
| Mu : Applied | 0.4665 k-ft | Max Upward Transient Deflection | 0.000 in Ratio = 0 <360.0 |
| Mn * Phi : Allowable | 2.518 k-ft | Max Downward Total Deflection | 0.009 in Ratio = 12497 >=180. |
| Location of maximum on span | 4.508 ft | Max Upward Total Deflection | 0.000 in Ratio = 0 <180.0 |
| Span # where maximum occurs | Span # 1 | | |

Vertical Reactions

Support notation : Far left is #1

| Load Combination | Support 1 | Support 2 |
|----------------------------|-----------|-----------|
| Overall MAXimum | 0.207 | 0.207 |
| Overall MINimum | 0.091 | 0.091 |
| +D+0.60W+H | 0.124 | 0.124 |
| +D+0.70E+H | 0.122 | 0.122 |
| +D+0.750Lr+0.750L+0.450W+H | 0.093 | 0.093 |
| +D+0.750L+0.750S+0.450W+H | 0.093 | 0.093 |
| +D+0.750L+0.750S+0.5250E+H | 0.091 | 0.091 |
| +0.60D+0.60W+0.60H | 0.124 | 0.124 |
| +0.60D+0.70E+0.60H | 0.122 | 0.122 |
| W Only | 0.207 | 0.207 |
| E Only | 0.174 | 0.174 |
| H Only | | |

CT ENGINEERING
 180 NICKERSON ST.
 SUITE 302
 SEATTLE, WA 98109

Project Title:
 Engineer:
 Project ID:
 Project Descr:

(206) 285-4512

Printed: 9 MAR 2021, 1:18PM

Concrete Beam

File: 20035 foo chimney .ec6
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Lic. # : KW-06002997

C.T. ENGINEERING

DESCRIPTION: chimney 6" conc. Walls span horiz.

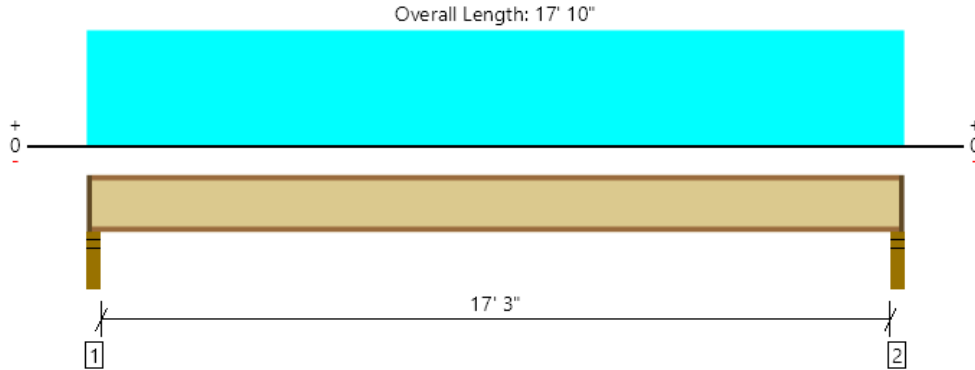
| Load Combination Segment | Span # | Location (ft) along Beam | Bending Stress Results (k-ft) | | |
|-----------------------------|--------|-----------------------------|-------------------------------|---------|--------------|
| | | | Mu : Max | Phi*Mnx | Stress Ratio |
| MAXimum BENDING Envelope | | | | | |
| Span # 1 | 1 | 9.000 | 0.47 | 2.52 | 0.19 |
| +0.90D+W+0.90H | | | | | |
| Span # 1 | 1 | 9.000 | 0.47 | 2.52 | 0.19 |
| +0.90D+E+0.90H | | | | | |
| Span # 1 | 1 | 9.000 | 0.39 | 2.52 | 0.16 |

Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl (in) | Location in Span (ft) | Load Combination | Max. "+" Defl (in) | Location in Span (ft) |
|------------------|------|--------------------|-----------------------|------------------|--------------------|-----------------------|
| W Only | 1 | 0.0086 | 4.500 | | 0.0000 | 0.000 |

MISCELLANEOUS CALCULATIONS

Level 2, Floor: Joist @ BDRM 2
 2 piece(s) 11 7/8" TJI @ 110 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

| Design Results | Actual @ Location | Allowed | Result | LDF | Load: Combination (Pattern) |
|-----------------------|-------------------|--------------|-----------------|------|-----------------------------|
| Member Reaction (lbs) | 870 @ 2 1/2" | 2083 (2.25") | Passed (42%) | 1.00 | 1.0 D + 1.0 L (All Spans) |
| Shear (lbs) | 851 @ 3 1/2" | 3120 | Passed (27%) | 1.00 | 1.0 D + 1.0 L (All Spans) |
| Moment (Ft-lbs) | 3741 @ 8' 11" | 6320 | Passed (59%) | 1.00 | 1.0 D + 1.0 L (All Spans) |
| Live Load Defl. (in) | 0.197 @ 8' 11" | 0.435 | Passed (L/999+) | -- | 1.0 D + 1.0 L (All Spans) |
| Total Load Defl. (in) | 0.364 @ 8' 11" | 0.871 | Passed (L/574) | -- | 1.0 D + 1.0 L (All Spans) |
| TJ-Pro™ Rating | 51 | 40 | Passed | -- | -- |

System : Floor
 Member Type : Joist
 Building Use : Residential
 Building Code : IBC 2015
 Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

| Supports | Bearing Length | | | Loads to Supports (lbs) | | | Accessories |
|--------------------|----------------|-----------|----------|-------------------------|------------|-------|------------------|
| | Total | Available | Required | Dead | Floor Live | Total | |
| 1 - Stud wall - HF | 3.50" | 2.25" | 1.75" | 404 | 476 | 880 | 1 1/4" Rim Board |
| 2 - Stud wall - HF | 3.50" | 2.25" | 1.75" | 404 | 476 | 880 | 1 1/4" Rim Board |

• Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

| Lateral Bracing | Bracing Intervals | Comments |
|------------------|-------------------|----------|
| Top Edge (Lu) | 4' 1" o/c | |
| Bottom Edge (Lu) | 17' 8" o/c | |

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

| Vertical Load | Location (Side) | Spacing | Dead (0.90) | Floor Live (1.00) | Comments |
|-------------------|-----------------|---------|-------------|-------------------|--------------|
| 1 - Uniform (PSF) | 0 to 17' 10" | 16" | 34.0 | 40.0 | Default Load |

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

| ForteWEB Software Operator | Job Notes |
|--|-----------|
| mark nagamatsu ct engineering inc (206) 285-4512 mnagamatsu@ctengineering.com | |



Project: FOO RESIDENCE

Date: _____

180 Nickerson St.
Suite 302
Seattle, WA
98109

(206) 285-4512

Client: _____

Page Number: _____

FAX:

(206) 285-0618

TORSION CAPACITY HSS 3 1/2 x 3 1/2 x 3/8

$$b/t = 3.5 / .375 = 9.33$$

$$b/t \leq 2.45 \sqrt{E/F_y} = 61.5$$

$$F_{cr} = 0.6 F_y = 0.6 (46) = 27.6 \text{ ksi}$$

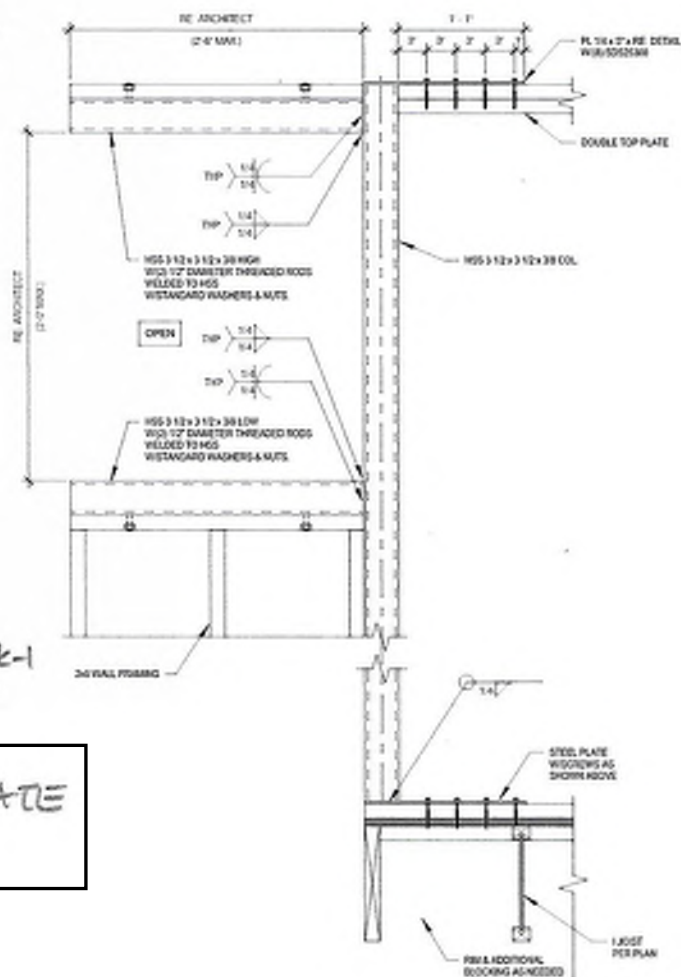
$$T_n = F_{cr} C = 27.6 \text{ ksi} (6.77) \\ = 186 \text{ k-in}$$

$$T_n / \Omega = 186 / 1.67 = 112 \text{ k-in} \\ = 9.32 \text{ k-ft}$$

$$w_e = 45 \text{ psf} \times 0.6 = 27 \text{ psf (ASD)}$$

$$M = 27 (2) (12) (1) = 648 \text{ ft-lb} < 9.32 \text{ k-ft}$$

HSS 3 1/2 x 3 1/2 x 3/8 ADEQUATE
FOR TORSION.



Project: FOO RESIDENCE Date: _____
Client: _____ Page Number: _____

WIND LOAD ON COMPONENTS AND CLADDING

Based on ASCE 7-10, Chapter 30, Part 2: Low-Rise Buildings (Simplified)

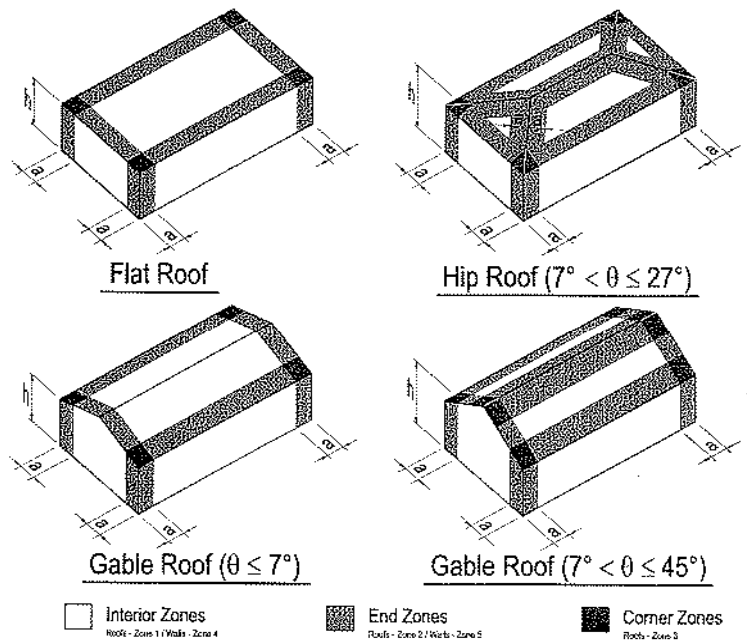
Wind Criteria:

- Risk Category, I, II, III or IV (Table 1.5-1 of ASCE 7-10) = **II**
- Basic Wind Speed **110** MPH (See Figure 26.5-1A, B or C of ASCE 7-10)
- Exposure **C**
- Average roof height **18.00** ft.
- Roof angle **27° to 45°**
- K_{zt} = **1.60** (See Section 26.8 and Figure 26.8-1 of ASCE 7-10)
- λ = **1.29**

Note: The minimum design wind pressure is 16 PSF per Section 30.2.2 of ASCE 7-10.

$P_{net} = \lambda * K_{zt} * P_{net30}$

| Zone | Area | P_{net30} (PSF) | | P_{net} (PSF) |
|------|------|-------------------|--------|-----------------|
| 1 | 10 | 19.90 | -21.80 | 45.00 |
| 1 | 20 | 19.40 | -20.70 | 42.72 |
| 1 | 50 | 18.60 | -19.20 | 39.63 |
| 1 | 100 | 18.10 | -18.10 | 37.36 |
| 2 | 10 | 19.90 | -25.50 | 52.63 |
| 2 | 20 | 19.40 | -24.30 | 50.16 |
| 2 | 50 | 18.60 | -22.90 | 47.27 |
| 2 | 100 | 18.10 | -21.80 | 45.00 |
| 3 | 10 | 19.90 | -25.50 | 52.63 |
| 3 | 20 | 19.40 | -24.30 | 50.16 |
| 3 | 50 | 18.60 | -22.90 | 47.27 |
| 3 | 100 | 18.10 | -21.80 | 45.00 |
| 4 | 10 | 21.80 | -23.60 | 48.71 |
| 4 | 20 | 20.80 | -22.60 | 46.65 |
| 4 | 50 | 19.50 | -21.30 | 43.96 |
| 4 | 100 | 18.50 | -20.40 | 42.11 |
| 4 | 500 | 16.20 | -18.10 | 37.36 |
| 5 | 10 | 21.80 | -29.10 | 60.06 |
| 5 | 20 | 20.80 | -27.20 | 56.14 |
| 5 | 50 | 19.50 | -24.60 | 50.77 |
| 5 | 100 | 18.50 | -22.60 | 46.65 |
| 5 | 500 | 16.20 | -18.10 | 37.36 |



Interior Zones Roofs - Zone 1 / Walls - Zone 4
End Zones Roofs - Zone 2 / Walls - Zone 5
Corner Zones Roofs - Zone 3

I Joist

- D. L. to resist uplift **12.00** PSF Uplift Force = 17psf(2)(20/2) = 340 lbs.
- Spacing **2.00** Ft. Simpson H1Tie Capacity = 425 lbs (Hem Fir)
- Span **10.00** Ft.
- Tributary Area **20.00** Sq. Ft.
- Wind Uplift ($p_{net} \times 0.6$) **28.36** PSF
- Net Uplift **16.36** PSF
- Specify **17.00** PSF