



## Structural Calculations (Renovations & Additions to Existing Structure)

Project: **Ford Residence**  
6805 SE 32<sup>nd</sup> St.  
Mercer Island, WA 98040

For: **Sturman Architects**  
9 – 103<sup>rd</sup> Avenue NE, Suite 203  
Bellevue, WA 98004

By: **Année Structural Engineering, LLC**  
1801 18<sup>th</sup> Ave S  
Seattle, WA 98144

Date: **November 21, 2025**



# Design Criteria



Project Name: **Ford Residence - Upper, Eastern portion**  
 Location: **6805 SE 32nd St, Mercer Island, WA**

Date: **10/10/2025**  
 Soil Bearing: **2000** psf  
 Frost Depth: **12"**

<b>Dead Loads:</b>	<u>Roof:</u>		<u>Floors:</u>		<u>Walls:</u>	
	Comp. Roofing	5.1 PSF	Flooring	3.0 PSF	Siding	2.3 PSF
	1/2" Sheathing	1.7 PSF		0.0 PSF	Plywood	1.7 PSF
	Rafters	2.2 PSF	3/4" Sheathing	2.5 PSF	2x Studs	1.8 PSF
	Insulation	0.9 PSF	Joists	2.2 PSF	Insulation	0.5 PSF
	5/8" Gypsum	2.8 PSF	5/8" Gypsum	2.8 PSF	1/2" Gypsum	2.2 PSF
	Miscellaneous	2.3 PSF	Miscellaneous	1.5 PSF	Miscellaneous	1.5 PSF
	<b>Total</b>	<b>15.0 PSF</b>	<b>Total</b>	<b>12.0 PSF</b>	<b>Total</b>	<b>10.0 PSF</b>
<b>Live Loads:</b>	Snow	<b>25.0 PSF</b>	Floor	<b>40.0 PSF</b>	Wind	<b>17.8 PSF</b>

**Seismic Loads:** *per 2021 IBC, Sect. 1613 & ASCE 7-16, Chapter 11*

Design Category = **D**                      Importance = **1.0**                      Redundancy = **1.00**  
 Site Class = **D**                              R = **6.5**  
 Latitude ( $^{\circ}$ N) = **47.582**                      (*per USGS*)                       $S_s = 1.410$                        $F_a = 1.200$                        $S_{DS} = 2/3(F_a \times S_s) = 1.128$   
 Longitude ( $^{\circ}$ W) = **122.274**                      (*per USGS*)                       $S_1 = 0.491$                        $F_v = 1.809$                        $S_{D1} = 2/3(F_v \times S_1) = 0.592$

Building                       $C_t = 0.02$  (wood)  
 Height                           $h_n = 22.1$  ft.  
 Period                           $T = C_t(h_n)^{3/4} = 0.20$  sec.                       $T_0 = 0.2 * (S_{D1}/S_{DS}) = 0.10$                        $T_s = (S_{D1}/S_{DS}) = 0.52$

$S_a = 1.128$                        $S_a = S_{DS}$  if  $T_0 < T < T_{sr}$ ,  $S_a = 0.6 * (S_{DS}/T_0) * T + 0.4 * S_{DS}$  if  $T < T_0$ ,  $S_a = S_{D1}/T$  if  $T > T_s$

Not greater than:                       $C_s = S_{D1}/T * (R/I) = 0.447$   
 Not less than:                           $C_s = 0.044 S_{DS} * I = 0.050$   
 Design Category E or F; not less than:                       $C_s = 0.5 S_1 / (R/I) = 0.038$   
 Seismic Design Coefficient:                       $C_s = S_{DS} / (R/I) = 0.174$

**$C_s = 0.174$**

**Seismic Weight Distribution:**

Diaphragm	$h_i$ (ft.)	$w_i$ (kips)	$h_i w_i$ (K-ft.)	$w_i h_i / \sum(w_i h_i)$	<b>F<sub>i</sub> (lbs.)</b>	Sum F <sub>i</sub> (lbs.)
			0	0	<b>0</b>	0
			0	0	<b>0</b>	0
Upper Roof	17.3	19.75	341.7	0.6479	<b>3,144</b>	3,144
Upper Floor	9.2	20.19	185.7	0.3521	<b>1,709</b>	4,852
			0	0	<b>0</b>	4,852
		39.944	527.5			

Design Base Shear (ASD) =  $0.7 * (0.174 * W) = 0.121 * W =$  **4,852 lbs.**

**Wind Loads:**

*per ASCE 7-16, Section 27.5*

*Section 30.4*

Cladding (ft2): 100                      20

Wind Speed (MPH)	110	Zone	(ASD)	Adj.	Zone	Pn30	P	Pn30	P		
Exposure	C	Wall - Ph	26.7	<b>16.0</b>	(p.352)	Wall	4	-20.4	-16.1	-22.6	<b>-17.8</b>
Roof Pitch (x:12)	6	Wall - Po	26.0	<b>15.6</b>			5	-22.6	-17.8	-27.2	<b>-21.4</b>
$K_1 =$	0.00	Roof - 1	10.5	<b>6.3</b>		Roof	1	-22.8	<b>-18.0</b>	-31.0	-24.4
$K_2 =$	0.00	Roof - 2	-8.0	<b>-4.8</b>			2e	-22.8	<b>-18.0</b>	-31.0	-24.4
$K_3 =$	0.00	Roof - 3	-25.9	<b>-15.5</b>			2n	-29.0	<b>-22.8</b>	-43.3	-34.1
$K_t = (1 + K_1 * K_2 * K_3)^2 =$	1.00	Roof - 4	-22.8	<b>-13.7</b>			2r	-29.0	<b>-22.8</b>	-43.3	-34.1
(p.362) $\lambda =$	1.31	Roof - 5	-18.8	<b>-11.3</b>			3e	-29.0	<b>-22.8</b>	-43.3	-34.1
(p.291) Exp. Fctr =	1.00						3r	-36.5	<b>-28.7</b>	-52.0	-41.0

# Design Criteria



Project Name: **Ford Residence - Lower, Western portion**  
 Location: **6805 SE 32nd St, Mercer Island, WA**

Date: **10/10/2025**  
 Soil Bearing: **2000** psf  
 Frost Depth: **12"**

<b>Dead Loads:</b>	<u>Roof:</u>		<u>Floors:</u>		<u>Walls:</u>	
	Comp. Roofing	5.1 PSF	Flooring	3.0 PSF	Siding	2.3 PSF
	1/2" Sheathing	1.7 PSF		0.0 PSF	Plywood	1.7 PSF
	Rafters	2.2 PSF	3/4" Sheathing	2.5 PSF	2x Studs	1.8 PSF
	Insulation	0.9 PSF	Joists	2.2 PSF	Insulation	0.5 PSF
	5/8" Gypsum	2.8 PSF	5/8" Gypsum	2.8 PSF	1/2" Gypsum	2.2 PSF
	Miscellaneous	2.3 PSF	Miscellaneous	1.5 PSF	Miscellaneous	1.5 PSF
	<b>Total</b>	<b>15.0 PSF</b>	<b>Total</b>	<b>12.0 PSF</b>	<b>Total</b>	<b>10.0 PSF</b>
<b>Live Loads:</b>	Snow	<b>25.0 PSF</b>	Floor	<b>40.0 PSF</b>	Wind	<b>17.8 PSF</b>

**Seismic Loads:** *per 2021 IBC, Sect. 1613 & ASCE 7-16, Chapter 11*

Design Category = **D**                      Importance = **1.0**                      Redundancy = **1.00**  
 Site Class = **D**                              R = **6.5**  
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 Longitude ( $^{\circ}$ W) = **122.274**                      (*per USGS*)                       $S_1 = 0.491$                        $F_v = 1.809$                        $S_{D1} = 2/3(F_v \times S_1) = 0.592$

Building                       $C_t = 0.02$  (wood)  
 Height                       $h_n = 22.1$  ft.  
 Period                       $T = C_t(h_n)^{3/4} = 0.20$  sec.                       $T_0 = 0.2 * (S_{D1} / S_{DS}) = 0.10$                        $T_s = (S_{D1} / S_{DS}) = 0.52$

$S_a = 1.128$                        $S_a = S_{DS}$  if  $T_0 < T < T_{sr}$ ,  $S_a = 0.6 * (S_{DS} / T_0) * T + 0.4 * S_{DS}$  if  $T < T_0$ ,  $S_a = S_{D1} / T$  if  $T > T_s$

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**$C_s = 0.174$**

**Seismic Weight Distribution:**

Diaphragm	$h_i$ (ft.)	$w_i$ (kips)	$h_i w_i$ (K-ft.)	$w_i h_i / \sum(w_i h_i)$	<b>F<sub>i</sub> (lbs.)</b>	Sum F <sub>i</sub> (lbs.)
			0	0	<b>0</b>	0
			0	0	<b>0</b>	0
			0	0	<b>0</b>	0
Lower Roof	17.2	29.27	504	0.6659	<b>4,905</b>	4,905
Main Floor	8.1	31.37	252.9	0.3341	<b>2,461</b>	7,366
		60.641	756.9			

Design Base Shear (ASD) =  $0.7 * (0.174 * W) = 0.121 * W =$  **7,366 lbs.**

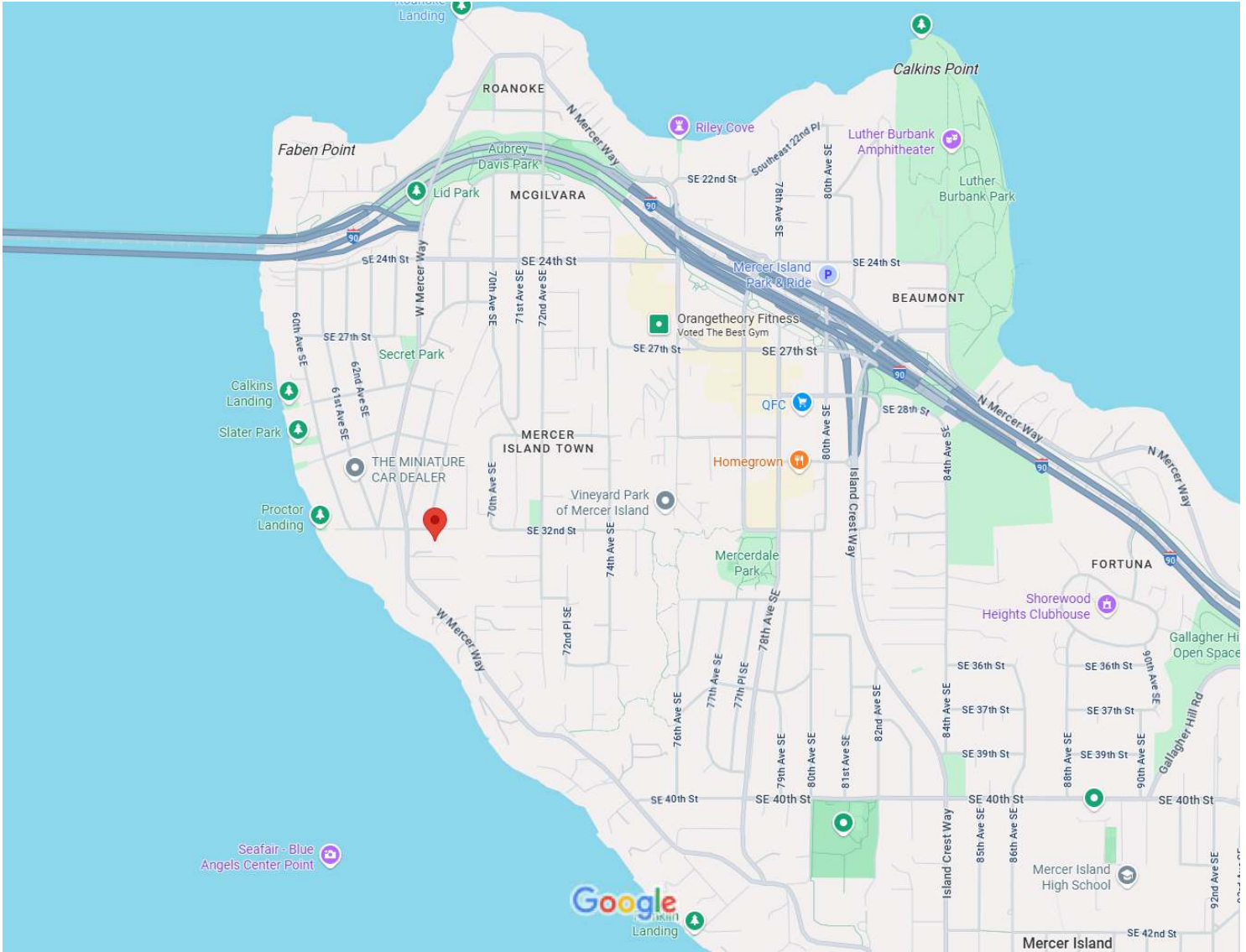
**Wind Loads:**

*per ASCE 7-16, Section 27.5*

*Section 30.4*

Cladding (ft2): 100                      20

Wind Speed (MPH)	110	Zone	(ASD)	Adj.	Zone	Pn30	P	Pn30	P		
Exposure	C	Wall - Ph	26.7	<b>16.0</b>	(p.352)	Wall	4	-20.4	-16.1	-22.6	<b>-17.8</b>
Roof Pitch (x:12)	6	Wall - Po	26.0	<b>15.6</b>			5	-22.6	-17.8	-27.2	<b>-21.4</b>
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$K_2 =$	0.00	Roof - 2	-8.0	<b>-4.8</b>			2e	-22.8	<b>-18.0</b>	-31.0	-24.4
$K_3 =$	0.00	Roof - 3	-25.9	<b>-15.5</b>			2n	-29.0	<b>-22.8</b>	-43.3	-34.1
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(p.291) Exp. Fctr =	1.00						3r	-36.5	<b>-28.7</b>	-52.0	-41.0



Map data ©2025 Google 1000 ft

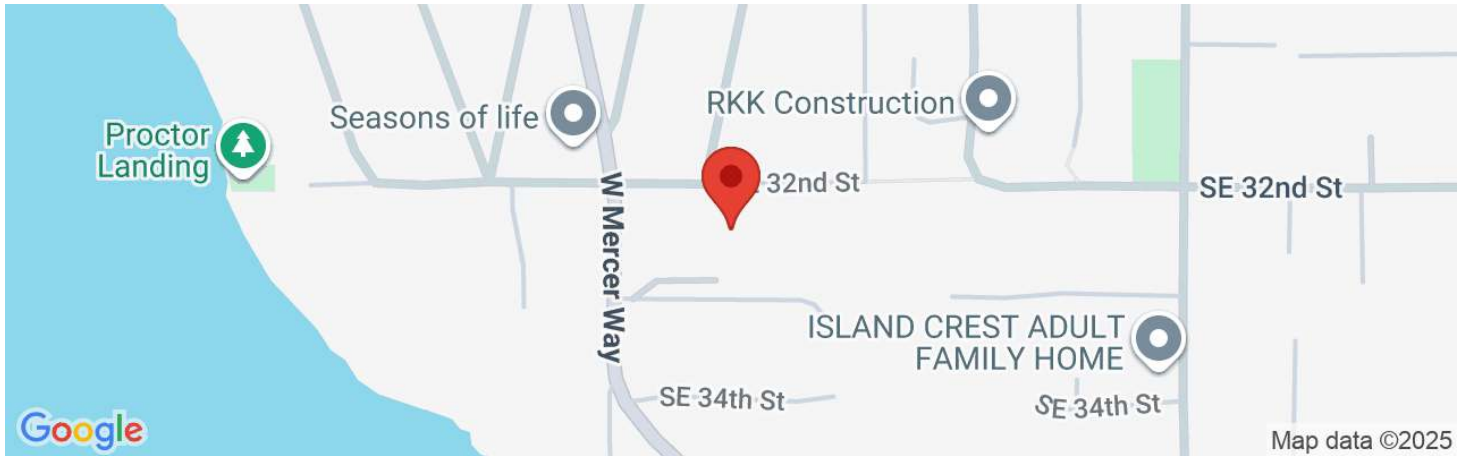
110 MPH, Exposure C, Kt = 1.00



# Ford Residence

6805 SE 32nd St, Mercer Island, WA 98040, USA

Latitude, Longitude: 47.5815663, -122.2474657



<b>Date</b>	9/19/2025, 1:19:45 PM
<b>Design Code Reference Document</b>	ASCE7-16
<b>Risk Category</b>	II
<b>Site Class</b>	D - Default (See Section 11.4.3)

Type	Value	Description
$S_S$	1.41	$MCE_R$ ground motion. (for 0.2 second period)
$S_1$	0.491	$MCE_R$ ground motion. (for 1.0s period)
$S_{MS}$	1.692	Site-modified spectral acceleration value
$S_{M1}$	null -See Section 11.4.8	Site-modified spectral acceleration value
$S_{DS}$	1.128	Numeric seismic design value at 0.2 second SA
$S_{D1}$	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
$F_a$	1.2	Site amplification factor at 0.2 second
$F_v$	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.603	$MCE_G$ peak ground acceleration
$F_{PGA}$	1.2	Site amplification factor at PGA
$PGA_M$	0.724	Site modified peak ground acceleration
$T_L$	6	Long-period transition period in seconds
$S_{sRT}$	1.41	Probabilistic risk-targeted ground motion. (0.2 second)
$S_{sUH}$	1.563	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
$S_{sD}$	3.353	Factored deterministic acceleration value. (0.2 second)
$S_{1RT}$	0.491	Probabilistic risk-targeted ground motion. (1.0 second)
$S_{1UH}$	0.547	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
$S_{1D}$	1.357	Factored deterministic acceleration value. (1.0 second)

### LATERAL ANALYSIS - SEISMIC WEIGHT:

AT WESTERN PORTION OF STRUCTURE (LOWER):

$$\text{AT LOWER ROOF, } W_{LR} = 1,661 \text{ k}^2 (15' \times 112')$$

$$+ 10' \times 112' \left( \frac{8.8'}{2} \times 99' \right) = 29,271^{\text{k}}$$

$$\text{AT MAIN FLOOR; } W_{MF} = 1,257 \text{ k}^2 (12' \times 104') + 653 \text{ k}^2 (10' \times 62')$$

$$+ 10' \times 104' \left( \frac{8.8'}{2} \times 99' + \frac{7.5'}{2} \times 144' \right) = 31,370^{\text{k}}$$

$$\text{DESIGN BASE SHEAR; } V_{\text{LOWER}} = \frac{49,005^{\text{k}} + 2,461^{\text{k}}}{7.366^{\text{k}}} \text{ (ASD)}$$

AT EASTERN PORTION OF STRUCTURE (MAIN):

$$\text{AT UPPER ROOF; } W_{UR} = 1,001 \text{ k}^2 (15' \times 117')$$

$$+ 10' \times 117' \left( \frac{8.1'}{2} \times 117' \right) = 19,754^{\text{k}}$$

$$\text{AT UPPER FLOOR; } W_{UF} = 832 \text{ k}^2 (12' \times 103') + 832 \text{ k}^2 (15' \times 103')$$

$$+ 10' \times 103' \left( \frac{8.1'}{2} \times 117' + \frac{8.2'}{2} \times 153' \right) = 20,190^{\text{k}}$$

$$\text{DESIGN BASE SHEAR; } V_{\text{UPPER}} = \frac{3,144^{\text{k}} + 1,709^{\text{k}}}{4.852^{\text{k}}} \text{ (ASD)}$$

WILL CALCULATE WIND TRIBUTARY TO EACH INDIVIDUAL LATERAL LINE - - -

### LATERAL LOAD DISTRIBUTION:

TO UPPER FLOOR WALLS;

$$\text{LINE (D); } V_{4E} = 50\% (3,144^{\text{k}}) = 1,572^{\text{k}} \text{ (SEIS.)}$$

$$V_{4W} = 127 \text{ k}^2 (6.3 - (-4.8)) \times 10' + 67 \text{ k}^2 (16.0 \times 10')$$
$$= 2,482^{\text{k}} \text{ (WIND)}$$

$$V_4 = 2,482^{\text{k}} / 15.1' = 164^{\text{k}}/\text{ft} \rightarrow \underline{\text{SW6/SW(E)}}$$

$$\text{LINE (B); } V_{3E} = 50\% (3,144^{\text{k}}) = 1,572^{\text{k}} \text{ (SEIS.)}$$

$$V_{3W} = 86 \text{ k}^2 (6.3 - (-4.8)) + 92 \text{ k}^2 (16.0)$$
$$= 2,427^{\text{k}} \text{ (WIND)}$$

$$V_3 = 2,427^{\text{k}} / 25.0' = 97^{\text{k}}/\text{ft} \rightarrow \underline{\text{SW6/SW(E)}}$$

$$\text{LINE (C); } V_{CE} = 1,572^{\text{k}} \text{ (SEIS.)}$$

$$V_{CW} = 17 (6.3 - (-4.8)) + 130 (16.0) = 2,269^{\text{k}} \text{ (WIND)}$$

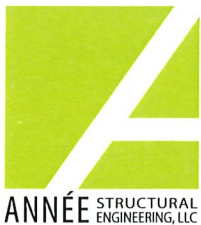
$$V_C = 2,269^{\text{k}} / 15.6' = 145^{\text{k}}/\text{ft} \rightarrow \underline{\text{SW6/SW(E)}}$$

$$\text{LINE (F)(G); } V_{FG} = 1,572^{\text{k}} \text{ (SEIS.)}$$

$$V_{FG} = 29 (6.3 - (-4.8)) + 112 (16.0) = 2,114^{\text{k}} \text{ (WIND)}$$

$$V_{FG} = 2,114^{\text{k}} / 4.5' = 470^{\text{k}}/\text{ft} \text{ (WIND)} \rightarrow \underline{\text{SW3}}$$

$$V_{FG} = 1,572^{\text{k}} / 4.5' = 349^{\text{k}}/\text{ft} \rightarrow \text{R.T.}$$



Project \_\_\_\_\_

Designer \_\_\_\_\_

Date \_\_\_\_\_

1801 18th Ave S, Seattle, WA 98144 206.658.5169

Sheet



TO MAIN FLOOR WALLS;

$$\text{LINE } \textcircled{1}-\textcircled{3}; V_{13} = 60\% (4,905^*) = 2,943^* \text{ (SEIS.)}$$

$$V_{13w} = 40(6.3(-4.8)) + 96(16.0) = 1,980^* \text{ (WIND)}$$

$$V_{13} = 2,943^* / 13.9' = 212^* / \text{ft.} \rightarrow \text{SW4}$$

DUE TO H:W RATIO

$$\text{LINE } \textcircled{4}; V_{4E} = 40\% (4,905^*) + 50\% (1,708^*) = 1,572^* \\ = 4,388^* \text{ (SEIS.)}$$

$$V_{4w} = 25(6.3(-4.8)) + 202(16.0) = 2,492^* \\ = 5,992^* \text{ (WIND)}$$

$$N_{4} = 5,992^* / 28.7' = 209^* / \text{ft.} \rightarrow \text{SW6}$$

LINES  $\textcircled{1}$ ,  $\textcircled{A}$ ,  $\textcircled{D}$ ,  $\textcircled{E}$  CONTROLLED BY WIND BY INSPECTION; AS THERE IS NO INCREASE IN SAIL AREA OR DECREASE IN LATERAL FORCE RESISTING SYSTEMS ALONG THESE LINES THERE ARE NO REQUIRED SEISMIC UPGRADES

OVERTURNING; FROM UPPER FLOOR WALLS;

$$\text{LINE } \textcircled{4}; T_{4} = 164^* / \text{ft.} (8.1') - \frac{12.9'}{2} (0.6 \times 81) = 1,015^* \\ \text{CS16}$$

$$\text{LINE } \textcircled{8}; T_{8} = 97^* / \text{ft.} (8.1') - \frac{20.6'}{2} (0.6 \times 81) = 293^* \\ \therefore \text{NO HD REQ'D}$$

LINE  $\textcircled{C}$ ; NO INCREASE IN LATERAL LOAD

$$\text{LINE } \textcircled{P}\textcircled{E}; T_{PE} = 470^* / \text{ft.} (8.1') - \frac{4.66'}{2} (0.6 \times 81) \\ = 3,694^* \rightarrow \text{MS1C48B3}$$

FROM MAIN PEN WALLS;

$$\text{LINE } \textcircled{1}, \textcircled{3}; T_{13} = 212^* / \text{ft.} (8.8') - \frac{8.1'}{2} (0.6 \times 88) \\ = 1,646^* \rightarrow \text{CS16}$$

$$\text{LINE } \textcircled{4}; T_{4} = 209^* / \text{ft.} (8.8') - \frac{9.8'}{2} (0.6 \times 88) \\ = 1,575^* \rightarrow \text{HD02}$$

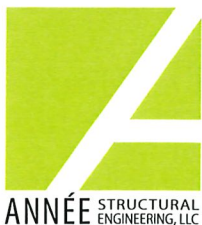
REVIEW LATERAL LOADS @ LINES  $\textcircled{2}$ ,  $\textcircled{3}$  AT LOWER FLOOR WALLS;

$$V_{23E} = 60\% (2,461^*) + 2,943^* = 4,420^* \text{ (SEIS.)}$$

$$V_{23w} = 152 \text{ ft}^2 (16.0) + 1,980^* = 4,412^* \text{ (WIND)}$$

$$V_{23} = 4,420^* / 25.2' = 175^* / \text{ft.} \rightarrow \text{SW6}$$

HOLDOWNS  $\rightarrow$  HD02 BY INSPECTION

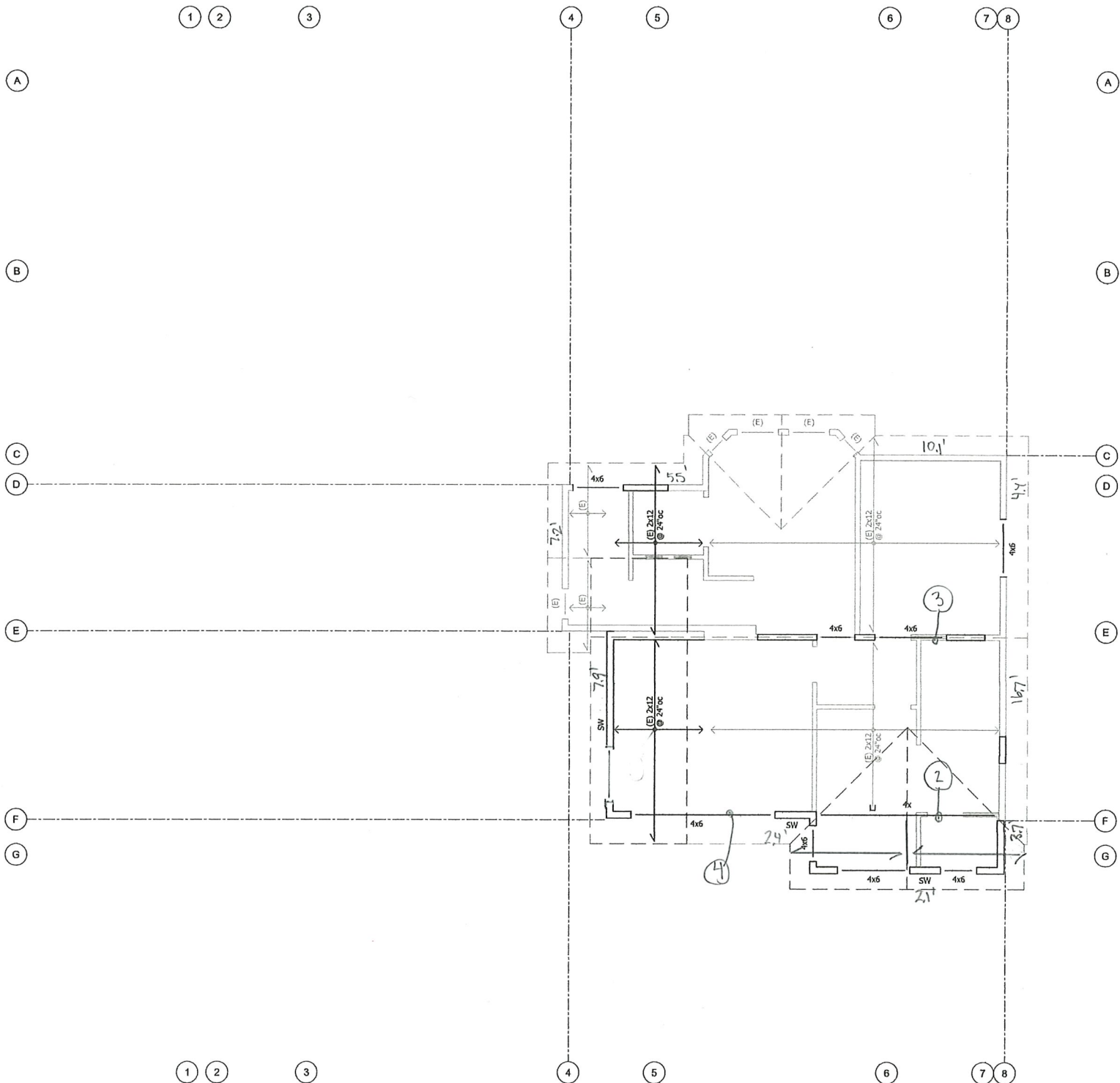


Project \_\_\_\_\_

Designer \_\_\_\_\_

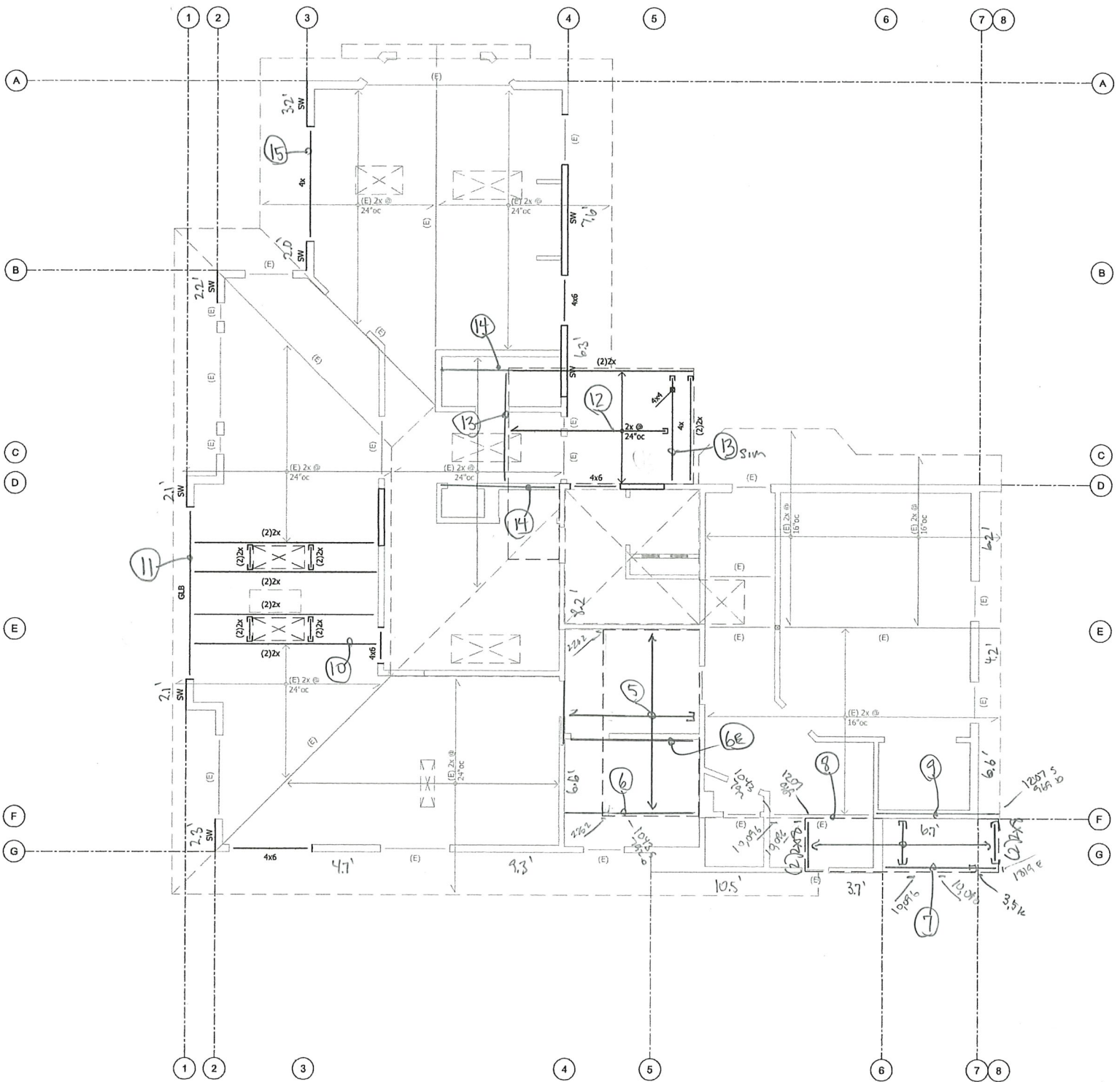
Date \_\_\_\_\_





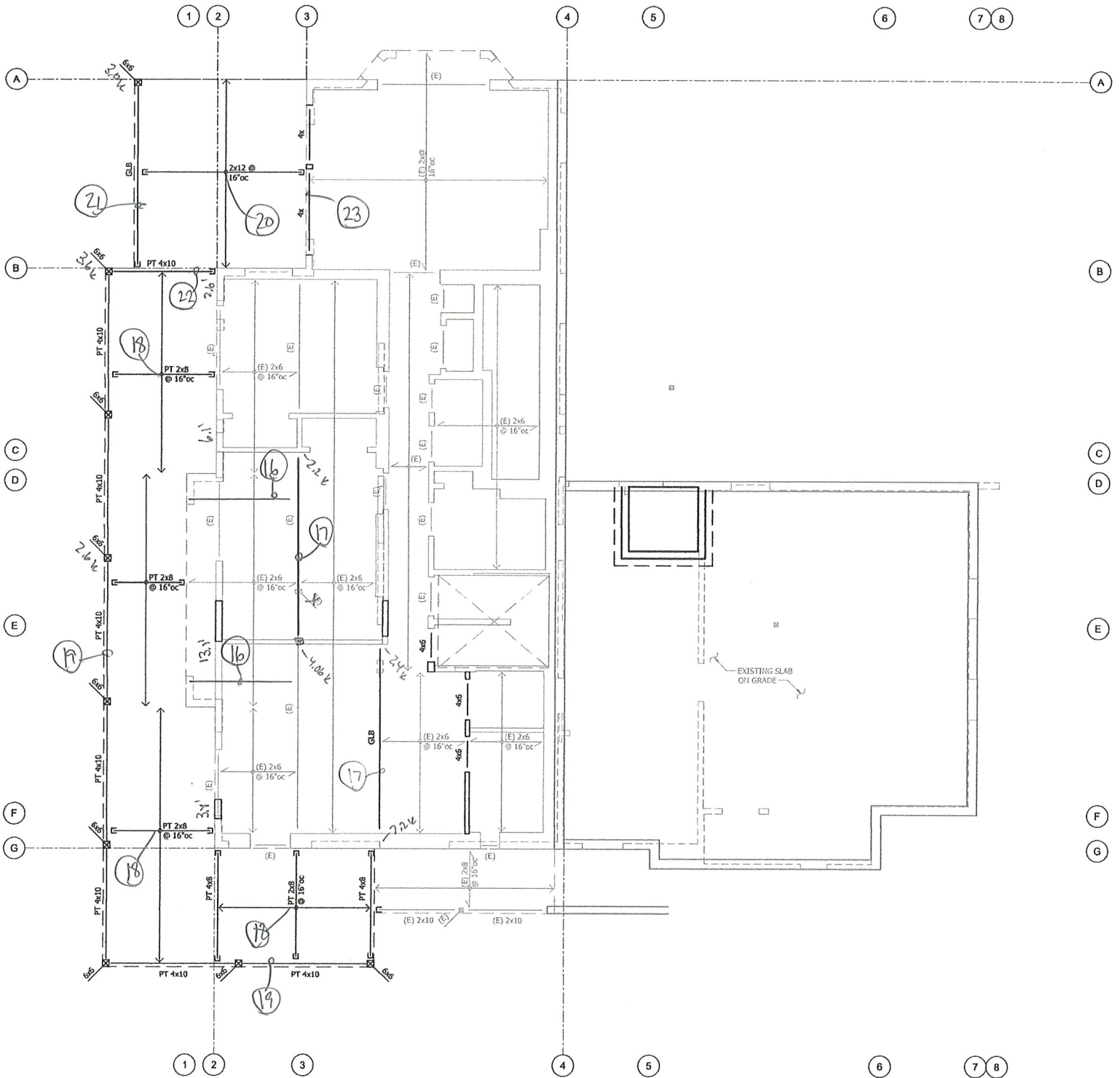
**Upper Roof Framing Plan**  
 SCALE: 1/4" = 1'-0"





**Lower Roof & Upper Floor Framing Plan**  
 SCALE: 1/4" = 1'-0"

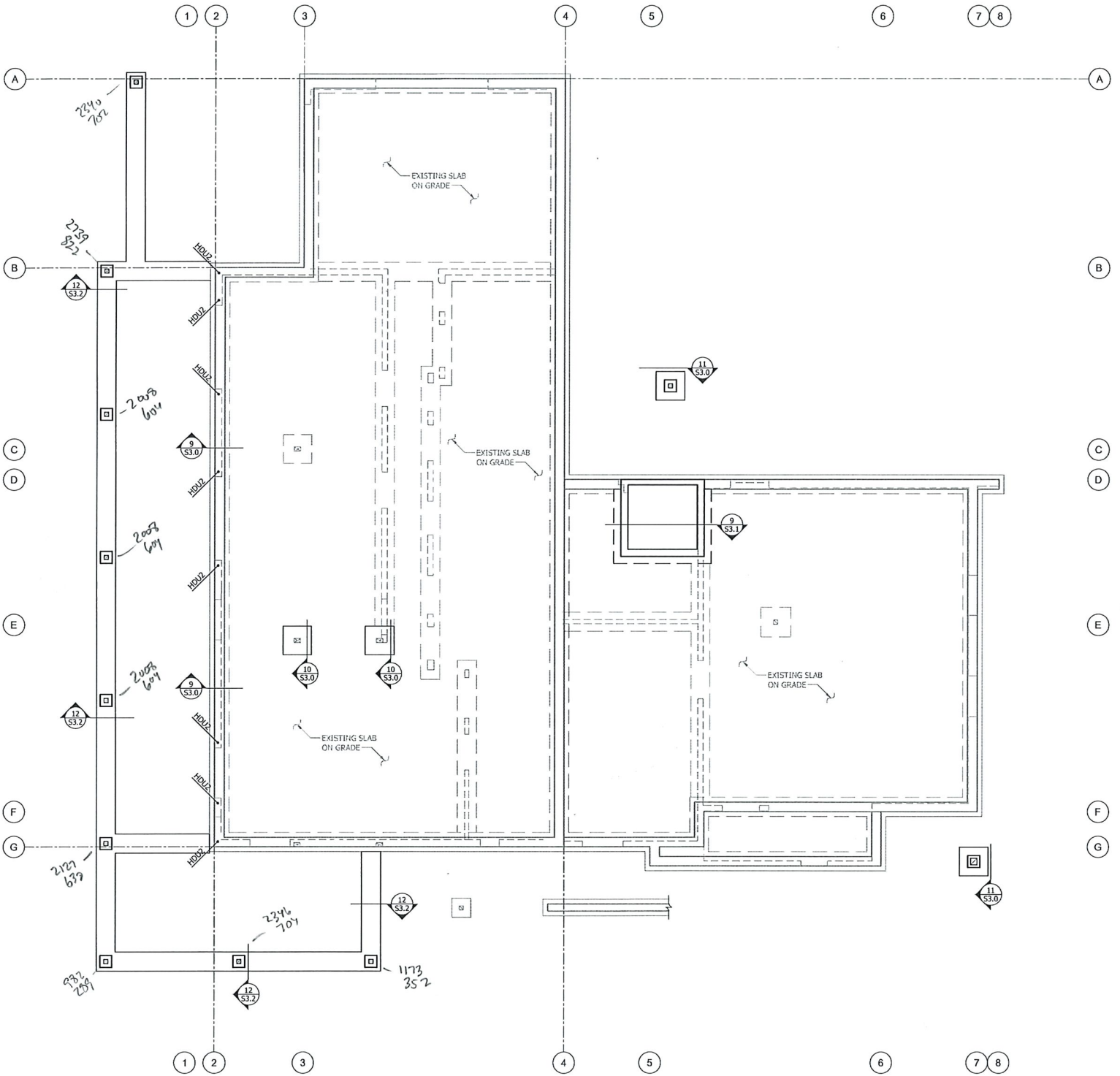




**Main Floor Framing & Foundation Plan**

SCALE: 1/4" = 1'-0"





Foundation Plan  
 SCALE: 1/4" = 1'-0"



GRAVITY ANALYSIS - ROOF FRAMING (25x15, 15x40) #/ft<sup>2</sup>:

①  $L = 12.4'$ ;  $w = \frac{24}{12} (25+15) = 88 \#/ft$

$R=V = 546 \#$ ;  $M = 1,691 \text{ ft}\cdot\#$

$f_v = 49 \text{ psi}$ ;  $f_b = 641 \text{ psi}$ ;  $\Delta_{TL} = 0.20" = 4/205$

$\therefore$  2x12 HP #2 @ 24" oc

② SEE ATTACHED CALC.  $\rightarrow$  3 1/2 x 10 1/2 CLB

③  $L = 5.3'$ ;  $w = \frac{24.9}{2} (25+19) = 548 \#/ft$

$R=V = 1,452 \#$ ;  $M = 1,924 \text{ ft}\cdot\#$

$f_v = 83 \text{ psi}$ ;  $f_b = 704 \text{ psi}$ ;  $\Delta_{TL} = 0.05" = 2/1290$

$\therefore$  4x8 DP #2

④  $L = 10.3'$ ;  $w = 8.1 (25+19) = 356 \#/ft$

$R=V = 1,835 \#$ ;  $M = 4,726 \text{ ft}\cdot\#$

$f_v = 105 \text{ psi}$ ;  $f_b = 1,728 \text{ psi}$ ;  $\Delta_{TL} = 0.41" = 4/304$

$\therefore$  3 1/2 x 7 1/2 CLB

UPPER POR. FRAMING (40x12, 20x12) #/ft<sup>2</sup>:

⑤ SEE ATTACHED CALC.  $\rightarrow$  2x8 HP #2 @ 16" oc

⑥ SEE ATTACHED CALC.  $\rightarrow$  (2) 1 3/4 x 7 1/4 LVL

⑦ SEE ATTACHED CALC.  $\rightarrow$  (2) 1 3/4 x 7 1/4 LVL

⑧ SEE ATTACHED CALC.  $\rightarrow$  (3) 1 3/4 x 7 1/4 LVL

⑨ SEE ATTACHED CALC.  $\rightarrow$  (3) 2x8 HP #2

⑩  $L = 13.3'$ ;  $w = \frac{37.1}{2} (25+15) = 74 \#/ft$

$R=V = 490 \#$ ;  $M = 1,692 \text{ ft}\cdot\#$

$f_v =$ ;  $f_b = 1,342 \text{ psi} = 850(1.15)^2 / 1.3$

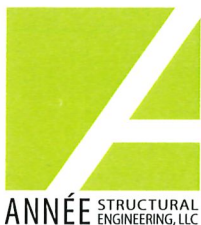
$\therefore$  (2) 2x6 HP #2

⑪  $L = 12.3'$ ;  $w = 191c + 115d = 306 \#/ft$

$R=V = 1,882 \#$ ;  $M = 5,787 \text{ ft}\cdot\#$

$f_v = 90 \text{ psi}$ ;  $f_b = 1,470 \text{ psi}$ ;  $\Delta_{TL} = 0.41" = 4/258$

$\therefore$  3 1/2 x 9 CLB



Project \_\_\_\_\_

Designer \_\_\_\_\_

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Date \_\_\_\_\_



⑫  $L = 10.3'$ ;  $w = \frac{24''}{12''}(25+15) = 80''/ft.$

$R = V = 412''$ ;  $M = 1,061 ft-lb$

$f_v = 57 psi$ ;  $f_b = 969 psi$ ;  $\Delta_{TL} = 0.33'' = 4/378$

$\therefore$  2x8 HP#2 @ 24" oc

⑬  $L = 8.0'$ ;  $w = \frac{16.5''}{2}(25+15) = 327''/ft.$

$R = V = 1,308''$ ;  $M = 2,616 ft-lb$

$f_v = 75 psi$ ;  $f_b = 957 psi$ ;  $\Delta_{TL} = 0.15'' = 4/639$

$\therefore$  4x8 OP#2

⑭  $L = 8.75'$ ;  $P = 1,368''$  @ mid-pt.

$R = V = 922''$ ;  $M = 2,861 ft-lb$

$f_v = 53 psi$ ;  $f_b = 1,946 psi$ ;  $\Delta_{TL} = 0.32'' = 4/323$

$\therefore$  (2) 1 3/4 x 5 1/2 LVL

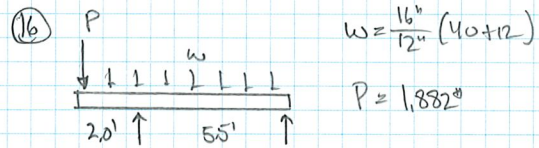
⑮  $L = 8.3'$ ;  $w = 8.4'(25+15) = 336''/ft.$

$R = V = 1,393''$ ;  $M = 2,891 ft-lb$

$f_v = 80 psi$ ;  $f_b = 1,057 psi$ ;  $\Delta_{TL} = 0.18'' = 4/547$

$\therefore$  4x8 OP#2

MAIN FR. FRMNG (40 LL, 12a) #/ft:



SEE ATTACHED CALC.  $\rightarrow$  (3) 1 3/4 x 5 1/2 LVL

⑰  $L = 14.3'$ ;  $w = \frac{11.8''}{2}(40+12) = 307''/ft.$

$R = V = 2,194''$ ;  $M = 7,842 ft-lb$

$f_v = 90 psi$ ;  $f_b = 1,463 psi$ ;  $\Delta_{TL} = 0.48'' = 4/361$

$\therefore$  3 1/2 x 10 1/2 CLB

⑱  $L = 7.4'$ ;  $w = \frac{16''}{12''}(60+8) = 91''/ft.$

$R = V = 335''$ ;  $M = 621 ft-lb$

$f_v = 46 psi$ ;  $f_b = 567 psi$ ;  $\Delta_{TL} = 0.10'' = 4/854$

$\therefore$  PT 2x8 HP#2 @ 16" oc

⑲  $L = 9.8'$ ;  $w = \frac{7.83''}{2}(60+8) = 266''/ft.$

$R = V = 1,305''$ ;  $M = 3,198 ft-lb$

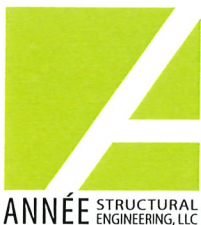
$f_v = 59 psi$ ;  $f_b = 729 psi \leq 850(0.80) 1.2$

$\Delta_{TL} = 0.18'' = 4/658 \therefore$  PT 4x10 HP#2

⑳  $L = 11.7'$ ;  $w = \frac{16''}{12''}(60+18) = 104''/ft.$

$R = V = 607''$ ;  $M = 1,769 ft-lb$ ;  $d_{avg min} = 8.39''$

$\Delta_{TL} = 0.34'' = 4/415 \therefore$  2x10 HP#2 @ 16" oc  
or 2x12 HP#2 (ripped 1/4" per ft.)



Project \_\_\_\_\_  
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Designer \_\_\_\_\_  
 Date \_\_\_\_\_



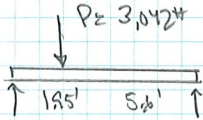
(21)  $L = 13.0'$ ;  $w = \frac{12}{2}(60+18) = 468 \text{ lb/ft}$ .

$R = V = 3,042 \text{ lb}$ ;  $M = 9,887 \text{ ft-lb}$

$f_v = 92 \text{ psi}$ ;  $f_b = 1,598 \text{ psi}$ ;  $\Delta_{HL} = 0.50" \approx \frac{1}{32}$

5 1/2 x 9 GLB

(22)



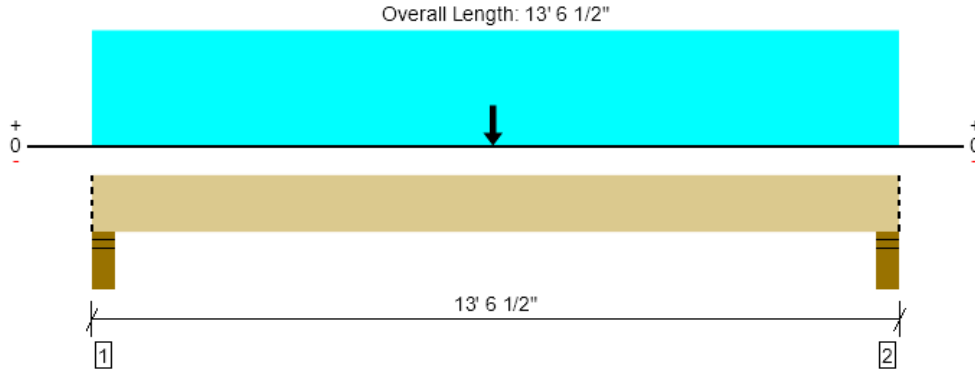
$R = V = 2,256 \text{ lb}$ ;  $M = 4,400 \text{ ft-lb}$

$f_v = 81 \text{ psi}$ ;  $f_b = 681 \text{ psi}$   $\therefore$  PT 4 x 12 HP #2

(23) SEE ATTACHED CALC.  $\rightarrow$  4 x 10 DF #2

Roof Framing, 2 - Beam

**1 piece(s) 3 1/2" x 9" 24F-V4 DF Glulam**



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2177 @ 4"	7796 (5.50")	Passed (28%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1840 @ 1' 2 1/2"	6400	Passed (29%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	7615 @ 6' 8 3/4"	10868	Passed (70%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.314 @ 6' 9 1/4"	0.644	Passed (L/492)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.565 @ 6' 9 1/4"	0.858	Passed (L/273)	--	1.0 D + 1.0 S (All Spans)

Member Length : 13' 6 1/2"  
 System : Roof  
 Member Type : Flush Beam  
 Building Use : Residential  
 Building Code : IBC 2018  
 Design Methodology : ASD  
 Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 1.00 was calculated for positive bending using length L = 12' 10 1/2".
- 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	Snow	Factored	
1 - Stud wall - HF	5.50"	5.50"	1.54"	970	1208	2177	Blocking
2 - Stud wall - HF	5.50"	5.50"	1.53"	968	1206	2174	Blocking

- 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)	13' 7" o/c	
Bottom Edge (Lu)	13' 7" o/c	

- Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 13' 6 1/2"	N/A	7.7	--	
1 - Uniform (PSF)	0 to 13' 6 1/2" (Front)	6' 2 1/8"	19.0	25.0	Default Load
2 - Point (lb)	6' 8 3/4" (Front)	N/A	245	322	

- Side loads are assumed to not induce cross-grain tension.

**Weyerhaeuser Notes**

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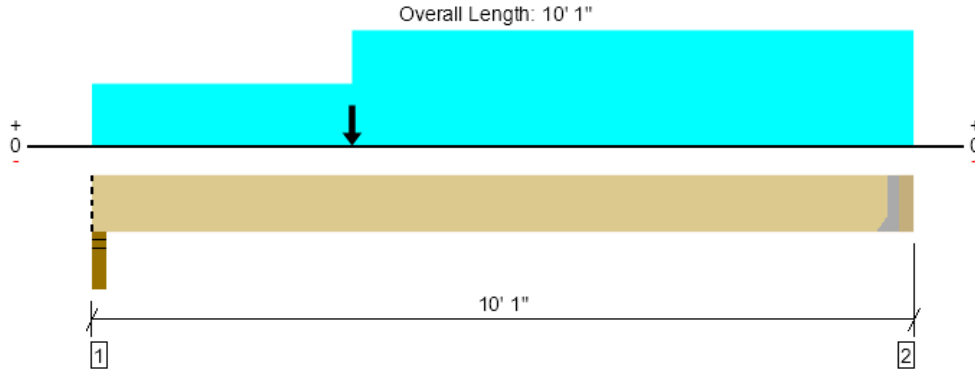
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Mike Annee Annee Structural Engineering LLC (206) 658-5169 mike@anneestructural.com	



10/10/2025 5:46:18 PM UTC  
 ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3  
 File Name: Ford Residence

Upper Floor Framing, 5 - Joist  
**1 piece(s) 2 x 8 HF No.2 @ 16" OC**



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	371 @ 9' 9 1/2"	911 (1.50")	Passed (41%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	340 @ 10 3/4"	1088	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	990 @ 4' 5 3/8"	1284	Passed (77%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.145 @ 5' 7/8"	0.240	Passed (L/793)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.262 @ 4' 11 1/8"	0.479	Passed (L/439)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 9' 9 1/2"  
 System : Floor  
 Member Type : Joist  
 Building Use : Residential  
 Building Code : IBC 2018  
 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 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.50"	181	193	56	374	Blocking
2 - Hanger on 7 1/4" HF beam	3.50"	Hanger <sup>1</sup>	1.50"	133	258	26	391	See note <sup>1</sup>

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- <sup>1</sup> See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 4" o/c	
Bottom Edge (Lu)	9' 10" 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
2 - Face Mount Hanger	LU26	1.50"	N/A	6-10dx1.5	4-10dx1.5	

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

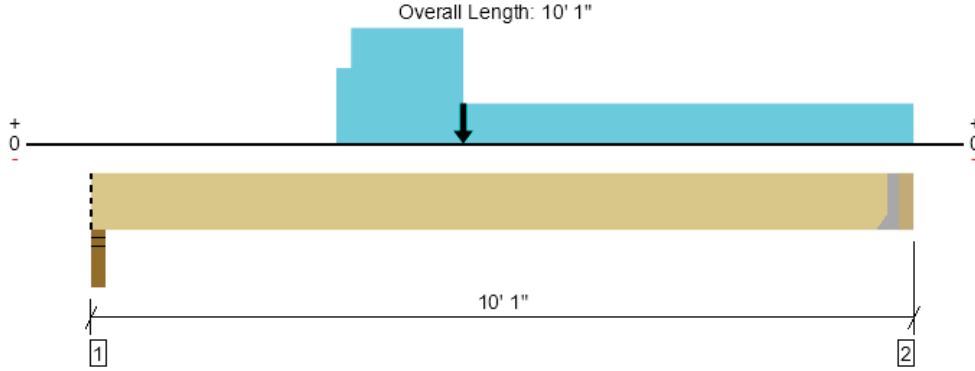
Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	3' 3"	16"	127.0	--	61.0	Default Load
2 - Uniform (PSF)	0 to 3' 3"	16"	8.0	20.0	--	Default Load
3 - Uniform (PSF)	3' 3" to 10' 1"	16"	12.0	40.0	--	Default Load

FortewEB Software Operator	Job Notes
Mike Annee Annee Structural Engineering LLC (206) 658-5169 mike@anneestructural.com	



10/10/2025 5:46:18 PM UTC  
 FortewEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3  
 File Name: Ford Residence

Upper Floor Framing, 6 - Beam  
**2 piece(s) 1 3/4" x 7 1/4" 2.0E Microllam® LVL @ 16" OC**



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1728 @ 9' 9 1/2"	3938 (1.50")	Passed (44%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1678 @ 10 3/4"	5544	Passed (30%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	6852 @ 4' 7 3/4"	8509	Passed (81%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.207 @ 4' 7 3/4"	0.240	Passed (L/556)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.426 @ 4' 11 1/4"	0.479	Passed (L/270)	--	1.0 D + 1.0 S (All Spans)
TJ-Pro™ Rating	64	40	Passed	--	--

Member Length : 9' 9 1/2"  
 System : Floor  
 Member Type : Joist  
 Building Use : Residential  
 Building Code : IBC 2018  
 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 4% increase in the moment capacity has been added to account for repetitive member usage.
- 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: 5/8" Gypsum ceiling.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.50"	849	119	830	1678	Blocking
2 - Hanger on 7 1/4" HF beam	3.50"	Hanger <sup>1</sup>	1.50"	1021	245	752	1773	See note <sup>1</sup>

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- <sup>1</sup> See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 10" o/c	
Bottom Edge (Lu)	9' 10" 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
2 - Face Mount Hanger	HHUS46	3.00"	N/A	14-10d	6-10d	

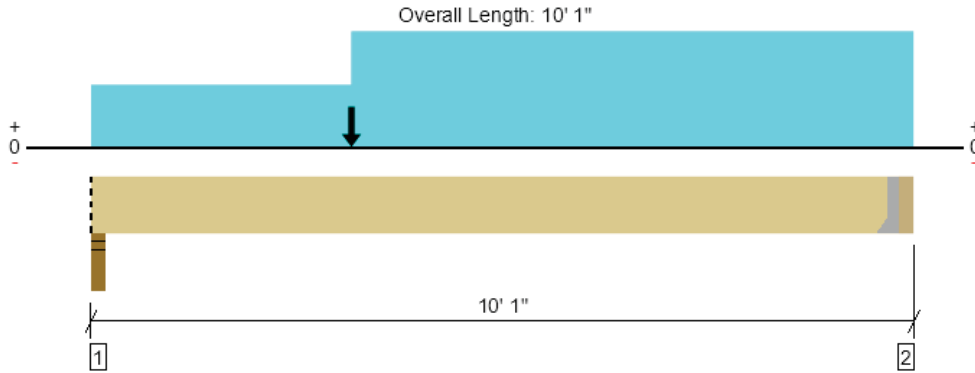
- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PLF)	3' 3" to 10' 1"	N/A	106.0	--	32.0	Default Load
2 - Uniform (PSF)	3' 3" to 10' 1"	16"	12.0	40.0	--	Default Load
3 - Uniform (PLF)	3' 3/4" to 4' 7 3/4"	N/A	154.0	--	202.0	
4 - Point (lb)	4' 7 3/4"	N/A	792	--	1043	

ForteWEB Software Operator	Job Notes
Mike Annee Annee Structural Engineering LLC (206) 658-5169 mike@anneestructural.com	



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 File Name: Ford Residence



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

Design Results	Actual @ Location	Allowed	Result	LDf	Load: Combination (Pattern)
Member Reaction (lbs)	1018 @ 9' 9 1/2"	2734 (1.50")	Passed (37%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	2086 @ 10 3/4"	5220	Passed (40%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	6317 @ 3' 3"	6166	Passed (102%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	0.048 @ 5' 7/8"	0.240	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.087 @ 4' 11 1/8"	0.479	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 9' 9 1/2"  
 System : Floor  
 Member Type : Joist  
 Building Use : Residential  
 Building Code : IBC 2018  
 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 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Seismic	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.50"	181	193	56	2735/-273 5	2095/-1806	Blocking
2 - Hanger on 7 1/4" HF beam	3.50"	Hanger <sup>1</sup>	1.50"	133	258	26	1272/-127 2	1023/-811	See note <sup>1</sup>

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- <sup>1</sup> See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6" o/c	
Bottom Edge (Lu)	6' 7" o/c	

•Maximum allowable bracing intervals based on applied load.

**Connector: Simpson Strong-Tie**

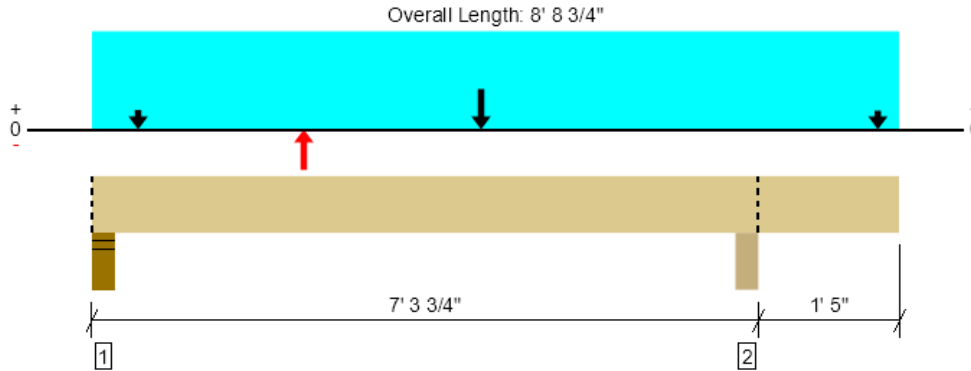
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	LUS28-3	2.00"	N/A	6-16d	4-16d	

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

Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Seismic (1.60)	Comments
1 - Point (PLF)	3' 3"	16"	127.0	--	61.0	3005.0	Default Load
2 - Uniform (PSF)	0 to 3' 3"	16"	8.0	20.0	--	--	Default Load
3 - Uniform (PSF)	3' 3" to 10' 1"	16"	12.0	40.0	--	--	Default Load

Forteweb Software Operator	Job Notes
Mike Annee Annee Structural Engineering LLC (206) 658-5169 mike@anneestructural.com	





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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3119 @ 4"	7796 (5.50")	Passed (40%)	--	1.0 D - 0.525 E + 0.75 L + 0.75 S (Alt Spans)
Shear (lbs)	2564 @ 6' 3"	7714	Passed (33%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	6363 @ 4' 2 1/2"	11383	Passed (56%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	0.019 @ 3' 6 15/16"	0.169	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Total Load Defl. (in)	0.046 @ 3' 5 11/16"	0.338	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)

Member Length : 8' 8 3/4"  
 System : Floor  
 Member Type : Flush Beam  
 Building Use : Residential  
 Building Code : IBC 2018  
 Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Seismic	Factored	
1 - Stud wall - HF	5.50"	5.50"	2.20"	932	206/-11	703	2867/-2867	3119/-1447	Blocking
2 - Column - DF	5.50"	5.50"	1.50"	1321	290	601	2867/-2867	3494/-1214	Blocking

• 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)	8' 9" o/c	
Bottom Edge (Lu)	8' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

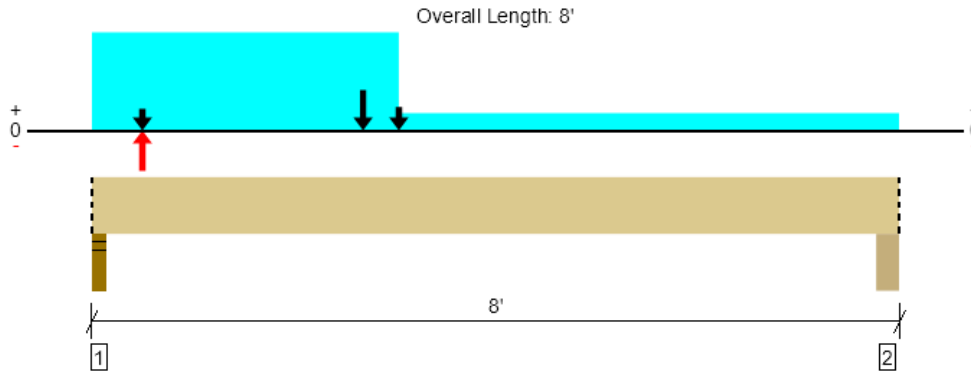
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	0 to 8' 8 3/4"	N/A	7.4	--	--	--	
1 - Uniform (PSF)	0 to 8' 8 3/4" (Front)	1' 10 1/4"	22.0	30.0	--	--	Default Load
2 - Point (lb)	2' 3 1/2" (Front)	N/A	--	--	--	-10096	
3 - Point (lb)	4' 2 1/2" (Front)	N/A	--	--	--	10096	
4 - Uniform (PLF)	0 to 8' 8 3/4" (Front)	N/A	126.0	--	59.0	--	
5 - Point (lb)	6" (Front)	N/A	401	--	527	--	
6 - Point (lb)	8' 6" (Front)	N/A	332	--	231	--	

• Side loads are assumed to not induce cross-grain tension.

ForteWEB Software Operator	Job Notes
Mike Annee Annee Structural Engineering LLC (206) 658-5169 mike@anneestructural.com	



Upper Floor Framing, 8 - Beam  
**3 piece(s) 1 3/4" x 7 1/4" 2.0E Microllam® LVL**



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	5326 @ 2"	7442 (3.50")	Passed (72%)	--	1.0 D - 0.525 E + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	4122 @ 10 3/4"	11571	Passed (36%)	1.60	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	13426 @ 2' 8 1/4"	17075	Passed (79%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	0.057 @ 3' 8 7/8"	0.188	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.153 @ 3' 8 11/16"	0.375	Passed (L/588)	--	1.0 D + 1.0 Lr (All Spans)

Member Length : 8'  
 System : Floor  
 Member Type : Flush Beam  
 Building Use : Residential  
 Building Code : IBC 2018  
 Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- -645 lbs uplift at support located at 2". Strapping or other restraint may be required.

Supports	Bearing Length			Loads to Supports (lbs)						Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Snow	Seismic	Factored	
1 - Stud wall - HF	3.50"	3.50"	2.50"	2360	79	744	1814	2945/-2945	5326/-645	Blocking
2 - Column - DF	5.50"	5.50"	1.50"	877	196	463	284	2945/-2945	2938/-1535	Blocking

• 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)	8' o/c	
Bottom Edge (Lu)	8' o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (1.25)	Snow (1.15)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	0 to 8'	N/A	11.1	--	--	--	--	
1 - Uniform (PSF)	3' 1/2" to 8" (Front)	1' 10 1/4"	22.0	30.0	--	--	--	Default Load
2 - Point (lb)	6" (Front)	N/A	797	--	--	1043	-10096	
3 - Point (lb)	2' 8 1/4" (Front)	N/A	--	--	--	--	10096	
4 - Uniform (PLF)	0 to 3' 1/2" (Front)	N/A	250.0	--	--	223.0	--	
5 - Point (lb)	3' 1/2" (Front)	N/A	1301	--	1207	231	--	
6 - Uniform (PSF)	0 to 3' 1/2" (Front)	1' 11"	15.0	--	--	25.0	--	

• Side loads are assumed to not induce cross-grain tension.

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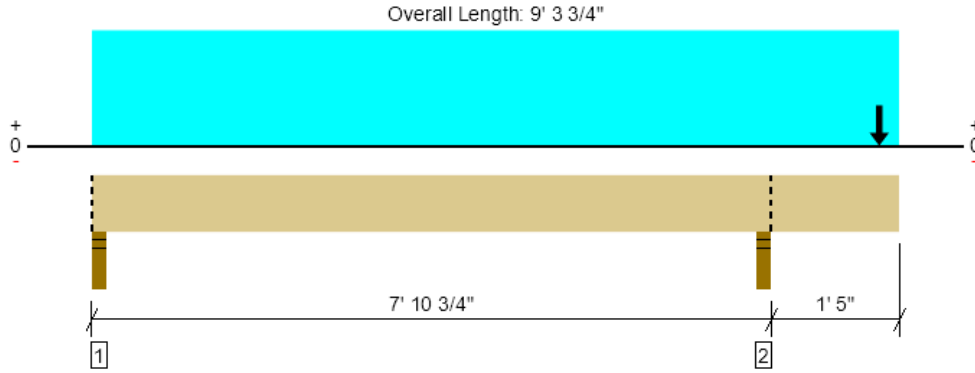
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

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 File Name: Ford Residence

Upper Floor Framing, 9 - Beam  
**3 piece(s) 2 x 8 HF No.2**



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2910 @ 7' 9"	6379 (3.50")	Passed (46%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2228 @ 8' 6"	3752	Passed (59%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	-2979 @ 7' 9"	3853	Passed (77%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.070 @ 9' 3 3/4"	0.200	Passed (2L/534)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.113 @ 9' 3 3/4"	0.200	Passed (2L/332)	--	1.0 D + 1.0 S (All Spans)

Member Length : 9' 3 3/4"  
 System : Floor  
 Member Type : Flush Beam  
 Building Use : Residential  
 Building Code : IBC 2018  
 Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (0.2") and TL (0.2").
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.50"	72	299/-12	-212	371/-141	Blocking
2 - Stud wall - HF	3.50"	3.50"	1.60"	1491	417	1419	2910	Blocking

• 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)	9' 4" o/c	
Bottom Edge (Lu)	9' 4" 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)	0 to 9' 3 3/4"	N/A	8.3	--	--	
1 - Uniform (PSF)	0 to 9' 3 3/4" (Front)	2' 6 1/4"	22.0	30.0	--	Default Load
2 - Point (lb)	9' 1" (Front)	N/A	969	--	1207	

• Side loads are assumed to not induce cross-grain tension.

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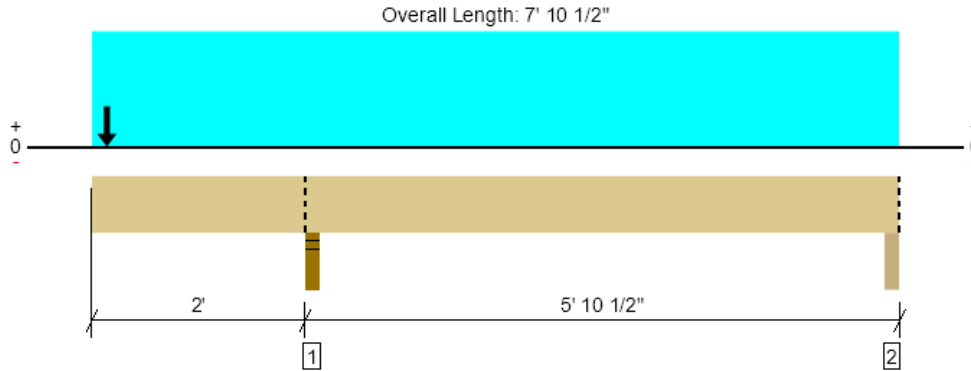
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Design Results	Actual @ Location	Allowed	Result	LDf	Load: Combination (Pattern)
Member Reaction (lbs)	7371 @ 2' 1 3/4"	7442 (3.50")	Passed (99%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	5373 @ 1' 6 1/2"	8778	Passed (61%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	-10734 @ 2' 1 3/4"	10611	Passed (101%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	0.161 @ 0	0.200	Passed (2L/320)	--	1.0 D + 1.0 S (All Spans)
<b>Total Load Defl. (in)</b>	<b>0.257 @ 0</b>	<b>0.215</b>	<b>Failed (2L/200)</b>	<b>--</b>	<b>1.0 D + 1.0 S (All Spans)</b>
TJ-Pro™ Rating	72	40	Passed	--	--

Member Length : 7' 10 1/2"  
 System : Floor  
 Member Type : Joist  
 Building Use : Residential  
 Building Code : IBC 2018  
 Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (0.2") and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 4% increase in the moment capacity has been added to account for repetitive member usage.
- A structural analysis of the deck has not been performed.
- No composite action between deck and joist was considered in analysis.
- Additional considerations for the TJ-Pro™ Rating include: 5/8" Gypsum ceiling.

Total load deflection acceptable when taking seismic overstrength factor into account.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Seismic	Factored	
1 - Stud wall - HF	3.50"	3.50"	3.47"	1047	284	1602	9035/-9035	7371/-5696	Blocking
2 - Beam - GLB	3.50"	3.50"	1.50"	-215	158/-22	-426	2403/-2403	1553/-1897	Blocking

• 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)	7' 11" o/c	
Bottom Edge (Lu)	6" o/c	

•Maximum allowable bracing intervals based on applied load.

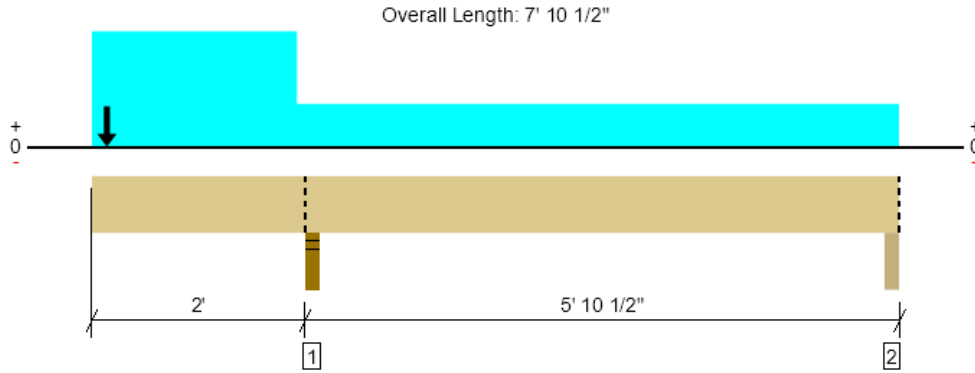
Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Seismic (1.60)	Comments
1 - Uniform (PSF)	0 to 7' 10 1/2"	16"	12.0	40.0	--	--	Default Load
2 - Point (lb)	1 3/4"	N/A	706	--	1176	6632	

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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6621 @ 2' 1 3/4"	7442 (3.50")	Passed (89%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	4802 @ 1' 6 1/2"	8778	Passed (55%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	-9523 @ 2' 1 3/4"	10611	Passed (90%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	0.008 @ 4' 10 7/8"	0.184	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.020 @ 0	0.215	Passed (2L/999+)	--	1.0 D + 1.0 L (Alt Spans)
TJ-Pro™ Rating	72	40	Passed	--	--

Member Length : 7' 10 1/2"  
 System : Floor  
 Member Type : Joist  
 Building Use : Residential  
 Building Code : IBC 2018  
 Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (2L/360) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 4% increase in the moment capacity has been added to account for repetitive member usage.
- A structural analysis of the deck has not been performed.
- No composite action between deck and joist was considered in analysis.
- Additional considerations for the TJ-Pro™ Rating include: 5/8" Gypsum ceiling.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Seismic	Factored	
1 - Stud wall - HF	3.50"	3.50"	3.11"	297	284	9035/-9035	6621/-6146	Blocking
2 - Beam - GLB	3.50"	3.50"	1.50"	4	158/-22	2403/-2403	1686/-1679	Blocking

• 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)	7' 11" o/c	
Bottom Edge (Lu)	7' 11" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Seismic (1.60)	Comments
1 - Uniform (PSF)	0 to 7' 10 1/2"	16"	12.0	40.0	--	Default Load
2 - Point (lb)	1 3/4"	N/A	--	--	6632	
3 - Uniform (PLF)	0 to 2'	N/A	87.7	--	--	

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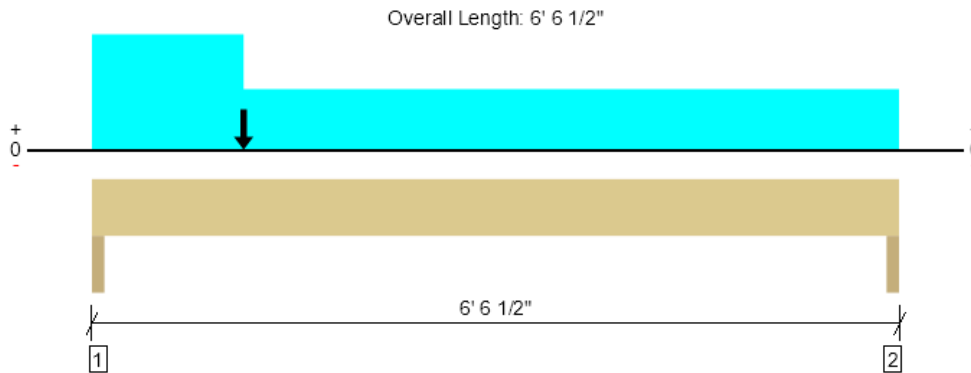
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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	5524 @ 1' 1/2"	6563 (3.00")	Passed (84%)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	4750 @ 1' 1/4"	6216	Passed (76%)	1.60	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	5533 @ 1' 2 3/4"	7187	Passed (77%)	1.60	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.035 @ 3' 2 1/8"	0.210	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.054 @ 3' 1 7/8"	0.315	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

Member Length : 6' 6 1/2"  
 System : Wall  
 Member Type : Header  
 Building Use : Residential  
 Building Code : IBC 2018  
 Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- -519 lbs uplift at support located at 6' 5". Strapping or other restraint may be required.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Seismic	Factored	
1 - Trimmer - HF	3.00"	3.00"	2.53"	1053	1178	956	5468/-5468	5524/-3196	None
2 - Trimmer - HF	3.00"	3.00"	1.50"	492	1178	173	1164/-1164	2116/-519	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 7" o/c	
Bottom Edge (Lu)	6' 7" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	0 to 6' 6 1/2"	N/A	8.2	--	--	--	
1 - Uniform (PSF)	0 to 6' 6 1/2"	6'	18.0	60.0	--	--	Default Load
2 - Uniform (PLF)	0 to 1' 2 3/4"	N/A	214.0	--	210.0	--	
3 - Point (lb)	1' 2 3/4"	N/A	522	--	871	6632	

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PROJECT: **Ford Residence**  
 MEMBER: **Deck columns**

DATE: 11/20/25  
 BY: **MTA**

**NDS 2018 Column Design - Combined Bending and Axial Load**

Section	Grade
A	#2 SPF
B	#1 DF
C	#2 DF
D	#1 HF
E	#2 HF
F	HF-STUD
G	1.8E PSL

Stud Spacing	12
Wind (psf)	0.0
Moment (ft.-lbs.)	0
Moment - Strong	0
Moment - Weak	0
Axial Load (plf)	3,600
Load/stud (lbs.)	3,600
Ke	1.0

# of members	1
Section Mark	<b>4x4</b>
Grade Mark	E
Axial Load (lbs.)	3,600
Moment Strong Axis (ft.-lbs.)	0
Moment Weak Axis (ft.-lbs.)	175
Load Duration Factor	1.00
Repetitive Factor Cr	1
Incised Lumber (Y/N)	Y
Unbraced Length Strong Axis (ft.)	7.21
Unbraced Length Weak Axis (ft.)	7.21
Grade	<b>#2 HF</b>
<b>Axial alone = fc/F*c</b>	<b>0.55</b>
Interaction Eq. Term 1	0.30
Interaction Eq. Term 2	0.00
Interaction Eq. Term 3	0.48
<b>Total Interaction Eq.</b>	<b>0.78</b>

Mk	Section
A	2x4
B	2x6
C	2x8
D	2x10
E	2x12
F	4x4
G	4x6
H	4x8
I	4x10
J	4x12
K	6x6
L	6x8
M	6x10
N	6x12
P	3-1/2x7
<b>1</b>	<b>4x4</b>

Strong axis deflection - uniform load over simple span of 7.21' =	<b>0.00</b>	L / 0
Weak axis deflection - uniform load over simple span of 7.21' =	0.10	L / 859
Strong axis deflection - point load at center of 7.21' span =	0.00	L / 0
Weak axis deflection - point load at center of 7.21' span =	0.08	L / 1074

Fbx (psi) = 1,275  
 Fby (psi) = 1,500  
 Fc (psi) = 1,725  
 Ex (psi) = 1.30E+06  
 Ex min (psi) = 4.70E+05  
 Ey (psi) = 1.30E+06  
 Ey min (psi) = 4.70E+05

fc = P/A (psi) = 293.9	AXIAL
F*c = Fc x Cd x Ci (psi) = 1380.0	
K*(le2/d2) = 24.7	<b>OK</b>
K*(le1/d1) = 24.7	<b>OK</b>
= 600.6	
F' = Fce/F*c = 0.435	
c = 0.8	
(1+F')/2c = 0.897	
Cp = 0.387	Column Stability Factor
<u>F*c = F*c x Cp (psi) = 533.4</u>	
fc/F*c = <b>0.551</b>	
(fc/F*c)^2 = <b>0.30</b>	Interaction Equation, 1st term

$fb_1 = M/S$ (psi) = 0.0	STRONG AXIS BENDING
$Fb^* = F_b \times C_d \times C_r \times C_i$ (psi) = 1020.0	
$l_u = 86.5$	in.
$l_e = 151.5$	in.
$R_b = \text{sq. rt.}(l_e \times d/b^2)$ 3.9	<b>OK</b>
$F_{be} = 1.2 \times E'_{min}/R_b^2$ (psi) = 35197.1	<b>OK</b>
$F = F_{be}/F_b^* = 34.507$	
$(1+F)/1.9 = 18.688$	
$CL = 0.999$	Beam Stability Factor
$F'_{b1}$ (psi) = <u>1018.5</u>	
$fb_1/F'_{b1} = \mathbf{0.000}$	
(psi) = 600.6	<b>OK</b>
$1-(f_c/F_{ce1}) = 0.511$	
$fb_1/[F'_{b1} \times (1-(f_c/F_{ce2}))] = \mathbf{0.00}$	Interaction Equation, 2nd term

$fb_2 = M/S$ (psi) = 293.9	WEAK AXIS BENDING
$Fb^* = F_b \times C_d \times C_r \times C_i$ (psi) = 1200.0	
$F_{be} = 1.2 \times E'_{min}/R_b^2$ (psi) = 35197.1	<b>OK</b>
$F = F_{be}/F_b^* = 29.331$	
$(1+F)/1.9 = 15.964$	
$CL = 0.998$	Beam Stability factor
$F'_{b2}$ (psi) = <u>1197.9</u>	
$fb_2/F'_{b2} = \mathbf{0.245}$	
(psi) = 600.6	<b>OK</b>
$1-(f_c/F_{ce2}) = 0.511$	
$fb_1/F_{be} = 0.000$	
$fb_2/[F'_{b2} \times (1-(f_c/F_{ce2})) - (fb_1/F_{be})^2] = \mathbf{0.48}$	Interaction Equation, 3rd term



**Compressive Member Design Guide  
Doug Fir No. 2 4x Posts**

$E' = 1.6E+06$  psi       $F_c = 1350$  psi       $F_{c\perp} = 625$  psi

height (feet)						8	9	10	11	12	13	14	18
height (in.)						96	108	120	132	144	156	168	216
Column	d (in.)	area (sq.in.)	$C_D$	$C_F$	$P_{\perp}$ (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)
4 X 4	3.5	12.25	1.00	1.15	7656	7664	6320	5261	4430	3772	3245	2819	1735
4 X 6	3.5	19.25	1.00	1.10	12031	11627	9561	7946	6683	5686	4890	4246	2610
4 X 8	3.5	25.38	1.00	1.05	15863	14766	12110	10048	8441	7176	6168	5354	3289
4 X 10	3.5	32.38	1.00	1.00	20238	18101	14806	12265	10293	8744	7512	6517	4000

Fce= 638.021 504.115 408.333 337.466 283.5648 241.6174 208.3333 126.0288

**Compressive Member Design Guide  
Douglas Fir No 1: (6X Posts**

$E' = 1.6E+06$  psi       $F_c = 1000$  psi       $F_{c\perp} = 625$  psi

height (feet)						8	9	10	11	12	13	14	18
height (in.)						96	108	120	132	144	156	168	216
Column	d (in.)	area (sq.in.)	$C_D$	$C_F$	$P_{\perp}$ (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)
6X6	5.5	30.25	1.00	1.00	18906	24841	23015	20989	18906	16902	15061	13421	8710
6X8	5.5	41.25	1.00	1.00	25781	33874	31384	28621	25781	23048	20538	18302	11877
6X10	5.5	52.25	1.00	1.00	32656	42908	39753	36253	32656	29194	26015	23183	15044

Fce= 1575.52 1244.86 1008.33 833.333 700.2315 596.6469 514.4558 311.214

**Compressive Member Design Guide  
PSL Posts**

$E' = 2.0E+06$  psi       $F_c = 2900$  psi       $F_{c\perp} = 750$  psi

height (feet)						8	9	10	11	12	13	14	18
height (in.)						96	108	120	132	144	156	168	216
Column	d (in.)	area (sq.in.)	$C_D$	$C_F$	$P_{\perp}$ (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)	Pmax (lbs.)
5.25X5.25	5.25	27.56	1.00	1.00	17225	40889	34029	28494	24081	20556	17719	15412	9512
7X7	7	49.00	1.00	1.00	30625	72697	60502	50660	42814	36548	31503	27402	16911

Fce= 1794.43 1417.82 1148.44 949.122 797.526 679.5488 585.9375 354.456

note: these tables do not account for lateral uniform loading (wind)

## Concrete Beam

Project File: Ford grade beam.ec6

LIC# : KW-06019266, Build:20.24.02.28

Annee Structural Engineering LLC

(c) ENERCALC INC 1983-2023

DESCRIPTION: --None--

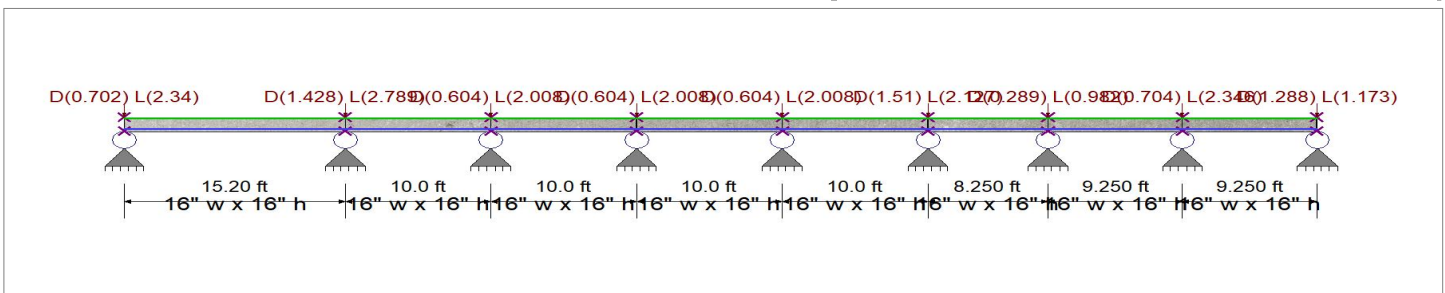
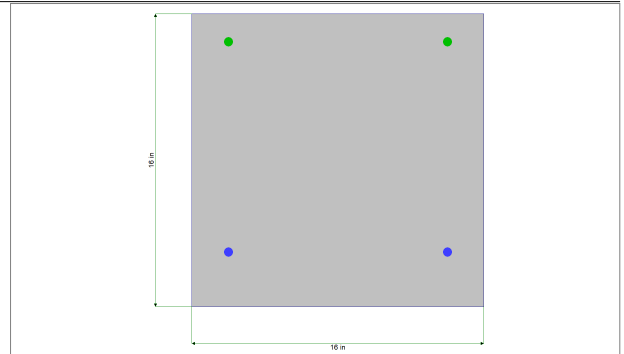
### CODE REFERENCES

Calculations per ACI 318-19, IBC 2021, ASCE 7-16

Load Combination Set : IBC 2021

### General Information

$f'_c$	=	2.50 ksi	$\phi$ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2} + 7.50$	=	375.0 psi		Shear :	0.750
$\psi$ Density	=	145.0 pcf	$\beta_1$	=	0.850
$\lambda$ 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 = 16.0 in, Height = 16.0 in

Span #1 Reinforcing....

2-#4 at 3.0 in from Bottom, from 0.0 to 15.20 ft in this span

2-#4 at 1.50 in from Top, from 0.0 to 15.20 ft in this span

Span #2 Reinforcing....

2-#4 at 3.0 in from Bottom, from 0.0 to 10.0 ft in this span

2-#4 at 1.50 in from Top, from 0.0 to 10.0 ft in this span

Span #3 Reinforcing....

2-#4 at 3.0 in from Bottom, from 0.0 to 10.0 ft in this span

2-#4 at 1.50 in from Top, from 0.0 to 10.0 ft in this span

Span #4 Reinforcing....

2-#4 at 3.0 in from Bottom, from 0.0 to 10.0 ft in this span

2-#4 at 1.50 in from Top, from 0.0 to 10.0 ft in this span

Span #5 Reinforcing....

2-#4 at 3.0 in from Bottom, from 0.0 to 10.0 ft in this span

2-#4 at 1.50 in from Top, from 0.0 to 10.0 ft in this span

Span #6 Reinforcing....

2-#4 at 3.0 in from Bottom, from 0.0 to 8.250 ft in this span

2-#4 at 1.50 in from Top, from 0.0 to 8.250 ft in this span

Span #7 Reinforcing....

2-#4 at 3.0 in from Bottom, from 0.0 to 9.250 ft in this span

2-#4 at 1.50 in from Top, from 0.0 to 9.250 ft in this span

Span #8 Reinforcing....

2-#4 at 3.0 in from Bottom, from 0.0 to 9.250 ft in this span

2-#4 at 1.50 in from Top, from 0.0 to 9.250 ft in this span

### Beam self weight calculated and added to loads

#### Load for Span Number 1

Point Load : D = 0.7020, L = 2.340 k @ 0.0 ft

#### Load for Span Number 2

Point Load : D = 1.428, L = 2.789 k @ 0.0 ft

#### Load for Span Number 3

Point Load : D = 0.6040, L = 2.008 k @ 0.0 ft

#### Load for Span Number 4

Point Load : D = 0.6040, L = 2.008 k @ 0.0 ft

#### Load for Span Number 5

Point Load : D = 0.6040, L = 2.008 k @ 0.0 ft

#### Load for Span Number 6

## Concrete Beam

Project File: Ford grade beam.ec6

LIC# : KW-06019266, Build:20.24.02.28

Annee Structural Engineering LLC

(c) ENERCALC INC 1983-2023

DESCRIPTION: --None--

### Applied Loads

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

Point Load : D = 1.510, L = 2.127 k @ 0.0 ft

#### Load for Span Number 7

Point Load : D = 0.2890, L = 0.9820 k @ 0.0 ft

#### Load for Span Number 8

Point Load : D = 1.288, L = 1.173 k @ 9.250 ft

Point Load : D = 0.7040, L = 2.346 k @ 0.0 ft

### DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.298 : 1		
Section used for this span	Typical Section		
Mu : Applied	6.916	k-ft	
Mn * Phi : Allowable	23.243	k-ft	
Location of maximum on span	6.218 ft		
Span # where maximum occurs	Span # 1		

### Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360.0	L Only
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0	L Only
Max Downward Total Deflection	0.010 in	Ratio =	18268 >=180.0	Span: 8 : +D+L
Max Upward Total Deflection	-0.001 in	Ratio =	107330 >=180.0	Span: 8 : +D+L

### Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5	Support 6	Support 7	Support 8	Support 9
Max Upward from all Load Conditions	4.638	8.255	4.651	5.321	5.203	6.007	3.370	5.800	3.393
Max Upward from Load Combinations	4.638	8.255	4.651	5.321	5.203	6.007	3.370	5.800	3.393
Max Upward from Load Cases	2.340	5.466	2.643	3.313	3.195	3.880	2.388	3.454	2.220
D Only	2.298	5.466	2.643	3.313	3.195	3.880	2.388	3.454	2.220
+D+L	4.638	8.255	4.651	5.321	5.203	6.007	3.370	5.800	3.393
+D+0.750L	4.053	7.558	4.149	4.819	4.701	5.475	3.124	5.214	3.100
+0.60D	1.379	3.280	1.586	1.988	1.917	2.328	1.433	2.072	1.332
L Only	2.340	2.789	2.008	2.008	2.008	2.127	0.982	2.346	1.173

### Shear Stirrup Requirements

Between 0.00 to 14.85 ft,  $V_u \leq \Phi \lambda \sqrt{f'c} b_w d$ , Req'd Vs = Not Req'd per 9.3.6.1, Stirrups are not required.

Between 15.20 to 15.20 ft,  $\Phi V_c < V_u$ , Req'd Vs = 0.6943, use #3 stirrups spaced at 7.000 in

Between 15.43 to 81.74 ft,  $V_u \leq \Phi \lambda \sqrt{f'c} b_w d$ , Req'd Vs = Not Req'd per 9.3.6.1, Stirrups are not required.

### Detailed Shear Information

Load Combination	Span Number	Distance 'd' (ft)	(in)	Vu Actual (k)	Design (k)	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+1.60L	1	0.00	13.00	6.50	6.50	0.00	1.00	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0
+1.40D	1	2.07	13.00	1.49	1.49	3.86	0.42	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0
+1.40D	1	4.15	13.00	0.74	0.74	6.16	0.13	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0
+1.40D	1	6.22	13.00	-0.01	0.01	6.92	0.00	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0
+1.40D	1	8.29	13.00	-0.76	0.76	6.12	0.13	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0
+1.40D	1	10.36	13.00	-1.51	1.51	3.77	0.43	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0
+1.40D	1	12.44	14.50	-2.25	2.25	0.12	1.00	7.54	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.5	0.0
+1.40D	1	14.51	14.50	-3.00	3.00	5.57	0.65	7.54	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.5	0.0
+1.40D	2	16.11	14.50	2.07	2.07	5.69	0.44	7.54	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.5	0.0
+1.40D	2	17.47	14.50	1.58	1.58	3.20	0.60	7.54	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.5	0.0
+1.40D	2	18.84	14.50	1.09	1.09	1.38	0.95	7.54	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.5	0.0
+1.40D	2	20.20	14.50	0.60	0.60	0.23	1.00	7.54	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.5	0.0
+1.40D	2	21.56	13.00	0.11	0.11	0.25	0.46	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0
+1.40D	2	22.93	13.00	-0.39	0.39	0.06	1.00	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0
+1.40D	2	24.29	14.50	-0.88	0.88	0.80	1.00	7.54	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.5	0.0
+1.40D	3	25.65	14.50	1.48	1.48	1.04	1.00	7.54	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.5	0.0
+1.40D	3	27.02	13.00	0.99	0.99	0.65	1.00	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0
+1.40D	3	28.38	13.00	0.50	0.50	1.67	0.32	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0
+1.40D	3	29.75	13.00	0.01	0.01	2.01	0.00	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0
+1.40D	3	31.11	13.00	-0.48	0.48	1.69	0.31	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0
+1.40D	3	32.47	13.00	-0.98	0.98	0.69	1.00	7.24	$V_u \leq \Phi \lambda \sqrt{f'c} b_w d$	Req'd	7.2	0.0

Project Title:  
 Engineer:  
 Project ID:  
 Project Descr:

**Concrete Beam**

Project File: Ford grade beam.ec6

LIC# : KW-06019266, Build:20.24.02.28

Annee Structural Engineering LLC

(c) ENERCALC INC 1983-2023

DESCRIPTION: --None--

**Detailed Shear Information**

Load Combination	Span Number	Distance 'd'		Vu (k)		Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
		(ft)	(in)	Actual	Design							
+1.40D	3	33.84	14.50	-1.47	1.47	0.98	1.00	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.20D+1.60L	4	35.20	14.50	5.51	5.51	2.84	1.00	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	4	36.56	14.50	1.34	1.34	1.15	1.00	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	4	37.93	13.00	0.85	0.85	0.34	1.00	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	4	39.29	13.00	0.36	0.36	1.16	0.33	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	4	40.65	13.00	-0.14	0.14	1.31	0.11	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	4	42.02	13.00	-0.63	0.63	0.79	0.86	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	4	43.38	14.50	-1.12	1.12	0.40	1.00	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	4	44.75	14.50	-1.61	1.61	2.27	0.86	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	5	46.11	14.50	1.52	1.52	1.50	1.00	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	5	47.47	13.00	1.03	1.03	0.24	1.00	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	5	48.84	13.00	0.54	0.54	1.30	0.45	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	5	50.20	13.00	0.05	0.05	1.70	0.03	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	5	51.56	13.00	-0.45	0.45	1.43	0.34	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	5	52.93	13.00	-0.94	0.94	0.49	1.00	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	5	54.29	14.50	-1.43	1.43	1.13	1.00	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	6	55.58	14.50	1.42	1.42	2.02	0.85	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	6	56.70	14.50	1.02	1.02	0.64	1.00	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	6	57.83	13.00	0.61	0.61	0.27	1.00	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	6	58.95	13.00	0.21	0.21	0.73	0.30	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	6	60.08	13.00	-0.20	0.20	0.74	0.29	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	6	61.20	13.00	-0.61	0.61	0.28	1.00	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	6	62.33	14.50	-1.01	1.01	0.63	1.00	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.20D+1.60L	7	63.45	14.50	3.22	3.22	1.71	1.00	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	7	64.71	14.50	1.07	1.07	0.36	1.00	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	7	65.97	13.00	0.61	0.61	0.70	0.95	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	7	67.23	13.00	0.16	0.16	1.18	0.14	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	7	68.50	13.00	-0.30	0.30	1.09	0.30	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	7	69.76	13.00	-0.76	0.76	0.42	1.00	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	7	71.02	14.50	-1.21	1.21	0.82	1.00	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	7	72.28	14.50	-1.67	1.67	2.63	0.77	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	8	73.54	14.50	1.73	1.73	1.78	1.00	7.54	Vu <= Phi*lambda	7.5	7.5	0.0
+1.40D	8	74.80	13.00	1.27	1.27	0.11	1.00	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	8	76.06	13.00	0.82	0.82	1.43	0.62	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	8	77.33	13.00	0.36	0.36	2.18	0.18	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	8	78.59	13.00	-0.09	0.09	2.35	0.04	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	8	79.85	13.00	-0.55	0.55	1.95	0.30	7.24	Vu <= Phi*lambda	7.2	7.2	0.0
+1.40D	8	81.11	13.00	-1.00	1.00	0.97	1.00	7.24	Vu <= Phi*lambda	7.2	7.2	0.0

**Maximum Forces & Stresses for Load Combinations**

Load Combination	Segment	Location (ft)		Bending Stress Results (k-ft)		
		Span #	along Beam	Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope						
	Span # 1	1	15.200	6.92	23.24	0.30
	Span # 2	2	10.000	-7.73	28.96	0.27
	Span # 3	3	10.000	-2.88	28.96	0.10
	Span # 4	4	10.000	-3.31	28.96	0.11
	Span # 5	5	10.000	-3.04	28.96	0.10
	Span # 6	6	8.250	-2.58	28.96	0.09
	Span # 7	7	9.250	-2.99	28.96	0.10
	Span # 8	8	9.250	-3.36	28.96	0.12
+1.40D	Span # 1	1	15.200	6.92	23.24	0.30
	Span # 2	2	10.000	-7.73	28.96	0.27
	Span # 3	3	10.000	-2.88	28.96	0.10
	Span # 4	4	10.000	-3.31	28.96	0.11
	Span # 5	5	10.000	-3.04	28.96	0.10
	Span # 6	6	8.250	-2.58	28.96	0.09
	Span # 7	7	9.250	-2.99	28.96	0.10
	Span # 8	8	9.250	-3.36	28.96	0.12
+1.20D+1.60L						

Project Title:  
 Engineer:  
 Project ID:  
 Project Descr:

**Concrete Beam**

Project File: Ford grade beam.ec6

LIC# : KW-06019266, Build:20.24.02.28

Annee Structural Engineering LLC

(c) ENERCALC INC 1983-2023

**DESCRIPTION: --None--**

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft)		
			Mu : Max	Phi*Mnx	Stress Ratio
Span # 1	1	15.200	5.93	23.24	0.26
Span # 2	2	10.000	-6.63	28.96	0.23
Span # 3	3	10.000	-2.47	28.96	0.09
Span # 4	4	10.000	-2.84	28.96	0.10
Span # 5	5	10.000	-2.61	28.96	0.09
Span # 6	6	8.250	-2.21	28.96	0.08
Span # 7	7	9.250	-2.56	28.96	0.09
Span # 8	8	9.250	-2.88	28.96	0.10
+0.90D					
Span # 1	1	15.200	4.45	23.24	0.19
Span # 2	2	10.000	-4.97	28.96	0.17
Span # 3	3	10.000	-1.85	28.96	0.06
Span # 4	4	10.000	-2.13	28.96	0.07
Span # 5	5	10.000	-1.95	28.96	0.07
Span # 6	6	8.250	-1.66	28.96	0.06
Span # 7	7	9.250	-1.92	28.96	0.07
Span # 8	8	9.250	-2.16	28.96	0.07

**Overall Maximum Deflections**

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L	1	0.0100	6.218	D Only	-0.0003	15.655
+D+L	2	0.0001	10.455	D Only	-0.0011	3.182
D Only	3	0.0011	5.000		0.0000	3.182
+D+L	4	0.0005	5.000		0.0000	3.182
D Only	5	0.0008	5.000	D Only	-0.0000	10.375
+D+L	6	0.0002	4.125	D Only	-0.0000	1.125
D Only	7	0.0004	3.784	+D+L	-0.0000	8.830
+D+L	8	0.0012	5.466		0.0000	8.830