Yaroslavsky Residence

CALCULATION PACKAGE COVER SHEET

Project Name: Yaroslavsky Residence

Project Number: 8119

Engineer of Record: Dustin Willms, P.E.

Project Architect: Andres Villaveces, Metrica LLC

Site Address: 9319 SE 43rd St. Mercer Island, WA 98040

Submission: Building Permit

Date: 05 March 2021

(Affix Engineer of Record Professional Seal Here)

Fast + Epp

PROJECT NAME: Yaroslavsky Residence

PROJECT NUMBER: 8119 DATE: 05 March 2021

DESIGN: BW, DW

INDEX OF CALCULATIONS

1.1 Project Description

- New custom home on Mercer Island, WA
- 2x6 exterior and 2x4 interior wood frame walls
- TJI/LVL joist floors with plywood sheathing
- Lateral – plywood sheathed wood shear walls and steel ordinary moment frame
- Foundation – concrete spread footings
- Primary codes (see general notes for full list):
    - SBC 2018
    - IBC 2018
    - ASCE 7-16

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PROJECT:	**Yaroslavsky Residence**	**PROJECT NUMBER:**	**8119**
SUBJECT:	**Gravity Loading**	**DATE:**	**20210210**
DESIGN BY:	**BJW**		

NOTES:

INPUT
RESULT

1.2 - DEAD LOAD (PSF)

LOWER LEVEL	**49**	**EXTERIOR DECKS**	**30**
5" Concrete Slab On Grade	49	Floor - allow for heavy build-up	22
		Waterproofing & insulation	3
MAIN/UPPER LEVEL	**30**	Plywood & I-Joists	5
Hardwood finish flooring	1.95		
Floor topping (3/4" underlayment)	7.5		
Floor mat (1/8" sound attenuation)	0.1	**ROOF**	**15**
Subflooring (23/32" plywood)	2	Roofing	2
Structural members (11-7/8" I-Joists @ 12" O.C.)	3	Plywood & I-Joists	5
Insulation (3-1/2" unfaced glass fiber)	1.75	Mechanical	3
Resilient channels (25 ga. @ 16" O.C.)	0.1	Finishes	5
Ceiling (2 layers of 5/8" gypsum board)	3.6		
Partitions (blanket)	10		

1.3 - LIVE LOAD (PSF) [ASCE 7-16 Table 4.3-1]

RESIDENTIAL (TYP.)	40
EXTERIOR DECKS	60
ROOF LIVE LOAD	20

1.4 - SNOW LOAD (PSF) [2018 IRC Table R301.2(1) w/ Mercer Island Amendments]

SNOW LOAD	**30**
FLAT ROOF SNOW LOAD	25
RAIN ON SNOW LOAD	5

1.5 | WIND LOADS

Hazards by Location

Search Information

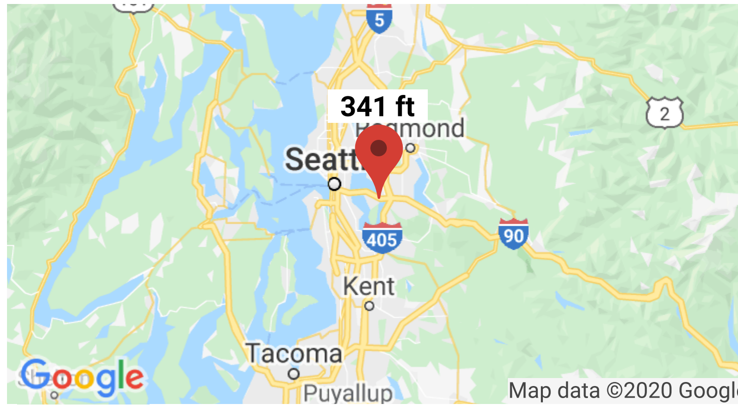
Address: 9319 SE 43rd St, Mercer Island, WA 98040, USA

Coordinates: 47.5693472, -122.2142869

Elevation: 341 ft

Timestamp: 2020-10-30T18:33:30.626Z

Hazard Type: Wind



ASCE 7-16

MRI 10-Year	67 mph
MRI 25-Year	73 mph
MRI 50-Year	78 mph
MRI 100-Year	83 mph
Risk Category I	92 mph
Risk Category II	98 mph
Risk Category III	105 mph
Risk Category IV	108 mph

ASCE 7-10

MRI 10-Year	72 mph
MRI 25-Year	79 mph
MRI 50-Year	85 mph
MRI 100-Year	91 mph
Risk Category I	100 mph
Risk Category II	110 mph
Risk Category III-IV	115 mph

ASCE 7-05

ASCE 7-05 Wind Speed	85 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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INPUT
RESULT

PROJECT:	**Yaroslavsky Residence**	**PROJECT NUMBER:**	**8119**
SUBJECT:	**MWFRS Total Building Wind Load**	**DATE:**	**2021-03-02**
DESIGN BY:	**BJW**		

NOTES: X-DIRECTION

ASCE 7-16- WIND LOADS (ALL HEIGHTS)

Exposure Category		B	Sec. 26.7.3
Basic Wind Speed	**V_ult=**	98	mph
Directionality Factor	K_d =	0.85	Table 26.6-1
Topographic Effects*	K_{zt} =	1.9	Fig. 26.8-1
Gust Effect Factor	G_{used} =	0.85	Sec. 26.11
Enclosure Type		Enclosed	Sec. 26.12
Importance Factor	Iw =	1	Table 1.5-1
Mean Building Height	H =	32.00	ft
Width Parallel to Wind	L =	64.58	ft
Width Normal to Wind	B =	55.33	ft
	L/B_X =	**1.17**	
	L/B_Y =	**1.17**	

PRINT WINDSPEED FROM ONLINE DATABASE TO CALC FOLDER

CONSERVATIVE, PER
RECOMMENDATION DURING
PRE-APPLICATION MEETING

	Kz=	**0.71**	(calculated, see table 27.3-1)
PRESSURE AT MEAN ROOF	q_h =	**28.34**	psf - ULTIMATE

GC_{pi}	**0.18**		
Cp - WW	**0.8**		
Cp - LW	**-0.46**	Worst Case	-0.48
Cp - SW	**-0.7**		

Description	**Floor**	Story H	**H**	**K_z**	**q_z (psf)**	**ULTIMATE P net (psf)**	**Story Wind Force, Fx kips**	**Story Shear kips**
Basement	1	0	0.00	0.57	22.8	**26.6**	7.5	47.0
Ground	2	10.23	10.23	0.57	22.8	**26.6**	15.1	39.4
Level 01	3	10.23	20.46	0.63	24.9	28.0	15.9	24.4
High Roof	4	10.23	30.69	0.71	28.0	30.1	8.5	8.5

Sec. 26.12	4	Enclosure	
	1	Select	N/A
	2	Open	0
	3	Partially	0.55
	4	Enclosed	0.18

Sec.26.7.3	2	Exp Cat
	1	A
	2	B
	3	C
	4	D

	L/B			
	Surface	1.2	Cp	
Fig 27.3-1	Windward Wall		0.8	qz
	Leeward Wall		-0.46	qh
	L/B 0-1	1	-0.5	
	L/B 2	2	-0.3	
	L/B >= 4	4	-0.2	
	Side Wall		-0.7	qh

L/B			
Surface	1.17	Cp	
Windward Wall		0.8	qz
Leeward Wall		-0.48	qh
L/B 0-1	1	-0.5	
L/B 2	2	-0.3	
L/B >= 4	4	-0.2	
Side Wall		-0.7	qh

Table	Calculate Kz									
26.9-1	a	z_g (ft)	a	b	a-bar	b-bar	c	l	psislon b	z_{min} (ft)
Exp A	5	1500	0.2	0.64	0.33333	0.3	0.45	180	0.5	60
Exp B	7	1200	0.1429	0.84	0.25	0.45	0.3	320	0.333	30
Exp C	9.5	900	0.1053	1	0.15385	0.65	0.2	500	0.2	15
Exp D	11.5	700	0.087	1.07	0.11111	0.8	0.15	600	0.125	7
calc->	7	1200	0.1429	0.84	0.25	0.45	0.3	320	0.333	30

Fig. 27.3-8 CASE 1, All Heights

	Kz	qz	Wward	Lward	Swall	Net	Wward	Lward	Swall	Net	Governs
H	Use	(psf)	Gcpi (+)			Pos	Gcpi (-)			Neg	
0	0.57	22.82	10.42	-16.18	-21.96	26.60	20.62	-5.98	-11.76	26.60	26.60
10.229	0.57	22.82	10.42	-16.18	-21.96	26.60	20.62	-5.98	-11.76	26.60	26.60
20.458	0.63	24.94	11.86	-16.18	-21.96	28.04	22.06	-5.98	-11.76	28.04	28.04
30.688	0.71	28.00	13.94	-16.18	-21.96	30.12	24.14	-5.98	-11.76	30.12	30.12

Fast + Epp

INPUT
RESULT

PROJECT: **Yaroslavsky Residence** **PROJECT NUMBER:** **8119**
SUBJECT: **MWFRS Total Building Wind Load** **DATE:** **2021-03-02**
DESIGN BY: **BJW**

NOTES: Y-DIRECTION

ASCE 7-16- WIND LOADS (ALL HEIGHTS)

Exposure Category		**B**	Sec. 26.7.3	
Basic Wind Speed	**V_ult=**	98	mph	PRINT WINDSPEED FROM ONLINE DATABASE TO CALC FOLDER
Directionality Factor	K_d =	0.85	Table 26.6-1	
Topographic Effects*	K_{zt} =	1.9	Fig. 26.8-1	CONSERVATIVE, PER RECOMMENDATION DURING PRE-APPLICATION MEETING
Gust Effect Factor	G_{used} =	0.85	Sec. 26.11	
Enclosure Type		Enclosed	Sec. 26.12	
Importance Factor	Iw =	1	Table 1.5-1	
Mean Building Height	H =	32.00	ft	
Width Parallel to Wind	L =	55.33	ft	
Width Normal to Wind	B =	64.58	ft	
	L/B_X =	**0.86**		
	L/B_Y =	**0.86**		
	Kz=	**0.71**	(calculated, see table 27.3-1)	
PRESSURE AT MEAN ROOF	q_h =	**28.34**	psf - ULTIMATE	

GC_{pi}	**0.18**		
Cp - WW	**0.8**		
Cp - LW	**-0.5**	Worst Case	-0.5
Cp - SW	**-0.7**		

Description	**Floor**	Story H	**H**	**K_z**	**$q_{z\ (psf)}$**	**ULTIMATE P net (psf)**	**Story Wind Force, Fx kips**	**Story Shear kips**
Basement	1	0	0.00	0.57	22.8	**27.6**	9.1	56.7
Ground	2	10.23	10.23	0.57	22.8	**27.6**	18.2	47.6
Level 01	3	10.23	20.46	0.63	24.9	29.0	19.2	29.4
High Roof	4	10.23	30.69	0.71	28.0	31.1	10.3	10.3

Sec. 26.12	4	Enclosure	
	1	Select	N/A
	2	Open	0
	3	Partially	0.55
	4	Enclosed	0.18

Sec.26.7.3	2	Exp Cat
	1	A
	2	B
	3	C
	4	D

	L/B			
	Surface	**0.9**	**Cp**	
Fig 27.3-1	Windward Wall		0.8	qz
	Leeward Wall		**-0.5**	qh
	L/B 0-1	1	-0.5	
	L/B 2	2	-0.3	
	L/B >= 4	4	-0.2	
	Side Wall		-0.7	qh

L/B			
Surface	**0.86**	**Cp**	
Windward Wall		0.8	qz
Leeward Wall		**-0.5**	qh
L/B 0-1	1	-0.5	
L/B 2	2	-0.3	
L/B >= 4	4	-0.2	
Side Wall		-0.7	qh

Table 26.9-1 **Calculate Kz**

	a	z_g (ft)	a	b	a-bar	b-bar	c	l	pislon b	z_{min} (ft)
Exp A	5	1500	0.2	0.64	0.33333	0.3	0.45	180	0.5	60
Exp B	7	1200	0.1429	0.84	0.25	0.45	0.3	320	0.333	30
Exp C	9.5	900	0.1053	1	0.15385	0.65	0.2	500	0.2	15
Exp D	11.5	700	0.087	1.07	0.11111	0.8	0.15	600	0.125	7
calc->	7	1200	0.1429	0.84	0.25	0.45	0.3	320	0.333	30

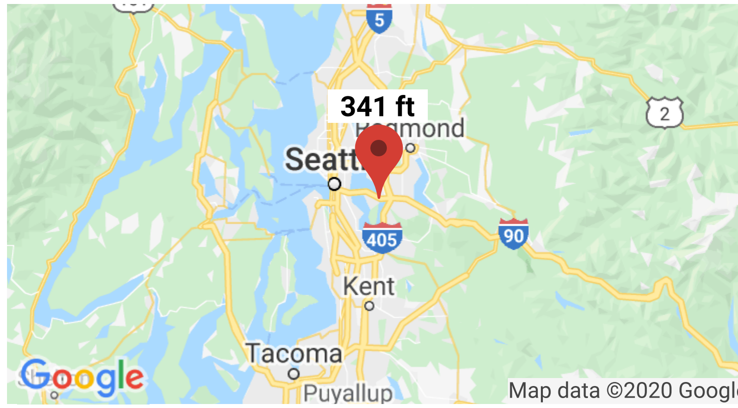
Fig. 27.3-8 **CASE 1, All Heights**

	Kz	qz	Wward	Lward	Swall	Net	Wward	Lward	Swall	Net	Governs
H	Use	(psf)	Gcpi (+)			Pos	Gcpi (-)			Neg	
0	0.57	22.82	10.42	-17.14	-21.96	27.56	20.62	-6.94	-11.76	27.56	27.56
10.229	0.57	22.82	10.42	-17.14	-21.96	27.56	20.62	-6.94	-11.76	27.56	27.56
20.458	0.63	24.94	11.86	-17.14	-21.96	29.00	22.06	-6.94	-11.76	29.00	29.00
30.688	0.71	28.00	13.94	-17.14	-21.96	31.08	24.14	-6.94	-11.76	31.08	31.08
			#VALUE!	-17.14	-21.96	#VALUE!	######	-6.94	-11.76	######	######
			#VALUE!	-17.14	-21.96	#VALUE!	######	-6.94	-11.76	######	######
			#VALUE!	-17.14	-21.96	#VALUE!	######	-6.94	-11.76	######	######
			#VALUE!	-17.14	-21.96	#VALUE!	######	-6.94	-11.76	######	######

1.6 | SEISMIC LOADS

ATC Hazards by Location

Search Information



Address: 9319 SE 43rd St, Mercer Island, WA 98040, USA

Coordinates: 47.5693472, -122.2142869

Elevation: 341 ft

Timestamp: 2020-10-30T18:37:39.948Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D

Basic Parameters

Name	Value	Description
S_S	1.415	MCE_R ground motion (period=0.2s)
S_1	0.492	MCE_R ground motion (period=1.0s)
S_{MS}	1.415	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.944	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.902	Coefficient of risk (0.2s)
CR_1	0.898	Coefficient of risk (1.0s)
PGA	0.606	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.666	Site modified peak ground acceleration

T_L	6	Long-period transition period (s)
SsRT	1.415	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.568	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	3.753	Factored deterministic acceleration value (0.2s)
S1RT	0.492	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.548	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.487	Factored deterministic acceleration value (1.0s)
PGAd	1.272	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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SEISMIC FORCES (ASCE 7-16)

Tedds calculation version 3.1.01

Site parameters

Site class — D, utilizing exception per 11.4.8(2)

Mapped acceleration parameters (Section 11.4.2)

at short period — S_S = **1.415**

at 1 sec period — S_1 = **0.492**

Site coefficientat short period (Table 11.4-1) — F_a = **1.000**

at 1 sec period (Table 11.4-2) — F_v = **1.808**

Spectral response acceleration parameters

at short period (Eq. 11.4-1) — $S_{MS} = F_a * S_S$ = **1.415**

at 1 sec period (Eq. 11.4-2) — $S_{M1} = F_v * S_1$ = **0.890**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3) — $S_{DS} = 2 / 3 * S_{MS}$ = **0.943**

at 1 sec period (Eq. 11.4-4) — $S_{D1} = 2 / 3 * S_{M1}$ = **0.593**

Seismic design category

Occupancy category (Table 1-1) — II

Seismic design category based on short period response acceleration (Table 11.6-1)

D

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

D

Seismic design category — D

Approximate fundamental period

Height above base to highest level of building — h_n = **30.69** ft

From Table 12.8-2:

Structure type — All other systems

Building period parameter C_t — C_t = **0.02**

Building period parameter x — x = **0.75**

Approximate fundamental period (Eq 12.8-7) — $T_a = C_t * (h_n)^x * 1sec / (1ft)^x$ = **0.261** sec

Building fundamental period (Sect 12.8.2) — $T = T_a$ = **0.261** sec

Long-period transition period — T_L = **6** sec

Limiting period — $T_S = S_{D1} / S_{DS} * 1$ sec = **0.629** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1) — A. Bearing_Wall_Systems

15. Light-frame (wood) walls sheathed with wood structural panels

Response modification factor (Table 12.2-1) — R = **6.5**

Seismic importance factor (Table 1.5-2) — I_e = **1.000**

Seismic response coefficient (Sect 11.4.8)

Calculated (Eq 12.8-3) — $C_{s\_calc} = S_{DS} / (R / I_e)$ = **0.1451**

Minimum (Eq 9.5.5.2.1-3) — $C_{s\_min} = \max(0.044 * S_{DS} * I_e, 0.01)$ = **0.0415**

Seismic response coefficient — C_s = **0.1451**

Tekla Tedds Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsky Residence			Job Ref. 8119	
	Section 1.6 Seismic Loads - LFW (X-Direction)			Sheet no./rev. 2	
	Calc. by BW	Date 2/23/2021	Chk'd by	Date	App'd by	Date

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure W = **147.0** kips

Seismic response coefficient C_s = **0.1451**

Seismic base shear (Eq 12.8-1) V = C_s * W = **21.3** kips → CONSERVATIVELY USE HIGHER WIND BASE SHEAR AND DESIGN PER SEISMIC PROVISIONS

Vertical distribution of seismic forces (Sect 12.8.3)

Vertical distribution factor (Eq 12.8-12) $C_{vx} = w_x * h_x^k / \Sigma(w_i * h_i^k)$ V = 47 kips

Lateral force induced at level i (Eq 12.8-11) $F_x = C_{vx} * V$

Minimum diaphragm forces (Section 12.10.1.1)

Calculated min. diaphragm force (Eq 12.10-1) $F_{px} = \Sigma F_i * w_{px} / \Sigma w_i$,(i=x to n)

$F_{pxmin} = 0.2 * S_{DS} * I_e * w_{px}$

$F_{pxmax} = 0.4 * S_{DS} * I_e * w_{px}$

Vertical force distribution table

Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips), w_x	Distribution exponent related to building period, k	Vertical distribution factor, C_{vx}	Lateral force induced at Level i (kips), F_x	Weight tributary to the diaphragm at Level i (kips), w_{px}	Minimum diaphragm force at Level i (kips), F_{px}
1	10.2;	58.0;	1.00;	0.231;	~~4.9~~ 13	58.0	~~10.9~~ 13
2	20.5;	74.0;	1.00;	0.590;	~~12.6~~ 26.1	74.0	~~14.0~~ 26.1
3	30.7;	15.0;	1.00;	0.179;	~~3.8~~ 8	15.0	~~3.8~~ 8

SEISMIC FORCES (ASCE 7-16)

Tedds calculation version 3.1.01

Site parameters

Site class	D, utilizing exception per 11.4.8(2)
Mapped acceleration parameters (Section 11.4.2)	
at short period	S_S = **1.415**
at 1 sec period	S_1 = **0.492**
Site coefficientat short period (Table 11.4-1)	F_a = **1.000**
at 1 sec period (Table 11.4-2)	F_v = **1.808**

Spectral response acceleration parameters

at short period (Eq. 11.4-1)	$S_{MS} = F_a * S_S$ = **1.415**
at 1 sec period (Eq. 11.4-2)	$S_{M1} = F_v * S_1$ = **0.890**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3)	$S_{DS} = 2 / 3 * S_{MS}$ = **0.943**
at 1 sec period (Eq. 11.4-4)	$S_{D1} = 2 / 3 * S_{M1}$ = **0.593**

Seismic design category

Occupancy category (Table 1-1) II

Seismic design category based on short period response acceleration (Table 11.6-1)
D

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)
D

Seismic design category D

Approximate fundamental period

Height above base to highest level of building h_n = **30.69** ft

From Table 12.8-2:

Structure type	All other systems
Building period parameter C_t	C_t = **0.02**
Building period parameter x	x = **0.75**

Approximate fundamental period (Eq 12.8-7)	$T_a = C_t * (h_n)^x * 1sec / (1ft)^x$= **0.261** sec
Building fundamental period (Sect 12.8.2)	$T = T_a$ = **0.261** sec
Long-period transition period	T_L = **6** sec
Limiting period	$T_S = S_{D1} / S_{DS} * 1$ sec = **0.629** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1)	C_MOMENT_RESISTING_FRAME_SYSTEMS 4. Steel ordinary moment frames
Response modification factor (Table 12.2-1)	R = **3.5**
Seismic importance factor (Table 1.5-2)	I_e = **1.000**
Seismic response coefficient (Sect 11.4.8)	
Calculated (Eq 12.8-3)	$Cs_{\_calc} = S_{DS} / (R / I_e)$ = **0.2695**
Minimum (Eq 9.5.5.2.1-3)	$Cs_{\_min} = \max(0.044 * S_{DS} * I_e, 0.01)$ = **0.0415**
Seismic response coefficient	C_s = **0.2695**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure — W = **143.0** kips

Seismic response coefficient — C_s = **0.2695**

Seismic base shear (Eq 12.8-1) — V = C_s * W = **38.5** kips → CONSERVATIVELY USE HIGHER WIND BASE SHEAR AND DESIGN PER SEISMIC PROVISIONS

Vertical distribution of seismic forces (Sect 12.8.3)

Vertical distribution factor (Eq 12.8-12) — $C_{vx} = w_x * h_x^k / \Sigma(w_i * h_i^k)$ — V = 56.7 kips

Lateral force induced at level i (Eq 12.8-11) — $F_x = C_{vx} * V$

Minimum diaphragm forces (Section 12.10.1.1)

Calculated min. diaphragm force (Eq 12.10-1) — $F_{px} = \Sigma F_i * w_{px} / \Sigma w_i$,(i=x to n)

$F_{pxmin} = 0.2 * S_{DS} * I_e * w_{px}$

$F_{pxmax} = 0.4 * S_{DS} * I_e * w_{px}$

Vertical force distribution table

Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips), w_x	Distribution exponent related to building period, k	Vertical distribution factor, C_{vx}	Lateral force induced at Level i (kips), F_x	Weight tributary to the diaphragm at Level i (kips), w_{px}	Minimum diaphragm force at Level i (kips), F_{px}
1	10.2;	58.0;	1.00;	0.239;	~~9.2~~ 15.6	58.0	~~15.6~~ 15.6
2	20.5;	70.0;	1.00;	0.576;	~~22.2~~ 31.6	70.0	~~24.2~~ 31.6
3	30.7;	15.0;	1.00;	0.185;	~~7.1~~ 9.6	15.0	~~5.7~~ 9.6

2 | GRAVITY DESIGN

2.1 | WOOD FRAMING DESIGN

High Roof				
Member Name	Results	Current Solution	Comments	
J9 Roof: Joist (11 7/8" TJI)	Passed	1 piece(s) 11 7/8" TJI® 360 @ 16" OC		
B15 High Roof: Beam (PSL)	Passed	1 piece(s) 3 1/2" x 11 7/8" 2.2E Parallam® PSL		
Garage Roof				
Member Name	Results	Current Solution	Comments	
J9 Roof: Joist (11 7/8" TJI)	Passed	1 piece(s) 11 7/8" TJI® 360 @ 16" OC		
B13 Garage Roof: Edge Beam (LVL)	Passed	2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL		
Main Level				
Member Name	Results	Current Solution	Comments	
J1 Kitchen: Joist (16" TJI)	Passed	1 piece(s) 16" TJI® 210 @ 16" OC		
J1 Family Room: Joist (16" TJI)	Passed	1 piece(s) 16" TJI® 210 @ 16" OC		
J2 Living Room: (16" TJI)	Passed	1 piece(s) 16" TJI® 230 @ 16" OC		
J3 Exterior Deck: Joist (9.5" LVL)	Passed	1 piece(s) 1 3/4" x 9 1/2" 2.0E Microllam® LVL @ 16" OC		
J4 Exterior Deck Short: Joist (2x10)	Passed	1 piece(s) 2 x 10 Douglas Fir-Larch No. 1 @ 16" OC		
J8 Main Level Shower: Joist (11-7/8" TJI)	Passed	1 piece(s) 11 7/8" TJI® 110 @ 16" OC		
B1 Kitchen: Flush Beam 1	Passed	1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL		
B1 Kitchen: Flush Beam 2	Passed	1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL		
B1 Kitchen: Flush Beam 3	Passed	1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL		
B1 Dining Room: Flush Beam	Passed	1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL		
B1 Main Level Shower: Flush Beam (16" PSL)	Passed	1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL		
B1 Main Level: Transfer Beam 1 (16" PSL)	Passed	1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL		
B1 Main Level: Transfer Beam 2 (16" PSL)	Passed	1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL		
B4 Exterior Deck: South Flush Beam (14" PSL)	Passed	1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL		
B5 Main Level: Wall Transfer Beam 1	Failed	1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL	Multiple Failures/Errors	SEE NOTES IN CALCULATIONS
B5 Main Level: Wall Transfer Beam 2	Failed	1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL	Multiple Failures/Errors	SEE NOTES IN CALCULATIONS
B6 Exterior Deck: Flush Beam	Passed	1 piece(s) 3 1/2" x 9 1/2" 2.2E Parallam® PSL		
B6 Exterior Deck: Flush Beam (East)	Passed	1 piece(s) 3 1/2" x 9 1/2" 2.2E Parallam® PSL		
B6 Kitchen: Transfer Beam	Failed	1 piece(s) 5 1/4" x 9 1/2" 2.2E Parallam® PSL	Multiple Failures/Errors	SEE NOTES IN CALCULATIONS
B12 Family Room: Wall Transfer Beam 2	Failed	1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL	Multiple Failures/Errors	SEE NOTES IN CALCULATIONS
B18 Family Room: Wall Transfer Beam 1	Failed	1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL	Multiple Failures/Errors	SEE NOTES IN CALCULATIONS
B19 Family Room: Transfer Beam 3	Failed	1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL	Multiple Failures/Errors	SEE NOTES IN CALCULATIONS
B19 Family Room: Transfer Beam 4	Failed	1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL	Multiple Failures/Errors	SEE NOTES IN CALCULATIONS
B20 Living Room: Drop Beam	Passed	1 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL		
C7 Post Transfer	Passed	1 piece(s) 5 1/4" x 7" 1.8E Parallam® PSL		

ForteWEB Software Operator	Job Notes
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Upper Level			
Member Name	Results	Current Solution	Comments
J5 Upper Level: Joist (14" TJI)	Passed	1 piece(s) 14" TJI® 360 @ 12" OC	
J6 Upper Deck: Joist - Long (11-7/8" TJI)	Passed	1 piece(s) 11 7/8" TJI® 560 @ 12" OC	
J8 Upper Deck: Joist - Short (11-7/8" TJI)	Passed	1 piece(s) 11 7/8" TJI® 110 @ 16" OC	
J8 Stair Roof: Joist (11-7/8" TJI)	Passed	1 piece(s) 11 7/8" TJI® 110 @ 16" OC	
J9 Upper Deck: Joist - Med (11-7/8" TJI)	Passed	1 piece(s) 11 7/8" TJI® 230 @ 12" OC	
J10 Upper Level Shower: Joist (9-1/2" TJI)	Passed	1 piece(s) 9 1/2" TJI® 110 @ 16" OC	
J11 Upper Deck: Joist (11-7/8" TJI)	Passed	1 piece(s) 11 7/8" TJI® 110 @ 24" OC	
B4 Upper Level Shower: Short Flush Beam (14" PSL)	Passed	1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL	
B4 Upper Level Shower: Long Flush Beam (14" PSL)	Passed	1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL	
B4 Upper Level: Flush Beam (14" PSL)	Passed	1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL	
B4 Upper Level: Transfer Beam 4 (14" PSL)	Passed	1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL	
B7 Upper Level: Typical Header Beam (2-2x10)	Passed	2 piece(s) 2 x 10 Spruce-Pine-Fir No. 1 / No. 2	
B12 Upper Level: Flush Beam (14" PSL)	Passed	1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL	
B12 Upper Level: Transfer Beam (14" PSL)	Failed	1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL	An excessive uplift of -2553 lbs at support located at 3 1/2" failed this product.
B12 Upper Level: Transfer Beam 2 (14" PSL)	Failed	1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL	Multiple Failures/Errors
B12 Upper Level: Transfer Beam 3 (14" PSL)	Failed	1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL	Multiple Failures/Errors
B13 Upper Deck: Edge Beam (11-7/8" LVL)	Passed	2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL	
B13 Upper Deck: Flush Beam (11-7/8" LVL)	Passed	2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL	
B13 Upper Deck: Edge Beam 2 (11-7/8" LVL)	Passed	2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL	
B13 Upper Deck: Edge Beam 3 (11-7/8" LVL)	Passed	2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL	
B14 Upper Deck: Long Flush Beam (11-7/8" PSL)	Passed	1 piece(s) 7" x 11 7/8" 2.2E Parallam® PSL	
B15 Upper Deck: Short Flush Beam (11-7/8" PSL)	Passed	1 piece(s) 3 1/2" x 11 7/8" 2.2E Parallam® PSL	

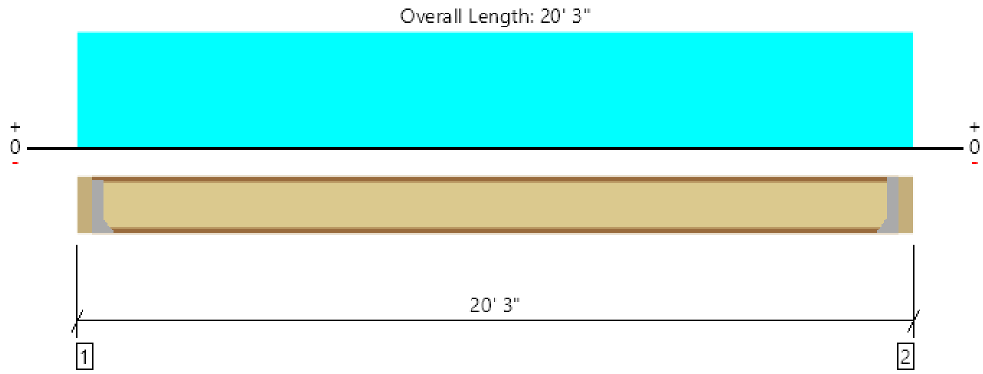
SEE NOTES IN CALCULATIONS

SEE NOTES IN CALCULATIONS

SEE NOTES IN CALCULATIONS

High Roof, J9 Roof: Joist (11 7/8" TJI)

1 piece(s) 11 7/8" TJI® 360 @ 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)	590 @ 3 1/2"	1242 (1.75")	Passed (48%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	590 @ 3 1/2"	1961	Passed (30%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	2901 @ 10' 1 1/2"	7107	Passed (41%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.356 @ 10' 1 1/2"	0.656	Passed (L/663)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.534 @ 10' 1 1/2"	0.983	Passed (L/442)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Total	
1 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	202	270	405	877	See note [1]
2 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	202	270	405	877	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)	5' 7" o/c	
Bottom Edge (Lu)	19' 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	IUS2.37/11.88	2.00"	N/A	10-10dx1.5	2-Strong-Grip	
2 - Face Mount Hanger	IUS2.37/11.88	2.00"	N/A	10-10dx1.5	2-Strong-Grip	

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

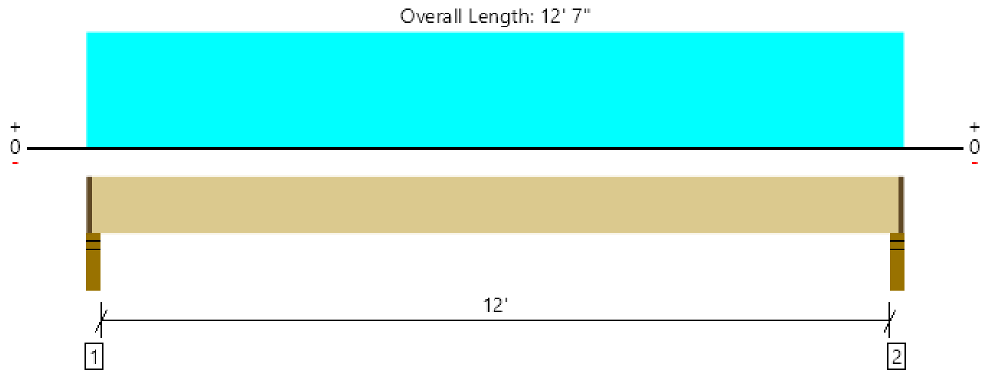
Vertical Load	Location (Side)	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 20' 3"	16"	15.0	20.0	30.0	Default Load

Weyerhaeuser Notes

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

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

High Roof, B15 High Roof: Beam (PSL)
1 piece(s) 3 1/2" x 11 7/8" 2.2E Parallam® PSL



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)	1926 @ 2"	3347 (2.25")	Passed (58%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	1560 @ 1' 3 3/8"	9241	Passed (17%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	5839 @ 6' 3 1/2"	22888	Passed (26%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.101 @ 6' 3 1/2"	0.306	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.161 @ 6' 3 1/2"	0.613	Passed (L/910)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.50"	733	653	979	2365	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.50"	733	653	979	2365	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)	12' 5" o/c	
Bottom Edge (Lu)	12' 5" 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 12' 5 3/4"	N/A	13.0	--	--	
1 - Uniform (PSF)	0 to 12' 7" (Front)	5' 2 1/4"	20.0	20.0	30.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
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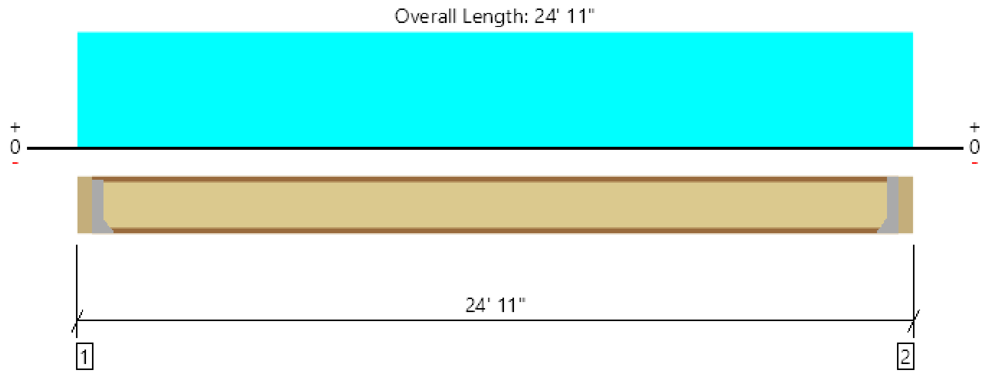
Garage Roof, J9 Roof: Joist (11 7/8" TJI)

1 piece(s) 11 7/8" TJI® 360 @ 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)	730 @ 3 1/2"	1242 (1.75")	Passed (59%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	730 @ 3 1/2"	1961	Passed (37%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	4441 @ 12' 5 1/2"	7107	Passed (62%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.806 @ 12' 5 1/2"	0.811	Passed (L/362)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	1.209 @ 12' 5 1/2"	1.217	Passed (L/241)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Total	
1 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	249	332	498	1079	See note [1]
2 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	249	332	498	1079	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' 5" o/c	
Bottom Edge (Lu)	24' 4" 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	IUS2.37/11.88	2.00"	N/A	10-10dx1.5	2-Strong-Grip	
2 - Face Mount Hanger	IUS2.37/11.88	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)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 24' 11"	16"	15.0	20.0	30.0	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
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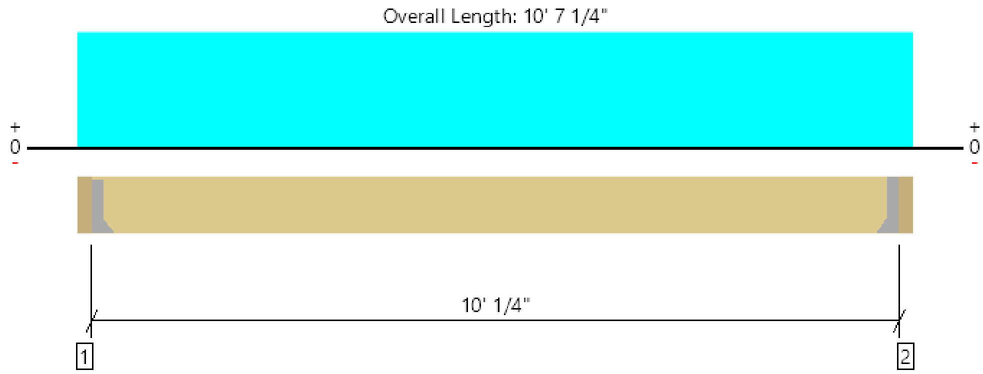
3/2/2021 11:50:11 PM UTC

ForteWEB v3.1, Engine: V8.1.6.2, Data: V8.0.1.0

File Name: 20210225 Permit Set

Garage Roof, B13 Garage Roof: Edge Beam (LVL)
2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL



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)	2748 @ 3 1/2"	3938 (1.50")	Passed (70%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	2205 @ 1' 3 3/8"	9081	Passed (24%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	6883 @ 5' 3 5/8"	20525	Passed (34%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.099 @ 5' 3 5/8"	0.251	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.146 @ 5' 3 5/8"	0.501	Passed (L/821)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 11 7/8" SPF beam	3.50"	Hanger[1]	1.50"	936	1166	1458	3560	See note [1]
2 - Hanger on 11 7/8" SPF beam	3.50"	Hanger[1]	1.50"	936	1166	1458	3560	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)	10' o/c	
Bottom Edge (Lu)	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
1 - Face Mount Hanger	HHUS48	3.00"	N/A	22-10d	8-10d	
2 - Face Mount Hanger	HHUS48	3.00"	N/A	22-10d	8-10d	

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

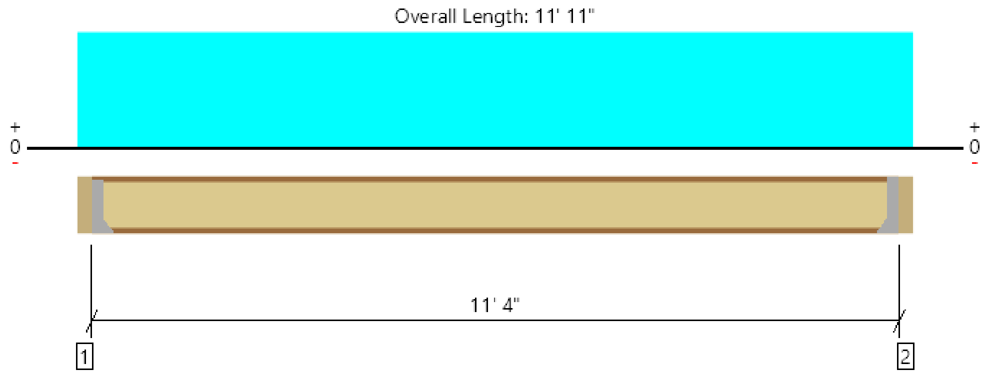
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	3 1/2" to 10' 3 3/4"	N/A	12.1	--	--	
1 - Uniform (PSF)	0 to 10' 7 1/4" (Front)	11'	15.0	20.0	25.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Main Level, J1 Kitchen: Joist (16" TJI)

1 piece(s) 16" TJI® 210 @ 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)	529 @ 3 1/2"	1005 (1.75")	Passed (53%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	529 @ 3 1/2"	2190	Passed (24%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1499 @ 5' 11 1/2"	5140	Passed (29%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.037 @ 5' 11 1/2"	0.283	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.066 @ 5' 11 1/2"	0.567	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	64	50	Passed	--	--

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 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: bridging or blocking at max. 8' o.c..

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 16" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	238	318	556	See note [1]
2 - Hanger on 16" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	238	318	556	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)	7' 2" o/c	
Bottom Edge (Lu)	11' 4" 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	IUS2.06/16	2.00"	N/A	14-10dx1.5	2-Strong-Grip	
2 - Face Mount Hanger	IUS2.06/16	2.00"	N/A	14-10dx1.5	2-Strong-Grip	

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

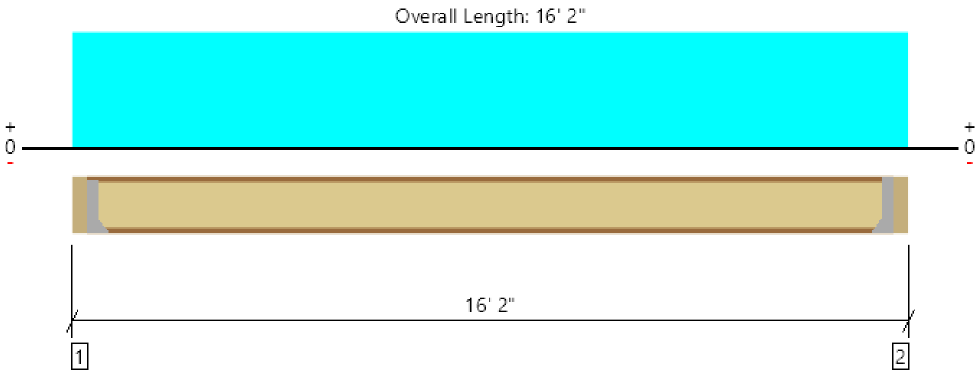
Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 11' 11"	16"	30.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Weyerhaeuser

Main Level, J1 Family Room: Joist (16" TJI)
1 piece(s) 16" TJI® 210 @ 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)	727 @ 3 1/2"	1005 (1.75")	Passed (72%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	727 @ 3 1/2"	2190	Passed (33%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2833 @ 8' 1"	5140	Passed (55%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.115 @ 8' 1"	0.390	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.201 @ 8' 1"	0.779	Passed (L/931)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	55	50	Passed	--	--

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 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: bridging or blocking at max. 8' o.c..

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 16" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	323	431	754	See note [1]
2 - Hanger on 16" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	323	431	754	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)	5' 2" o/c	
Bottom Edge (Lu)	15' 7" 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	IUS2.06/16	2.00"	N/A	14-10dx1.5	2-Strong-Grip	
2 - Face Mount Hanger	IUS2.06/16	2.00"	N/A	14-10dx1.5	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)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 16' 2"	16"	30.0	40.0	Default Load

Weyerhaeuser Notes

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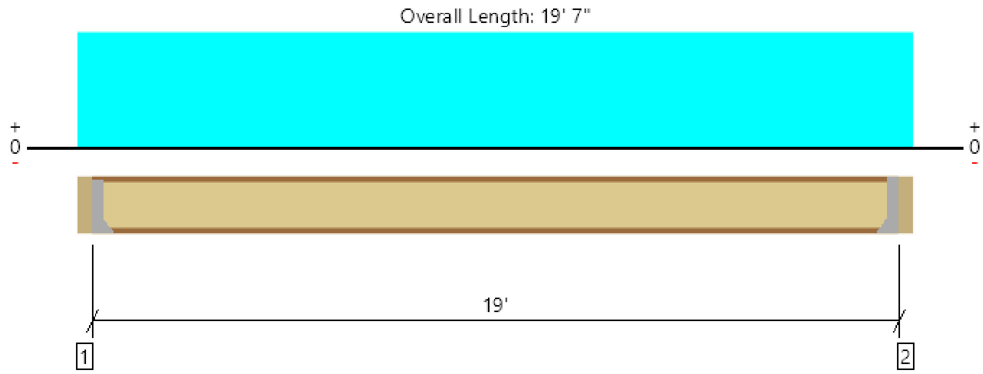
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Weyerhaeuser

Main Level, J2 Living Room: (16" TJI)

1 piece(s) 16" TJI® 230 @ 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)	887 @ 3 1/2"	1060 (1.75")	Passed (84%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	887 @ 3 1/2"	2190	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	4212 @ 9' 9 1/2"	5710	Passed (74%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.236 @ 9' 9 1/2"	0.475	Passed (L/964)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.414 @ 9' 9 1/2"	0.950	Passed (L/551)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	50	45	Passed	--	--

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 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 nailed down.
- Additional considerations for the TJ-Pro™ Rating include: bridging or blocking at max. 8' o.c..

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
1 - Hanger on 16" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	392	522	914	See note [1]
2 - Hanger on 16" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	392	522	914	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' 10" o/c	
Bottom Edge (Lu)	19' 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	IUS2.37/16	2.00"	N/A	14-10dx1.5	2-Strong-Grip	
2 - Face Mount Hanger	Connector not found	N/A	N/A	N/A	N/A	

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

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 19' 7"	16"	30.0	40.0	

Weyerhaeuser Notes

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

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

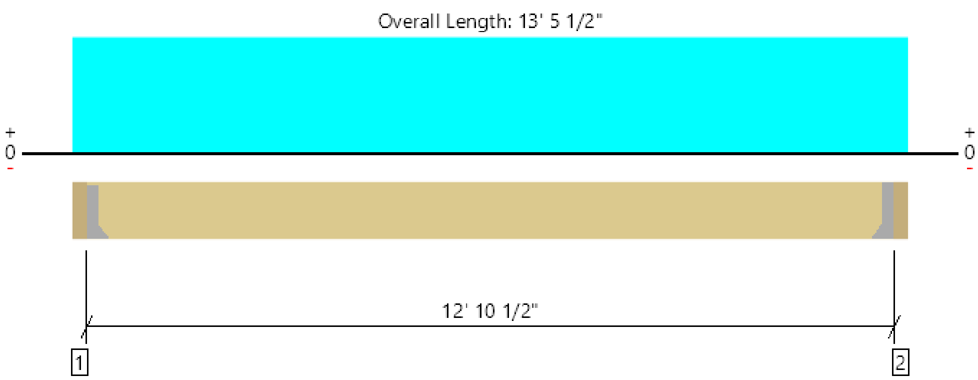
FORTE® WEB

MEMBER REPORT

PASSED

Main Level, J3 Exterior Deck: Joist (9.5" LVL)

1 piece(s) 1 3/4" x 9 1/2" 2.0E Microllam® LVL @ 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)	837 @ 3 1/2"	1969 (1.50")	Passed (43%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	678 @ 1' 1"	3159	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2486 @ 6' 8 3/4"	6123	Passed (41%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.209 @ 6' 8 3/4"	0.322	Passed (L/738)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.302 @ 6' 8 3/4"	0.644	Passed (L/511)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
TJ-Pro™ Rating	51	49	Passed	--	--

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: bridging or blocking at max. 8' o.c..

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 9 1/2" PSL beam	3.50"	Hanger[1]	1.50"	269	538	269	1076	See note [1]
2 - Hanger on 9 1/2" PSL beam	3.50"	Hanger[1]	1.50"	269	538	269	1076	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)	12' 11" o/c	
Bottom Edge (Lu)	12' 11" 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 - Face Mount Hanger	IUS1.81/9.5	2.00"	N/A	8-10dx1.5	2-10dx1.5	
2 - Face Mount Hanger	IUS1.81/9.5	2.00"	N/A	8-10dx1.5	2-10dx1.5	

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

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 13' 5 1/2"	16"	30.0	60.0	30.0	Default Load

Weyerhaeuser Notes

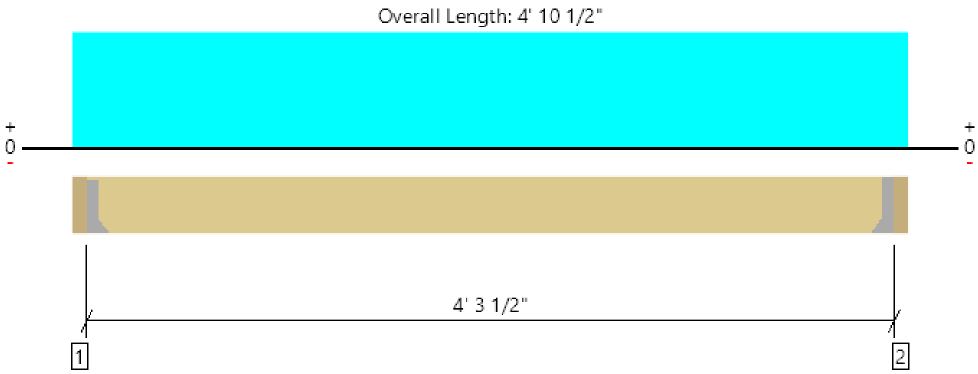
ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Weyerhaeuser

MEMBER REPORT

PASSED

Main Level, J4 Exterior Deck Short: Joist (2x10)
1 piece(s) 2 x 10 Douglas Fir-Larch No. 1 @ 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)	279 @ 3 1/2"	1406 (1.50")	Passed (20%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	165 @ 1' 3/4"	1665	Passed (10%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	276 @ 2' 5 1/4"	2255	Passed (12%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.004 @ 2' 5 1/4"	0.107	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.006 @ 2' 5 1/4"	0.215	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

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	Total	
1 - Hanger on 9 1/4" PSL beam	3.50"	Hanger¹	1.50"	97	195	97	389	See note ¹
2 - Hanger on 9 1/4" PSL beam	3.50"	Hanger¹	1.50"	97	195	97	389	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)	4' 4" o/c	
Bottom Edge (Lu)	4' 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 - Face Mount Hanger	LUS28	1.75"	N/A	6-10dx1.5	3-10d	
2 - Face Mount Hanger	LUS28	1.75"	N/A	6-10dx1.5	3-10d	

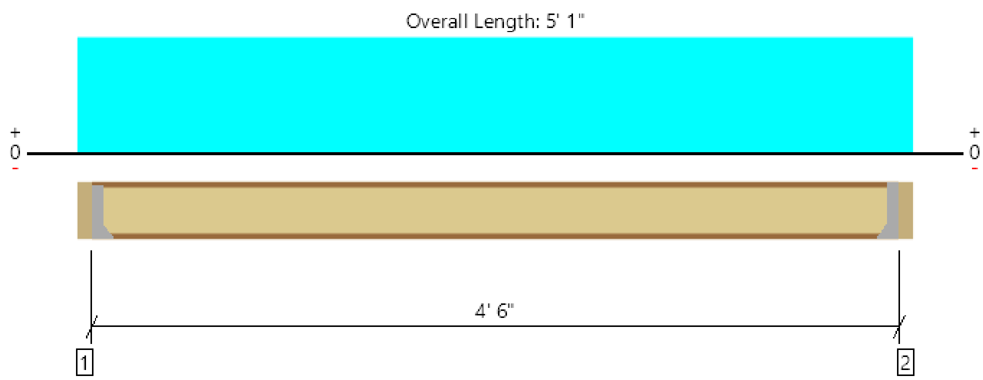
- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 4' 10 1/2"	16"	30.0	60.0	30.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Main Level, J8 Main Level Shower: Joist (11-7/8" TJI)
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)	210 @ 3 1/2"	910 (1.75")	Passed (23%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	210 @ 3 1/2"	1560	Passed (13%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	236 @ 2' 6 1/2"	3160	Passed (7%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.004 @ 2' 6 1/2"	0.112	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.007 @ 2' 6 1/2"	0.225	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	72	50	Passed	--	--

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 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: bridging or blocking at max. 8' o.c..

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	102	136	238	See note [1]
2 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	102	136	238	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' 6" o/c	
Bottom Edge (Lu)	4' 6" 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)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 5' 1"	16"	30.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

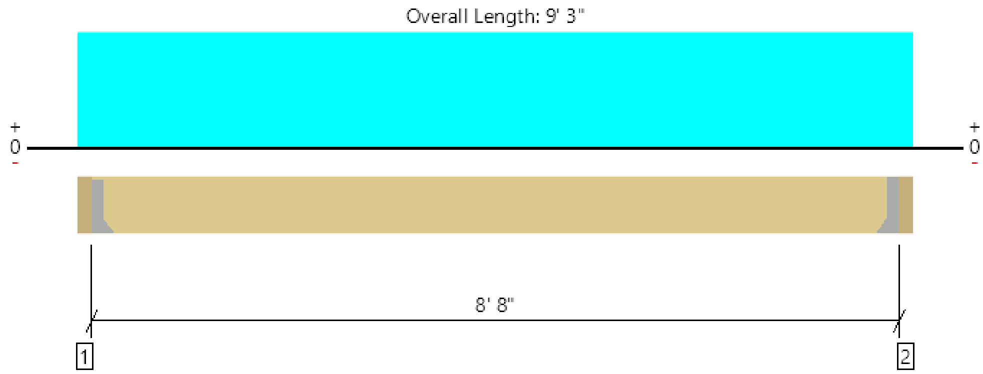
Weyerhaeuser

MEMBER REPORT

PASSED

Main Level, B1 Kitchen: Flush Beam 1

1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL



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)	1833 @ 3 1/2"	4922 (1.50")	Passed (37%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1269 @ 1' 7 1/2"	16240	Passed (8%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3971 @ 4' 7 1/2"	52432	Passed (8%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.010 @ 4' 7 1/2"	0.217	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.019 @ 4' 7 1/2"	0.433	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 16" SPF beam	3.50"	Hanger[1]	1.50"	900	1048	1948	See note [1]
2 - Hanger on 16" SPF beam	3.50"	Hanger[1]	1.50"	900	1048	1948	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)	8' 8" o/c	
Bottom Edge (Lu)	8' 8" 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 - Face Mount Hanger	MGU5.50-SDS H=15.938	4.50"	N/A	24-SDS25212	16-SDS25212	
2 - Face Mount Hanger	MGU5.50-SDS H=15.938	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)	Comments
0 - Self Weight (PLF)	3 1/2" to 8' 11 1/2"	N/A	26.3	--	
1 - Uniform (PSF)	0 to 9' 3" (Front)	5' 8"	30.0	40.0	Default Load

Weyerhaeuser Notes

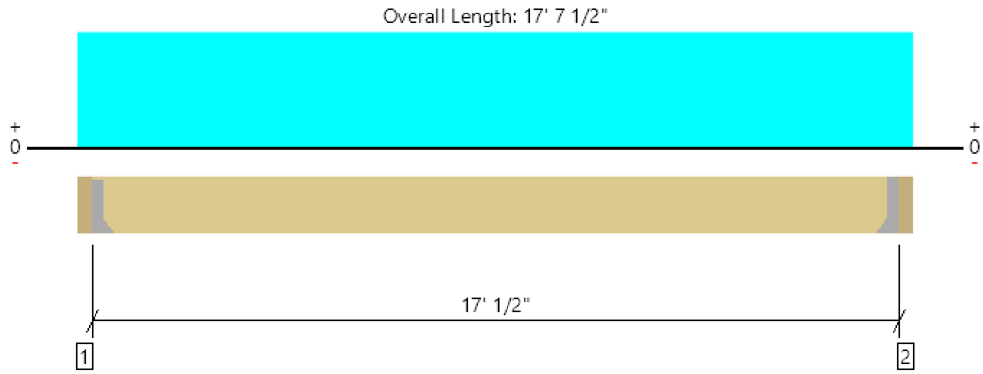
ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

MEMBER REPORT

PASSED

Main Level, B1 Kitchen: Flush Beam 2

1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL



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)	3604 @ 3 1/2"	4922 (1.50")	Passed (73%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	3040 @ 1' 7 1/2"	16240	Passed (19%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	15353 @ 8' 9 3/4"	52432	Passed (29%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.119 @ 8' 9 3/4"	0.426	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.223 @ 8' 9 3/4"	0.852	Passed (L/918)	--	1.0 D + 1.0 L (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 16" SPF beam	3.50"	Hanger[1]	1.50"	1722	1998	3720	See note [1]
2 - Hanger on 16" SPF beam	3.50"	Hanger[1]	1.50"	1722	1998	3720	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)	17' 1" o/c	
Bottom Edge (Lu)	17' 1" 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 - Face Mount Hanger	MGU5.50-SDS H=15.938	4.50"	N/A	24-SDS25212	16-SDS25212	
2 - Face Mount Hanger	MGU5.50-SDS H=15.938	4.50"	N/A	24-SDS25212	16-SDS25212	

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

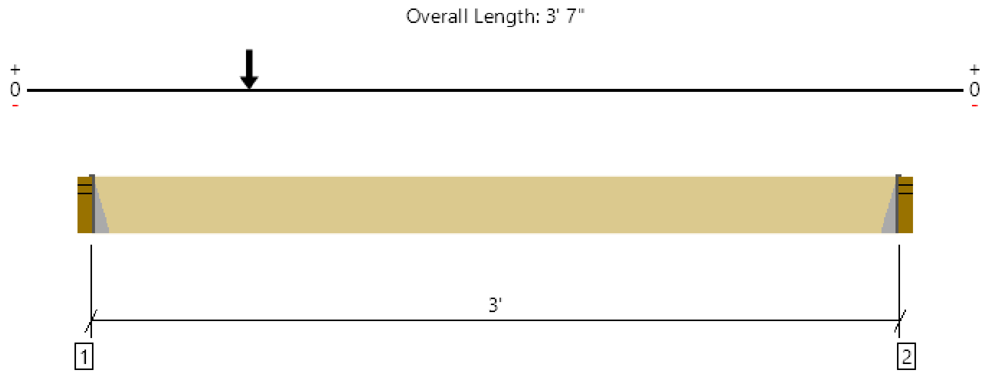
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	3 1/2" to 17' 4"	N/A	26.3	--	
1 - Uniform (PSF)	0 to 17' 7 1/2" (Front)	5' 8"	30.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Main Level, B1 Kitchen: Flush Beam 3

1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL



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)	4604 @ 3 1/2"	4922 (1.50")	Passed (94%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1382 @ 1' 7 1/2"	16240	Passed (9%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2681 @ 10 1/2"	52432	Passed (5%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.002 @ 10 1/2"	0.075	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.004 @ 10 1/2"	0.150	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on SPF studWall	3.50"	Hanger¹	1.50"	2152	2453	4605	See note ¹
2 - Hanger on SPF studWall	3.50"	Hanger¹	1.50"	549	592	1141	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)	3' o/c	
Bottom Edge (Lu)	3' 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 - Top Mount Hanger	HB5.50/16	3.50"	6-16d	16-16d	10-16d	

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

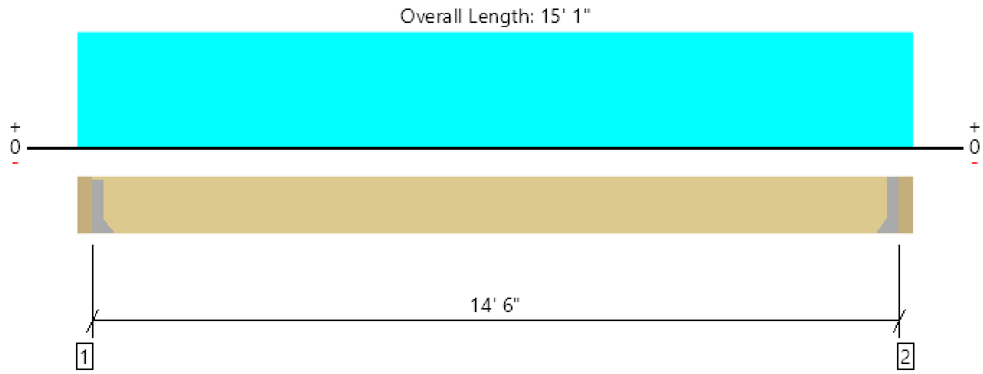
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	3 1/2" to 3' 3 1/2"	N/A	26.3	--	
1 - Point (lb)	10 1/2" (Front)	N/A	2622	3045	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Main Level, B1 Dining Room: Flush Beam

1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL



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)	5963 @ 3 1/2"	5963 (1.82")	Passed (100%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	4866 @ 1' 7 1/2"	16240	Passed (30%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	21616 @ 7' 6 1/2"	52432	Passed (41%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.130 @ 7' 6 1/2"	0.363	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.234 @ 7' 6 1/2"	0.725	Passed (L/742)	--	1.0 D + 1.0 L (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 16" SPF beam	3.50"	Hanger[1]	1.82"	2764	3431	6195	See note [1]
2 - Hanger on 16" SPF beam	3.50"	Hanger[1]	1.82"	2764	3431	6195	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)	14' 6" o/c	
Bottom Edge (Lu)	14' 6" 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 - Face Mount Hanger	MGU5.50-SDS H=15.938	4.50"	N/A	24-SDS25212	16-SDS25212	
2 - Face Mount Hanger	MGU5.50-SDS H=15.938	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)	Comments
0 - Self Weight (PLF)	3 1/2" to 14' 9 1/2"	N/A	26.3	--	
1 - Uniform (PSF)	0 to 15' 1" (Front)	11' 4 1/2"	30.0	40.0	Default Load

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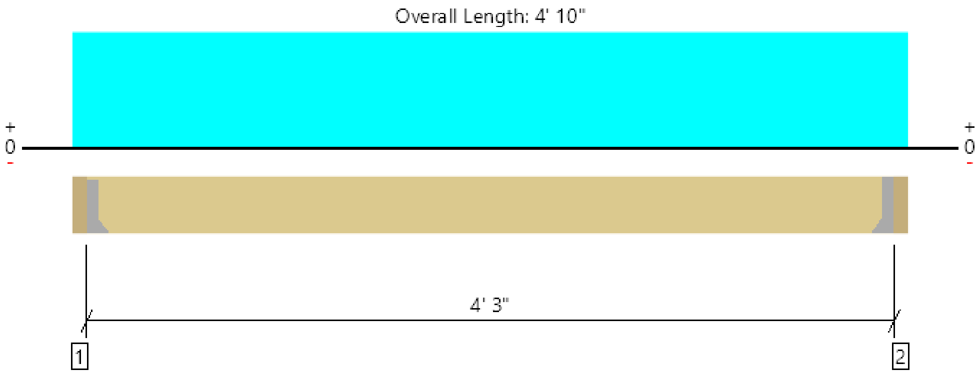
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Main Level, B1 Main Level Shower: Flush Beam (16" PSL)
1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL



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)	1229 @ 3 1/2"	4922 (1.50")	Passed (25%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	458 @ 1' 7 1/2"	16240	Passed (3%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1306 @ 2' 5"	52432	Passed (2%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.001 @ 2' 5"	0.106	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.003 @ 2' 5"	0.213	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 16" SPF beam	3.50"	Hanger[1]	1.50"	627	762	1389	See note [1]
2 - Hanger on 16" SPF beam	3.50"	Hanger[1]	1.50"	627	762	1389	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)	4' 3" o/c	
Bottom Edge (Lu)	4' 3" 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 - Face Mount Hanger	MGU5.50-SDS H=15.938	4.50"	N/A	24-SDS25212	16-SDS25212	
2 - Face Mount Hanger	MGU5.50-SDS H=15.938	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)	Comments
0 - Self Weight (PLF)	3 1/2" to 4' 6 1/2"	N/A	26.3	--	
1 - Uniform (PSF)	0 to 4' 10" (Front)	7' 10 5/8"	30.0	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
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

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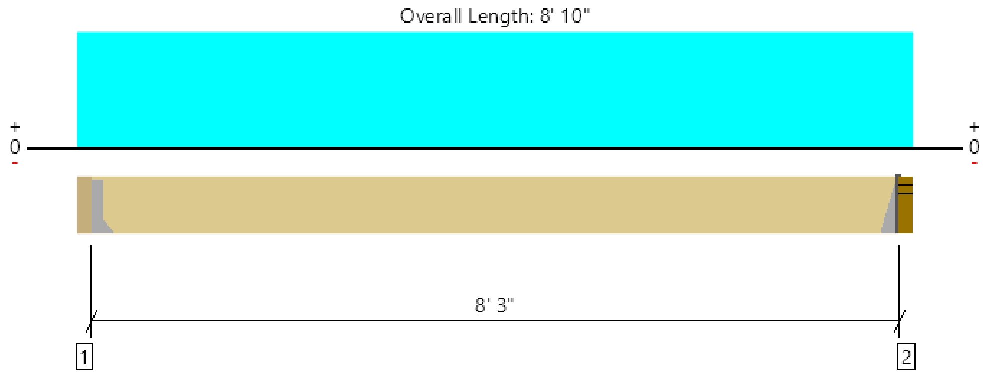
PASSED

Main Level, B1 Main Level: Transfer Beam 1 (16" PSL)
1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL



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)	3786 @ 3 1/2"	4922 (1.50")	Passed (77%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	2563 @ 1' 7 1/2"	16240	Passed (16%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	7809 @ 4' 5"	52432	Passed (15%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.021 @ 4' 5"	0.275	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.034 @ 4' 5"	0.412	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

System : Floor
Member Type : Flush Beam
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.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 16" SPF beam	3.50"	Hanger[1]	1.50"	1588	2459	4047	See note [1]
2 - Hanger on SPF studWall	3.50"	Hanger[1]	1.50"	1588	2459	4047	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)	8' 3" o/c	
Bottom Edge (Lu)	8' 3" 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 - Face Mount Hanger	MGU5.50-SDS H=15.938	4.50"	N/A	24-SDS25212	16-SDS25212	
2 - Top Mount Hanger	HB5.50/16	3.50"	6-16d	16-16d	10-16d	

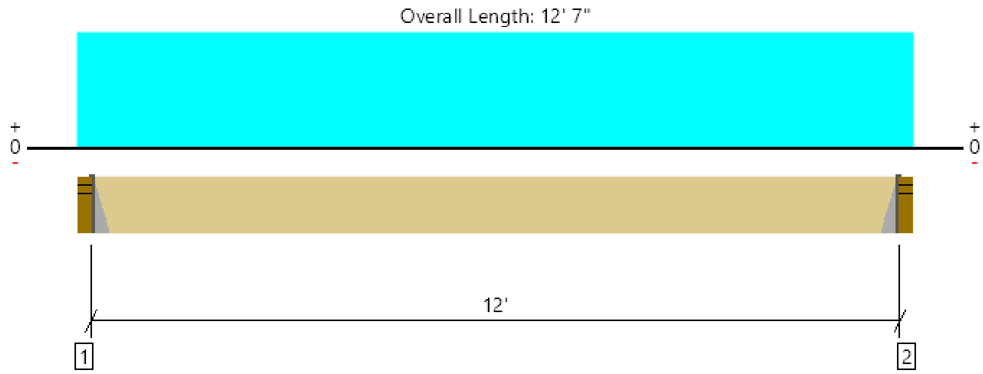
- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	3 1/2" to 8' 6 1/2"	N/A	26.3	--	
1 - Uniform (PSF)	0 to 8' 10" (Front)	5' 6"	30.0	60.0	
2 - Uniform (PSF)	0 to 8' 10" (Front)	5' 8"	30.0	40.0	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Main Level, B1 Main Level: Transfer Beam 2 (16" PSL)
1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL



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)	3482 @ 3 1/2"	4922 (1.50")	Passed (71%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	2709 @ 1' 7 1/2"	16240	Passed (17%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	10447 @ 6' 3 1/2"	52432	Passed (20%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.047 @ 6' 3 1/2"	0.400	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.082 @ 6' 3 1/2"	0.600	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

System : Floor
Member Type : Flush Beam
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.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on SPF studWall	3.50"	Hanger¹	1.50"	1557	2087	3644	See note ¹
2 - Hanger on SPF studWall	3.50"	Hanger¹	1.50"	1557	2087	3644	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)	12' o/c	
Bottom Edge (Lu)	12' 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	HB5.50/16	3.50"	6-16d	16-16d	10-16d	
2 - Top Mount Hanger	HB5.50/16	3.50"	6-16d	16-16d	10-16d	

- 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)	Comments
0 - Self Weight (PLF)	3 1/2" to 12' 3 1/2"	N/A	26.3	--	
1 - Uniform (PSF)	0 to 12' 7" (Front)	1' 9"	30.0	60.0	
2 - Uniform (PSF)	0 to 12' 7" (Front)	5' 8"	30.0	40.0	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
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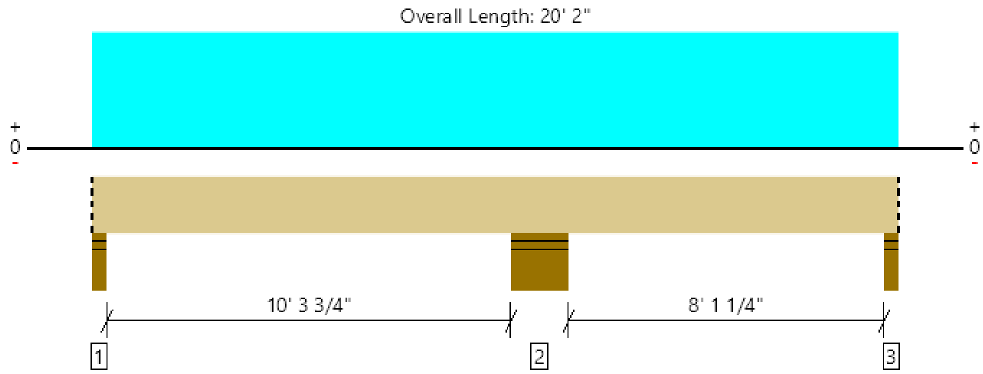
Main Level, B4 Exterior Deck: South Flush Beam (14" PSL)
1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL



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)	3077 @ 2"	5206 (3.50")	Passed (59%)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Shear (lbs)	2952 @ 9' 5 1/4"	9473	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	-7654 @ 11' 2 1/4"	27162	Passed (28%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.067 @ 5' 3 5/8"	0.276	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Total Load Defl. (in)	0.094 @ 5' 2 3/4"	0.551	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Stud wall - SPF	3.50"	3.50"	2.07"	951	1916/-153	920	3787/-153	Blocking
2 - Stud wall - SPF	14.00"	14.00"	5.46"	2633	4883	2441	9957	None
3 - Stud wall - SPF	3.50"	3.50"	1.60"	657	1593/-373	703	2953/-373	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)	20' 2" o/c	
Bottom Edge (Lu)	20' 2" 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 20' 2"	N/A	15.3	--	--	
1 - Uniform (PSF)	0 to 20' 2" (Front)	6' 6"	30.0	60.0	30.0	Default Load

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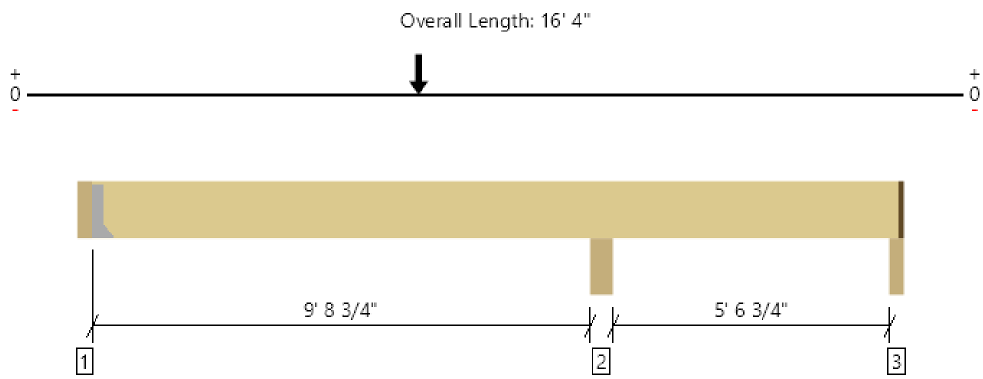
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Main Level, B5 Main Level: Wall Transfer Beam 1
1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL

An excessive uplift of -4316 lbs at support located at 3 1/2" failed this product.
An excessive uplift of -16885 lbs at support located at 10' 3" failed this product.
An excessive uplift of -3548 lbs at support located at 16' 2" failed this product.

SUPPORTS HAVE SUFFICIENT UPLIFT CAPACITY



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)	17325 @ 10' 3"	18047 (5.50")	Passed (96%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	13594 @ 8' 8 1/4"	25984	Passed (52%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	27880 @ 6' 7 1/2"	83891	Passed (33%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	-0.131 @ 6' 7 1/2"	0.249	Passed (L/909)	--	0.6 D - 0.7 E (All Spans)
Total Load Defl. (in)	0.133 @ 6' 7 1/2"	0.498	Passed (L/902)	--	1.0 D + 0.7 E (All Spans)

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.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Seismic	Total	
1 - Hanger on 16" SPF beam	3.50"	Hanger¹	1.50"	106	6256/-6256	6362/-6256	See note ¹
2 - Column - SPF	5.50"	5.50"	5.28"	275	24357/-24357	24632/-24357	None
3 - Column - SPF	3.50"	2.25"	1.50"	38	5101/-5101	5139/-5101	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- 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' 11" o/c	
Bottom Edge (Lu)	15' 11" 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 - Face Mount Hanger	MGU5.50-SDS H=15.938	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)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 16' 2 3/4"	N/A	26.3	--	
1 - Point (lb)	6' 7 1/2" (Front)	N/A	-	25512	

ForteWEB Software Operator	Job Notes
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Weyerhaeuser

Weyerhaeuser Notes

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

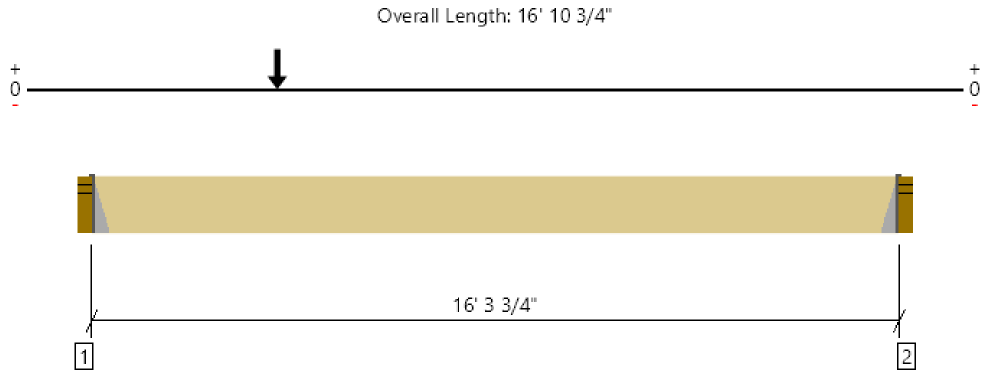
Main Level, B5 Main Level: Wall Transfer Beam 2

1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL

An excessive uplift of -13625 lbs at support located at 3 1/2" failed this product.

An excessive uplift of -3977 lbs at support located at 16' 7 1/4" failed this product.

SUPPORTS HAVE SUFFICIENT UPLIFT CAPACITY



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)	13967 @ 3 1/2"	13967 (4.26")	Passed (100%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	13932 @ 1' 7 1/2"	25984	Passed (54%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	52192 @ 4' 1/2"	83891	Passed (62%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	-0.509 @ 7' 5 7/16"	0.408	Failed (L/385)	--	0.6 D - 0.7 E (All Spans)
Total Load Defl. (in)	0.520 @ 7' 5 11/16"	0.816	Passed (L/376)	--	1.0 D + 0.7 E (All Spans)

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.

SEISMIC CASE W/ OVERSTRENGTH IS NOT APPLICABLE FOR SERVICEABILITY DEFLECTION

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Seismic	Total	
1 - Hanger on SPF studWall	3.50"	Hanger[1]	4.26"	214	19647/-19647	19861/-19647	See note [1]
2 - Hanger on SPF studWall	3.50"	Hanger[1]	1.50"	214	5865/-5865	6079/-5865	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)	16' 4" o/c	
Bottom Edge (Lu)	16' 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 - Top Mount Hanger	Connector not found	N/A	N/A	N/A	N/A	

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

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 16' 7 1/4"	N/A	26.3	--	
1 - Point (lb)	4' 1/2" (Front)	N/A	-	25512	

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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
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

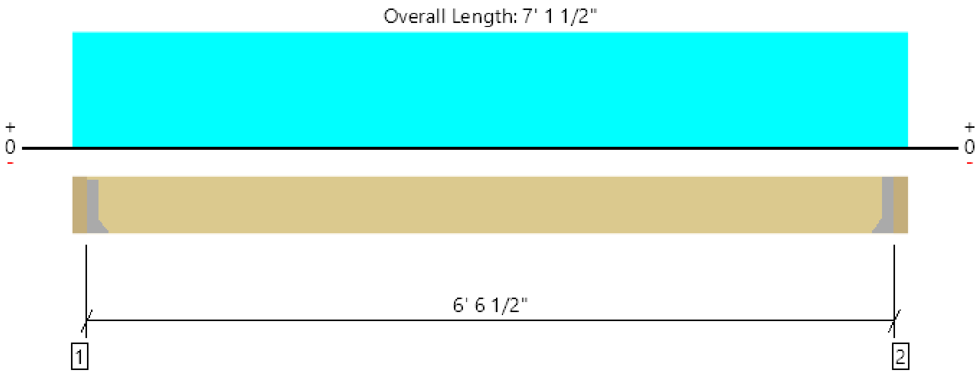
Weyerhaeuser

MEMBER REPORT

PASSED

Main Level, B6 Exterior Deck: Flush Beam

1 piece(s) 3 1/2" x 9 1/2" 2.2E Parallam® PSL



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)	2167 @ 3 1/2"	3281 (1.50")	Passed (66%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	1518 @ 1' 1"	6428	Passed (24%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3275 @ 3' 6 3/4"	13057	Passed (25%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.041 @ 3' 6 3/4"	0.164	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.061 @ 3' 6 3/4"	0.327	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 9 1/2" SPF beam	3.50"	Hanger[1]	1.50"	749	1429	715	2893	See note [1]
2 - Hanger on 9 1/2" SPF beam	3.50"	Hanger[1]	1.50"	749	1429	715	2893	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)	6' 7" 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
1 - Face Mount Hanger	HHUS48	3.00"	N/A	22-10d	8-10d	
2 - Face Mount Hanger	HHUS48	3.00"	N/A	22-10d	8-10d	

- 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 6' 10"	N/A	10.4	--	--	
1 - Uniform (PSF)	0 to 7' 1 1/2" (Front)	6' 8 1/4"	30.0	60.0	30.0	Default Load

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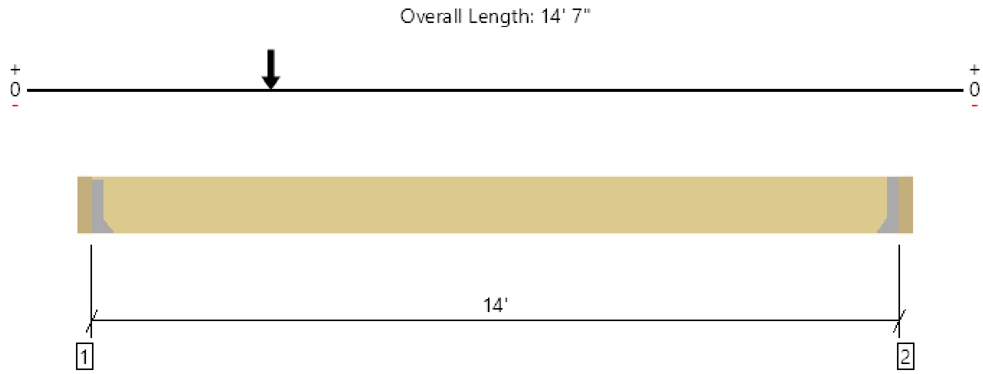
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Main Level, B6 Exterior Deck: Flush Beam (East)

1 piece(s) 3 1/2" x 9 1/2" 2.2E Parallam® PSL



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)	1749 @ 3 1/2"	3281 (1.50")	Passed (53%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	1612 @ 1' 1"	6428	Passed (25%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	4979 @ 3' 4 3/4"	13057	Passed (38%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.178 @ 6' 5 7/16"	0.350	Passed (L/946)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.273 @ 6' 6 1/16"	0.700	Passed (L/615)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 9 1/2" SPF beam	3.50"	Hanger¹	1.50"	589	1031	516	2136	See note ¹
2 - Hanger on 9 1/2" SPF beam	3.50"	Hanger¹	1.50"	220	294	147	661	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)	14' o/c	
Bottom Edge (Lu)	14' 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 - Face Mount Hanger	HHUS48	3.00"	N/A	22-10d	8-10d	
2 - Face Mount Hanger	HHUS48	3.00"	N/A	22-10d	8-10d	

- 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 14' 3 1/2"	N/A	10.4	--	--	
1 - Point (lb)	3' 4 3/4" (Front)	N/A	663	1325	663	Default Load

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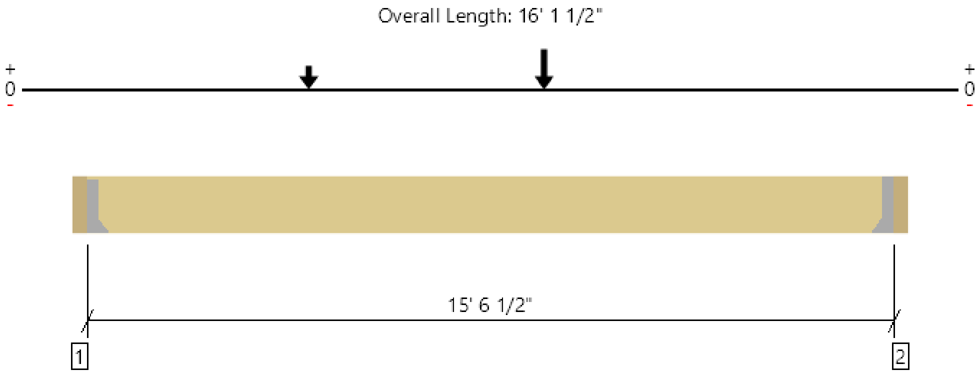
ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

FAILED

Main Level, B6 Kitchen: Transfer Beam
1 piece(s) 5 1/4" x 9 1/2" 2.2E Parallam® PSL

An excessive uplift of -1300 lbs at support located at 3 1/2" failed this product.
An excessive uplift of -2090 lbs at support located at 15' 10" failed this product.

SUPPORTS HAVE SUFFICIENT UPLIFT CAPACITY



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)	2679 @ 15' 10"	4922 (1.50")	Passed (54%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	2667 @ 15' 1/2"	15428	Passed (17%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	17676 @ 9' 1 1/4"	31337	Passed (56%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	-0.682 @ 8' 4 1/8"	0.389	Failed (L/274)	--	0.6 D - 0.7 E (All Spans)
Total Load Defl. (in)	0.819 @ 8' 2 7/16"	0.777	Failed (L/228)	--	1.0 D + 0.7 E (All Spans)

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

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

SEISMIC CASE W/ OVERSTRENGTH IS NOT APPLICABLE FOR SERVICEABILITY DEFLECTION

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Seismic	Total	
1 - Hanger on 9 1/2" SPF beam	3.50"	Hanger[1]	1.50"	774	760	2521/-2521	4055/-2521	See note [1]
2 - Hanger on 9 1/2" SPF beam	3.50"	Hanger[1]	1.50"	368	288	3301/-3301	3957/-3301	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' 7" o/c	
Bottom Edge (Lu)	15' 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
1 - Face Mount Hanger	HU68	2.50"	N/A	14-16d	6-16d	
2 - Face Mount Hanger	HHUS5.50/10	3.00"	N/A	30-10d	10-10d	

- 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)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 15' 10"	N/A	15.6	--	--	
1 - Point (lb)	9' 1 1/4" (Front)	N/A	-	-	5822	Default Load
2 - Point (lb)	4' 6 3/4" (Front)	N/A	900	1048	-	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

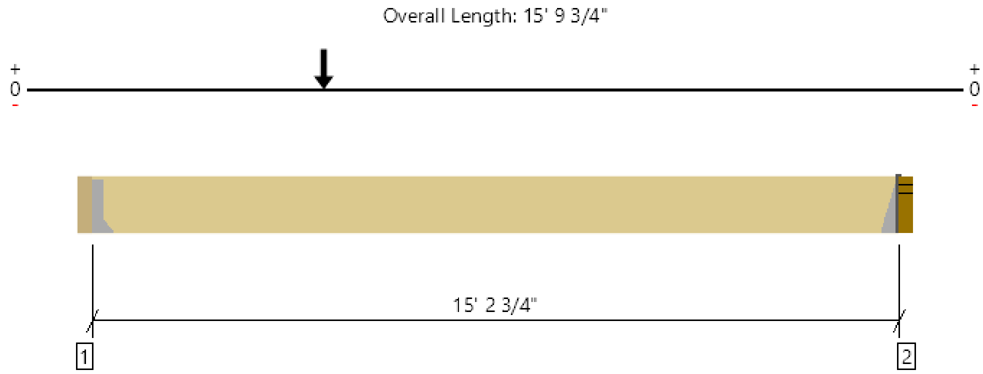
Weyerhaeuser

FAILED

Main Level, B12 Family Room: Wall Transfer Beam 2
1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL

An excessive uplift of -5443 lbs at support located at 3 1/2" failed this product.
An excessive uplift of -2131 lbs at support located at 15' 6 1/4" failed this product.

SUPPORTS HAVE SUFFICIENT UPLIFT CAPACITY



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)	5723 @ 3 1/2"	5723 (1.74")	Passed (100%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	5696 @ 1' 5 1/2"	22736	Passed (25%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	24819 @ 4' 8"	65188	Passed (38%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	-0.318 @ 7' 1 3/4"	0.381	Passed (L/575)	--	0.6 D - 0.7 E (All Spans)
Total Load Defl. (in)	0.329 @ 7' 2 1/16"	0.761	Passed (L/555)	--	1.0 D + 0.7 E (All Spans)

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

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

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Seismic	Total	Accessories
1 - Hanger on 14" SPF beam	3.50"	Hanger¹	1.74"	175	7926/-7926	8101/-7926	See note ¹
2 - Hanger on SPF studWall	3.50"	Hanger¹	1.50"	175	3195/-3195	3370/-3195	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' 3" o/c	
Bottom Edge (Lu)	15' 3" 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 - Face Mount Hanger	MGU5.50-SDS H=13.938	4.50"	N/A	24-SDS25212	16-SDS25212	
2 - Top Mount Hanger	Connector not found	N/A	N/A	N/A	N/A	

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

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 15' 6 1/4"	N/A	23.0	--	
1 - Point (lb)	4' 8" (Front)	N/A	-	11121	

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ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

FORTE WEB

MEMBER REPORT

FAILED

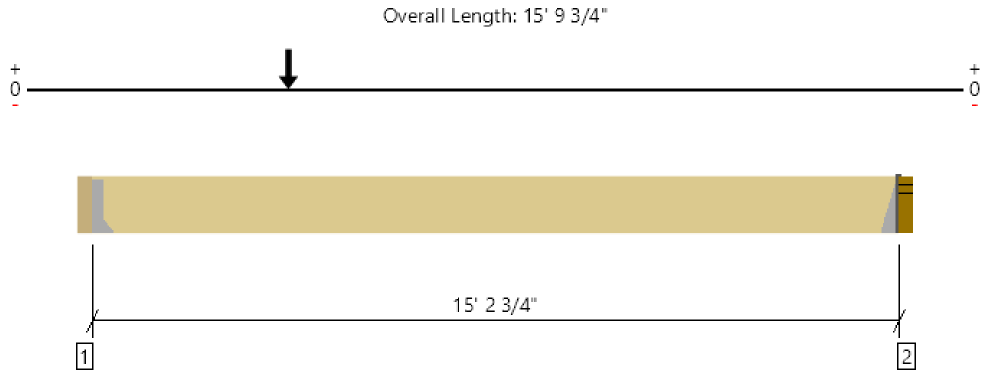
Main Level, B18 Family Room: Wall Transfer Beam 1

1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL

An excessive uplift of -11355 lbs at support located at 3 1/2" failed this product.

An excessive uplift of -3607 lbs at support located at 15' 6 1/4" failed this product.

SUPPORTS HAVE SUFFICIENT UPLIFT CAPACITY



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)	11541 @ 3 1/2"	11541 (5.28")	Passed (100%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	11523 @ 1' 5 1/2"	15157	Passed (76%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	42693 @ 4'	43459	Passed (98%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	-0.813 @ 7' 3/16"	0.381	Failed (L/225)	--	0.6 D - 0.7 E (All Spans)
Total Load Defl. (in)	0.824 @ 7' 5/16"	0.761	Failed (L/222)	--	1.0 D + 0.7 E (All Spans)

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.

SEISMIC CASE W/ OVERSTRENGTH IS NOT APPLICABLE FOR SERVICEABILITY DEFLECTION

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Seismic	Total	
1 - Hanger on 14" SPF beam	3.50"	Hanger[1]	5.28"	117	16321/-16321	16438/-16321	See note [1]
2 - Hanger on SPF studWall	3.50"	Hanger[1]	1.73"	117	5253/-5253	5370/-5253	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)	2' 9" o/c	
Bottom Edge (Lu)	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 - Face Mount Hanger	Connector not found	N/A	N/A	N/A	N/A	
2 - Top Mount Hanger	Connector not found	N/A	N/A	N/A	N/A	

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

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 15' 6 1/4"	N/A	15.3	--	
1 - Point (lb)	4' (Front)	N/A	-	21574	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Weyerhaeuser

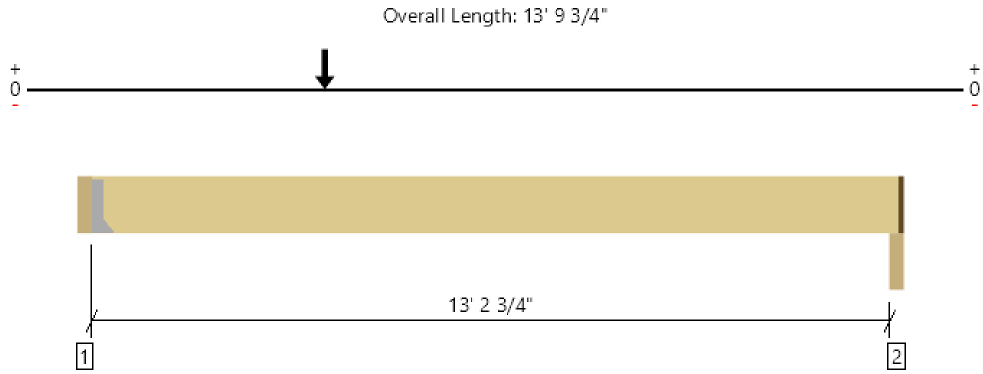
FAILED

Main Level, B19 Family Room: Transfer Beam 3
1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL

An excessive uplift of -8038 lbs at support located at 3 1/2" failed this product.

An excessive uplift of -3131 lbs at support located at 13' 7 3/4" failed this product.

SUPPORTS HAVE SUFFICIENT UPLIFT CAPACITY



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)	8418 @ 3 1/2"	8418 (2.57")	Passed (100%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	8391 @ 1' 5 1/2"	22736	Passed (37%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	31751 @ 4' 1"	65188	Passed (49%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	-0.320 @ 6' 3 1/2"	0.334	Passed (L/501)	--	0.6 D - 0.7 E (All Spans)
Total Load Defl. (in)	0.330 @ 6' 3 11/16"	0.668	Passed (L/486)	--	1.0 D + 0.7 E (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Seismic	Total	
1 - Hanger on 14" SPF beam	3.50"	Hanger[1]	2.57"	237	11686/-11686	11923/-11686	See note [1]
2 - Column - SPF	3.50"	2.25"	1.50"	188	4634/-4634	4822/-4634	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- 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)	13' 5" o/c	
Bottom Edge (Lu)	13' 5" 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 - Face Mount Hanger	HGU5.50-SDS H=13.938	5.25"	N/A	36-SDS25212	24-SDS25212	

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

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 13' 8 1/2"	N/A	23.0	--	
1 - Point (lb)	4' 1" (Front)	N/A	117	16320	

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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
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Weyerhaeuser

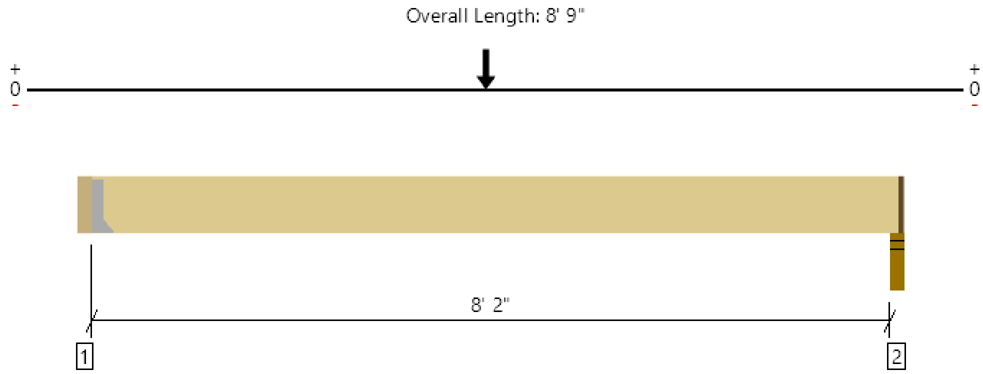
Main Level, B19 Family Room: Transfer Beam 4

1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL

An excessive uplift of -2879 lbs at support located at 3 1/2" failed this product.

An excessive uplift of -2624 lbs at support located at 8' 7" failed this product.

SUPPORTS HAVE SUFFICIENT UPLIFT CAPACITY



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)	3178 @ 3 1/2"	4922 (1.50")	Passed (65%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	3151 @ 1' 5 1/2"	22736	Passed (14%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	12398 @ 4' 3"	65188	Passed (19%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	-0.061 @ 4' 3"	0.207	Passed (L/999+)	--	0.6 D - 0.7 E (All Spans)
Total Load Defl. (in)	0.064 @ 4' 3"	0.415	Passed (L/999+)	--	1.0 D + 0.7 E (All Spans)

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.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Seismic	Total	
1 - Hanger on 14" SPF beam	3.50"	Hanger[1]	1.50"	187	4273/-4273	4460/-4273	See note [1]
2 - Stud wall - SPF	3.50"	2.25"	1.50"	180	3903/-3903	4083/-3903	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- 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)	8' 4" o/c	
Bottom Edge (Lu)	8' 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 - Face Mount Hanger	HGUS5.50/10	4.00"	N/A	46-10d	16-10d	

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

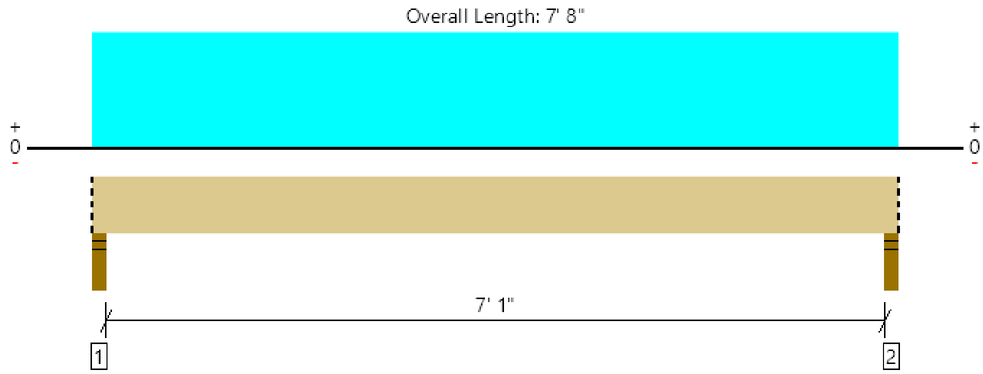
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 8' 7 3/4"	N/A	23.0	--	
1 - Point (lb)	4' 3" (Front)	N/A	175	8176	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Main Level, B20 Living Room: Drop Beam

1 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL



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)	2750 @ 2"	5206 (3.50")	Passed (53%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1584 @ 1' 7 1/2"	10827	Passed (15%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	4823 @ 3' 10"	34955	Passed (14%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.015 @ 3' 10"	0.244	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.027 @ 3' 10"	0.367	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	3.50"	1.85"	1217	1533	2750	Blocking
2 - Stud wall - SPF	3.50"	3.50"	1.85"	1217	1533	2750	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' 8" o/c	
Bottom Edge (Lu)	7' 8" o/c	

•Maximum allowable bracing intervals based on applied load.

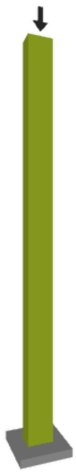
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 7' 8"	N/A	17.5	--	
1 - Uniform (PSF)	0 to 7' 8" (Front)	10'	30.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Main Level, C7 Post Transfer

1 piece(s) 5 1/4" x 7" 1.8E Parallam® PSL



Drawing is Conceptual

Post Height: 9' 6"

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	22	50	Passed (43%)	--	--
Compression (lbs)	23195	55282	Passed (42%)	1.60	1.0 D + 0.525 E + 0.75 L + 0.75 S
Base Bearing (lbs)	23195	23336	Passed (99%)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S
Bending/Compression	N/A	1	Passed (N/A)	--	N/A

- Input axial load eccentricity for the design is zero
- Applicable calculations are based on NDS.
- Bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogeneous material with sufficient stiffness to distribute applied load.

Supports	Type	Material
Base	Plate	Parallam® PSL

Member Type : Free Standing Post
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Vertical Load	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Seismic (1.60)	Comments
1 - Point (lb)	5751	5572	5751	17050	Default Load

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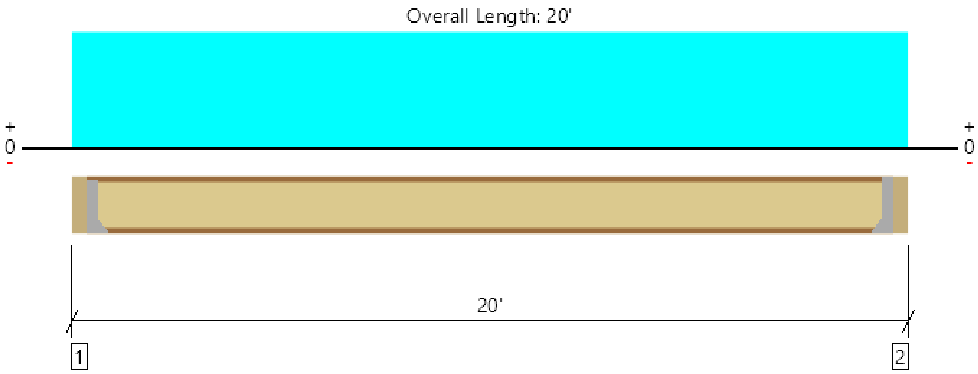
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Weyerhaeuser

Upper Level, J5 Upper Level: Joist (14" TJI)
1 piece(s) 14" TJI® 360 @ 12" 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)	680 @ 3 1/2"	1080 (1.75")	Passed (63%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	680 @ 3 1/2"	1955	Passed (35%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3299 @ 10'	7335	Passed (45%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.212 @ 10'	0.485	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.371 @ 10'	0.971	Passed (L/628)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	52	50	Passed	--	--

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 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: bridging or blocking at max. 8' o.c..

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 14" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	300	400	700	See note [1]
2 - Hanger on 14" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	300	400	700	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)	5' 8" o/c	
Bottom Edge (Lu)	19' 5" 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	IUS2.37/14	2.00"	N/A	12-10dx1.5	2-Strong-Grip	
2 - Face Mount Hanger	IUS2.37/14	2.00"	N/A	12-10dx1.5	2-Strong-Grip	

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

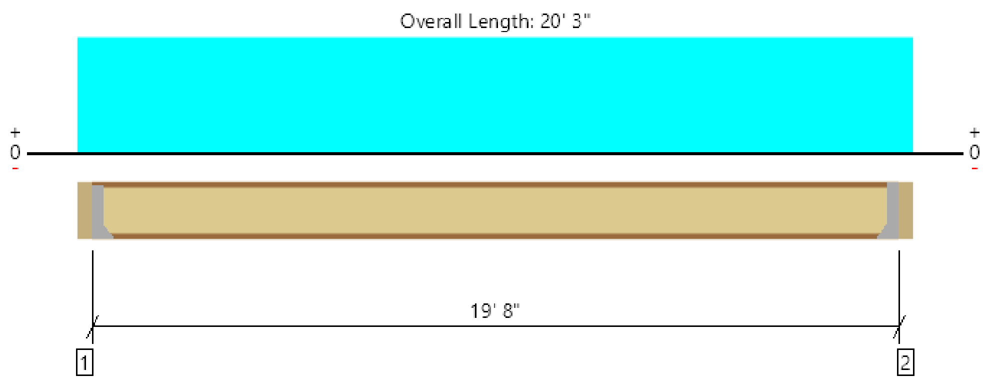
Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 20'	12"	30.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Upper Level, J6 Upper Deck: Joist - Long (11-7/8" TJI)

1 piece(s) 11 7/8" TJI® 560 @ 12" 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)	885 @ 3 1/2"	1265 (1.75")	Passed (70%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	885 @ 3 1/2"	2050	Passed (43%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	4351 @ 10' 1 1/2"	9500	Passed (46%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.373 @ 10' 1 1/2"	0.492	Passed (L/632)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.539 @ 10' 1 1/2"	0.983	Passed (L/438)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
TJ-Pro™ Rating	51	50	Passed	--	--

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 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: bridging or blocking at max. 8' o.c..

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	304	607	304	1215	See note [1]
2 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	304	607	304	1215	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)	8' 2" o/c	
Bottom Edge (Lu)	19' 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	IUS3.56/11.88	2.00"	N/A	12-10dx1.5	2-Strong-Grip	
2 - Face Mount Hanger	IUS3.56/11.88	2.00"	N/A	12-10dx1.5	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)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 20' 3"	12"	30.0	60.0	30.0	Default Load

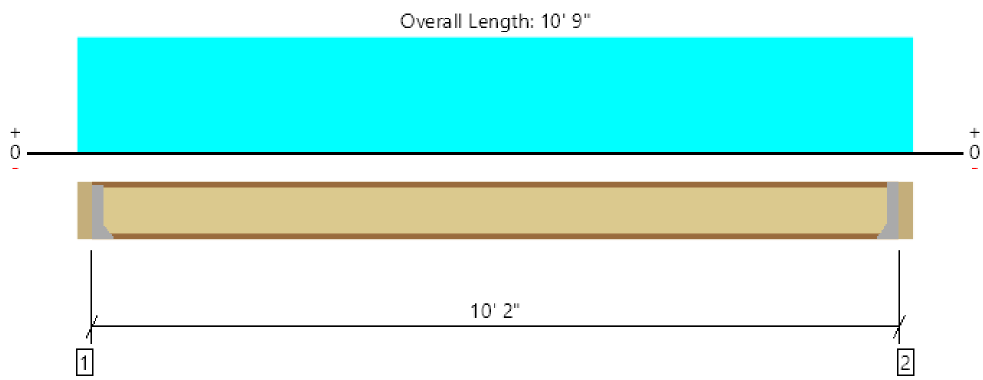
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Weyerhaeuser

Upper Level, J8 Upper Deck: Joist - Short (11-7/8" TJI)

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)	610 @ 3 1/2"	910 (1.75")	Passed (67%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	610 @ 3 1/2"	1560	Passed (39%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1550 @ 5' 4 1/2"	3160	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.085 @ 5' 4 1/2"	0.254	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.123 @ 5' 4 1/2"	0.508	Passed (L/990)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
TJ-Pro™ Rating	60	50	Passed	--	--

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 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: bridging or blocking at max. 8' o.c..

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	215	430	215	860	See note [1]
2 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	215	430	215	860	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' 5" o/c	
Bottom Edge (Lu)	10' 2" 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	

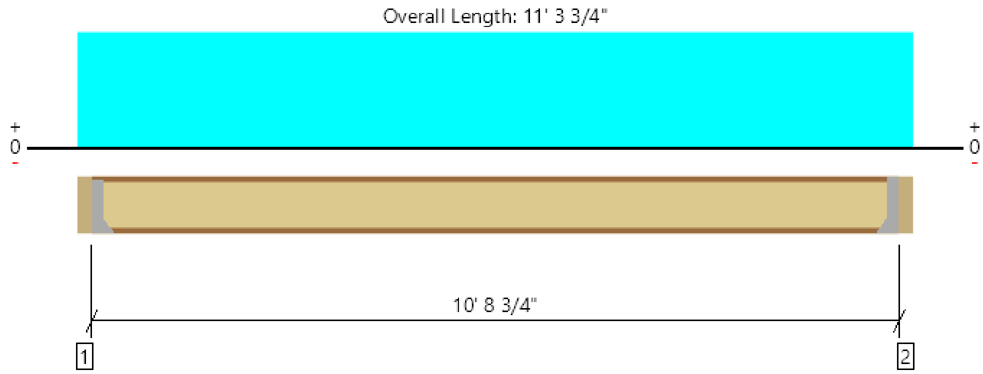
- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 10' 9"	16"	30.0	60.0	30.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Upper Level, J8 Stair Roof: Joist (11-7/8" TJI)
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)	376 @ 3 1/2"	1047 (1.75")	Passed (36%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	376 @ 3 1/2"	1794	Passed (21%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	1007 @ 5' 7 7/8"	3634	Passed (28%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.057 @ 5' 7 7/8"	0.268	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.080 @ 5' 7 7/8"	0.536	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
TJ-Pro™ Rating	58	50	Passed	--	--

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 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: bridging or blocking at max. 8' o.c..

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	113	151	226	490	See note [1]
2 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	113	151	226	490	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)	5' 9" o/c	
Bottom Edge (Lu)	10' 9" 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)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 11' 3 3/4"	16"	15.0	20.0	30.0	Default Load

Weyerhaeuser Notes

Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC-ES under evaluation reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

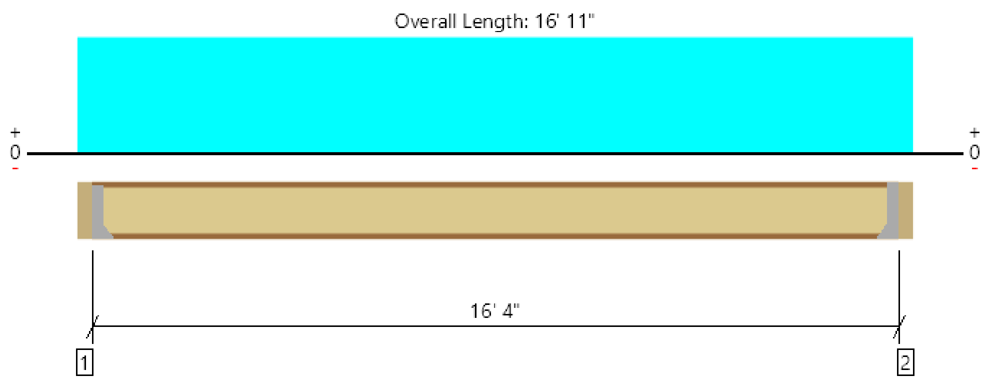
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

MEMBER REPORT

PASSED

Upper Level, J9 Upper Deck: Joist - Med (11-7/8" TJI)
1 piece(s) 11 7/8" TJI® 230 @ 12" 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)	735 @ 3 1/2"	1060 (1.75")	Passed (69%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	735 @ 3 1/2"	1655	Passed (44%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3001 @ 8' 5 1/2"	4215	Passed (71%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.306 @ 8' 5 1/2"	0.408	Passed (L/641)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.442 @ 8' 5 1/2"	0.817	Passed (L/444)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
TJ-Pro™ Rating	50	50	Passed	--	--

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 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: bridging or blocking at max. 8' o.c..

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	254	507	254	1015	See note [1]
2 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	254	507	254	1015	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' 8" o/c	
Bottom Edge (Lu)	16' 4" 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	IUS2.37/11.88	2.00"	N/A	10-10dx1.5	2-Strong-Grip	
2 - Face Mount Hanger	IUS2.37/11.88	2.00"	N/A	10-10dx1.5	2-Strong-Grip	

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

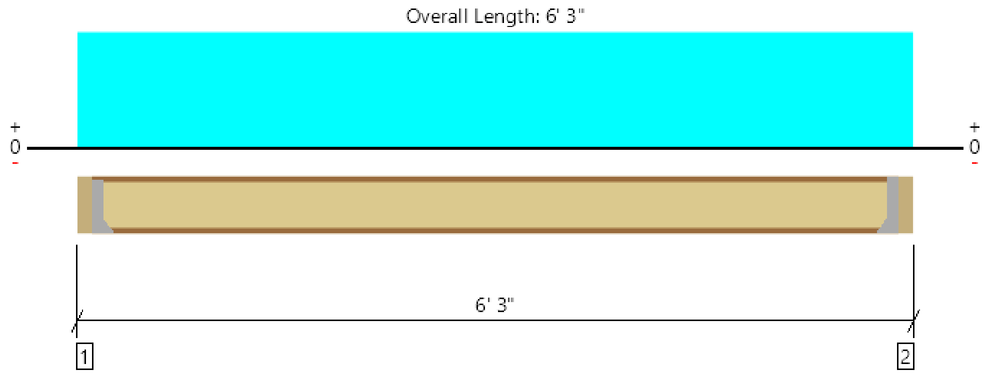
Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 16' 11"	12"	30.0	60.0	30.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Upper Level, J10 Upper Level Shower: Joist (9-1/2" TJI)

1 piece(s) 9 1/2" 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)	264 @ 3 1/2"	910 (1.75")	Passed (29%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	264 @ 3 1/2"	1220	Passed (22%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	375 @ 3' 1 1/2"	2500	Passed (15%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.011 @ 3' 1 1/2"	0.142	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.019 @ 3' 1 1/2"	0.283	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	67	50	Passed	--	--

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 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: bridging or blocking at max. 8' o.c..

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 9 1/2" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	125	167	292	See note [1]
2 - Hanger on 9 1/2" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	125	167	292	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)	5' 8" o/c	
Bottom Edge (Lu)	5' 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	IUS1.81/9.5	2.00"	N/A	8-10dx1.5	2-Strong-Grip	
2 - Face Mount Hanger	IUS1.81/9.5	2.00"	N/A	8-10dx1.5	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)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 6' 3"	16"	30.0	40.0	Default Load

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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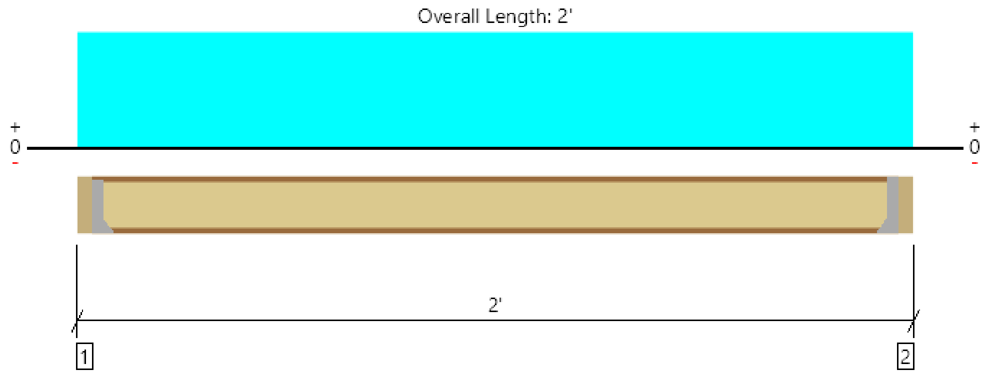
Upper Level, J11 Upper Deck: Joist (11-7/8" TJI)
1 piece(s) 11 7/8" TJI® 110 @ 24" OC



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)	96 @ 3 1/2"	1047 (1.75")	Passed (9%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	96 @ 3 1/2"	1794	Passed (5%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	34 @ 1'	3634	Passed (1%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.000 @ 3 1/2"	0.035	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.000 @ 3 1/2"	0.071	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
TJ-Pro™ Rating	72	50	Passed	--	--

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 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: bridging or blocking at max. 8' o.c..

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	60	40	60	160	See note [1]
2 - Hanger on 11 7/8" PSL beam	3.50"	Hanger[1]	1.75" / - [2]	60	40	60	160	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)	1' 5" o/c	
Bottom Edge (Lu)	1' 5" 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)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 2'	24"	30.0	20.0	30.0	Default Load

Weyerhaeuser Notes

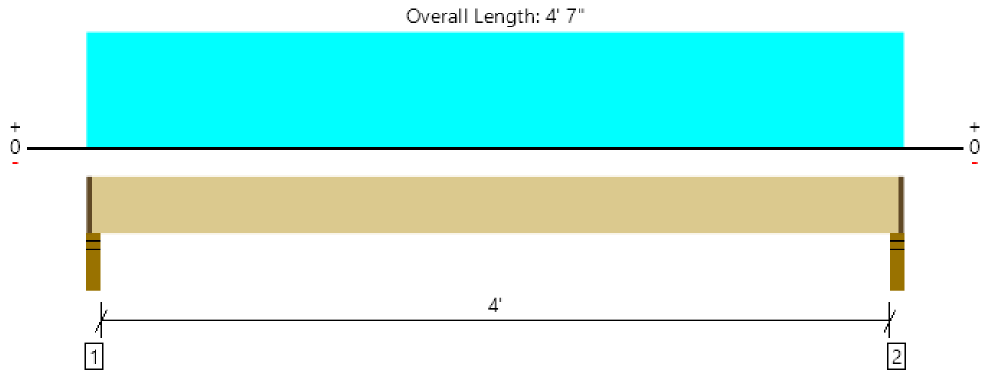
ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

FORTE WEB

MEMBER REPORT

PASSED

Upper Level, B4 Upper Level Shower: Short Flush Beam (14" PSL)
1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL



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)	1552 @ 2"	3347 (2.25")	Passed (46%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	591 @ 1' 5 1/2"	9473	Passed (6%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1602 @ 2' 3 1/2"	27162	Passed (6%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.004 @ 2' 3 1/2"	0.106	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.006 @ 2' 3 1/2"	0.213	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.50"	715	909	1624	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.50"	715	909	1624	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' 5" o/c	
Bottom Edge (Lu)	4' 5" o/c	

- Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 4' 5 3/4"	N/A	15.3	--	
1 - Uniform (PSF)	0 to 4' 7" (Front)	9' 11"	30.0	40.0	Default Load

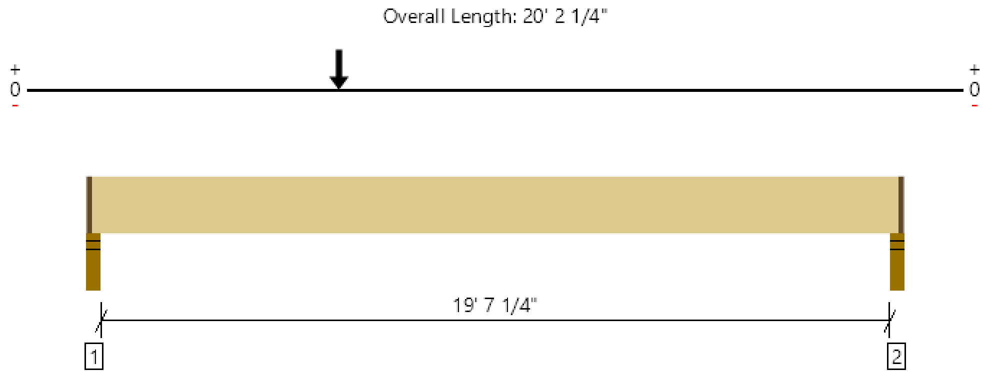
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Weyerhaeuser

Upper Level, B4 Upper Level Shower: Long Flush Beam (14" PSL)

1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL



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)	1281 @ 2"	3347 (2.25")	Passed (38%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1260 @ 1' 5 1/2"	9473	Passed (13%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	7479 @ 6' 2 3/4"	27162	Passed (28%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.125 @ 9' 1 5/8"	0.496	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.254 @ 9' 3"	0.993	Passed (L/938)	--	1.0 D + 1.0 L (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.50"	650	631	1281	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.50"	371	278	649	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)	20' o/c	
Bottom Edge (Lu)	20' o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 20' 1"	N/A	15.3	--	
1 - Point (lb)	6' 2 3/4" (Front)	N/A	715	909	Default Load

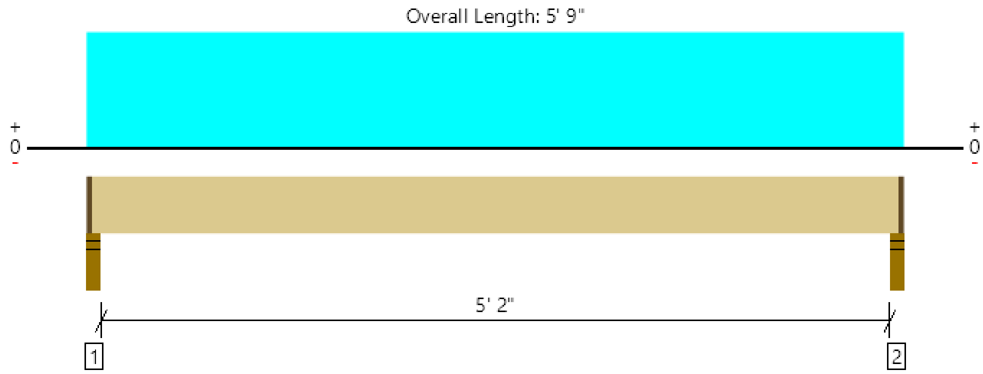
Weyerhaeuser Notes

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

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Upper Level, B4 Upper Level: Flush Beam (14" PSL)

1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL



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)	1538 @ 2"	3347 (2.25")	Passed (46%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	786 @ 1' 5 1/2"	9473	Passed (8%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2035 @ 2' 10 1/2"	27162	Passed (7%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.006 @ 2' 10 1/2"	0.135	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.010 @ 2' 10 1/2"	0.271	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.50"	707	886	1593	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.50"	707	886	1593	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)	5' 7" o/c	
Bottom Edge (Lu)	5' 7" o/c	

- Maximum allowable bracing intervals based on applied load.

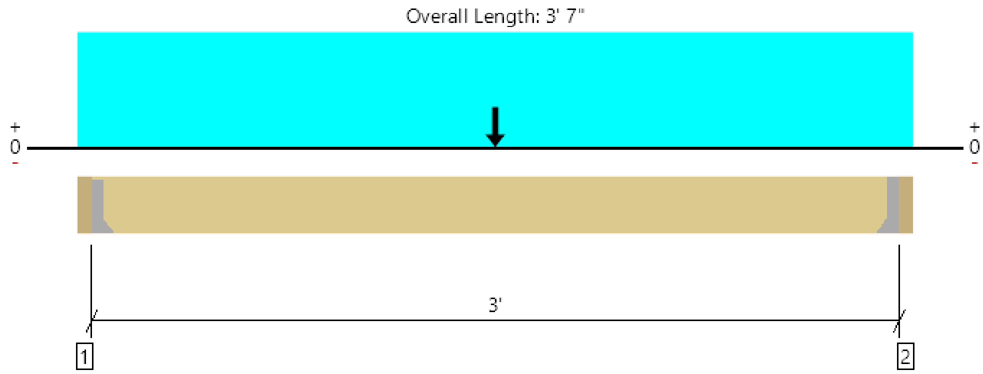
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 5' 7 3/4"	N/A	15.3	--	
1 - Uniform (PSF)	0 to 5' 9" (Front)	7' 8 1/2"	30.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Weyerhaeuser

Upper Level, B4 Upper Level: Transfer Beam 4 (14" PSL)
1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL



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)	2776 @ 3 1/2"	3281 (1.50")	Passed (85%)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	1764 @ 1' 5 1/2"	15157	Passed (12%)	1.60	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	2011 @ 1' 9 1/2"	27162	Passed (7%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.007 @ 1' 9 1/2"	0.100	Passed (L/999+)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.009 @ 1' 9 1/2"	0.150	Passed (L/999+)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)

System : Floor
Member Type : Flush Beam
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.
- -649 lbs uplift at support located at 3 1/2". Strapping or other restraint may be required.
- -649 lbs uplift at support located at 3' 3 1/2". Strapping or other restraint may be required.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Seismic	Total	
1 - Hanger on 14" SPF beam	3.50"	Hanger[1]	1.50"	1060	1334	1837/-1837	4231/-1837	See note [1]
2 - Hanger on 14" SPF beam	3.50"	Hanger[1]	1.50"	1060	1334	1837/-1837	4231/-1837	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)	3' o/c	
Bottom Edge (Lu)	3' 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 - Face Mount Hanger	HHUS410	3.00"	N/A	30-10d	10-10d	
2 - Face Mount Hanger	HHUS410	3.00"	N/A	30-10d	10-10d	

- 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)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 3' 3 1/2"	N/A	15.3	--	--	
1 - Point (lb)	1' 9 1/2" (Front)	N/A	612	546	3673	Default Load
2 - Uniform (PSF)	0 to 3' 7" (Front)	2' 4 3/4"	30.0	60.0	-	
3 - Uniform (PSF)	0 to 3' 7" (Front)	11' 2 1/2"	30.0	40.0	-	

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

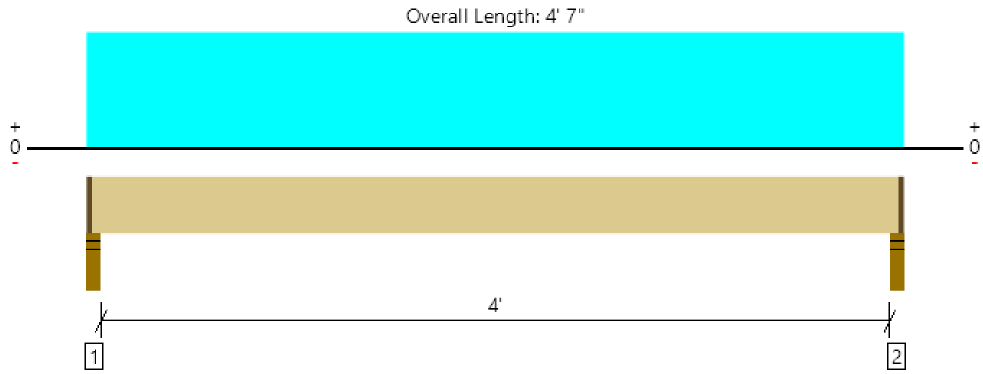
Weyerhaeuser Notes

Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC-ES under evaluation reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

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

PASSED

Upper Level, B7 Upper Level: Typical Header Beam (2-2x10)
2 piece(s) 2 x 10 Spruce-Pine-Fir No. 1 / No. 2



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)	59 @ 2"	2869 (2.25")	Passed (2%)	--	1.0 D (All Spans)
Shear (lbs)	33 @ 1' 3/4"	2248	Passed (1%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	61 @ 2' 3 1/2"	3088	Passed (2%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.000 @ 1 1/4"	0.106	Passed (L/999+)	--	1.0 D (All Spans)
Total Load Defl. (in)	0.001 @ 2' 3 1/2"	0.213	Passed (L/999+)	--	1.0 D (All Spans)

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.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)		Accessories
	Total	Available	Required	Dead	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.50"	61	61	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.50"	61	61	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' 5" o/c	
Bottom Edge (Lu)	4' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Comments
0 - Self Weight (PLF)	1 1/4" to 4' 5 3/4"	N/A	7.0	
1 - Uniform (PLF)	0 to 4' 7" (Front)	N/A	20.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
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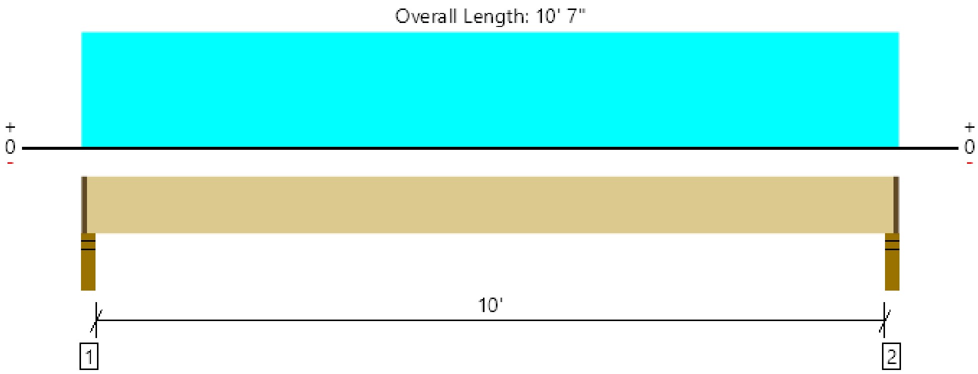
Upper Level, B12 Upper Level: Flush Beam (14" PSL)

1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL



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)	3679 @ 2"	5020 (2.25")	Passed (73%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	2718 @ 1' 5 1/2"	14210	Passed (19%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	9313 @ 5' 3 1/2"	40743	Passed (23%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.044 @ 5' 3 1/2"	0.256	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.080 @ 5' 3 1/2"	0.512	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.65"	1675	2075	3750	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.65"	1675	2075	3750	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)	10' 5" o/c	
Bottom Edge (Lu)	10' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 10' 5 3/4"	N/A	23.0	--	
1 - Uniform (PSF)	0 to 10' 7" (Front)	9' 9 5/8"	30.0	40.0	Default Load
2 - Uniform (PLF)	0 (Front)	N/A	20.0	-	

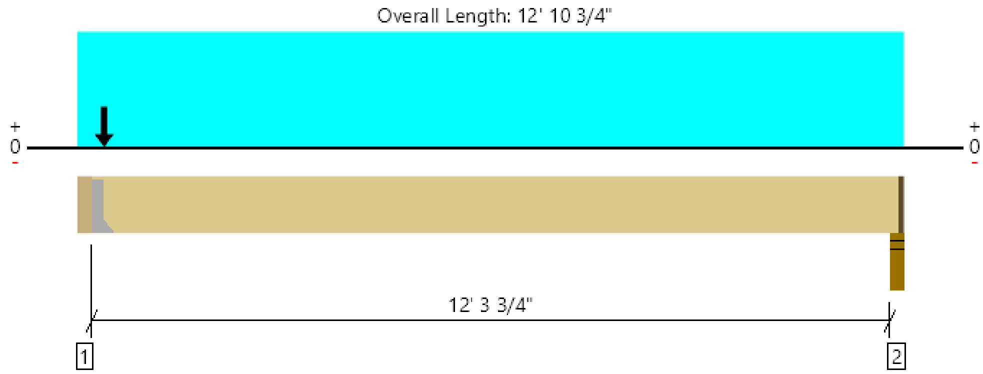
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Upper Level, B12 Upper Level: Transfer Beam (14" PSL)

1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL

An excessive uplift of -2553 lbs at support located at 3 1/2" failed this product. SUPPORT HAS SUFFICIENT UPLIFT CAPACITY



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)	7224 @ 3 1/2"	7224 (2.20")	Passed (100%)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	3959 @ 1' 5 1/2"	14210	Passed (28%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	15152 @ 6' 6 1/8"	40743	Passed (37%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.102 @ 6' 6 1/8"	0.311	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.181 @ 6' 6 1/8"	0.622	Passed (L/823)	--	1.0 D + 1.0 L (All Spans)

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.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Seismic	Total	
1 - Hanger on 14" SPF beam	3.50"	Hanger¹	2.20"	2214	2881	179	5545/-5545	10819/-5545	See note ¹
2 - Stud wall - SPF	3.50"	2.25"	2.21"	2176	2826	176	85/-85	5263/-85	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- 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)	12' 6" o/c	
Bottom Edge (Lu)	12' 6" 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 - Face Mount Hanger	Connector not found	N/A	N/A	N/A	N/A	

- 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)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 12' 9 1/2"	N/A	23.0	--	--	--	
1 - Uniform (PSF)	0 to 12' 10 3/4" (Front)	9' 8 1/4"	30.0	40.0	-	-	Default Load
2 - Uniform (PSF)	0 to 12' 10 3/4" (Front)	11"	30.0	60.0	30.0	-	
3 - Point (lb)	5 3/4" (Front)	N/A	-	-	-	5630	

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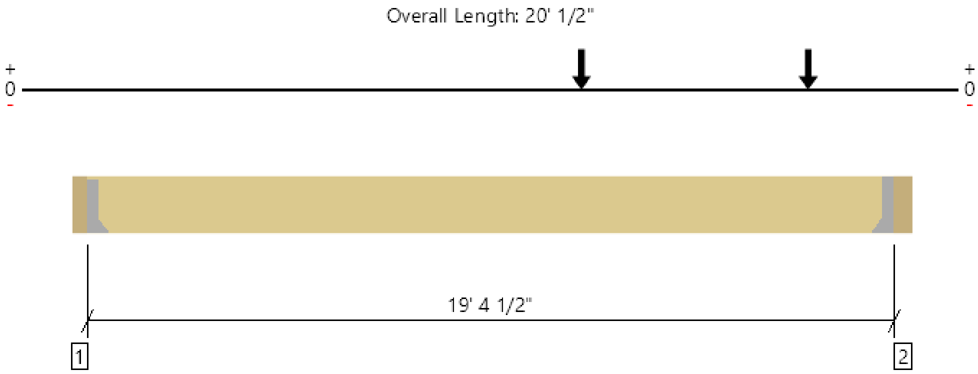
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Upper Level, B12 Upper Level: Transfer Beam 2 (14" PSL)
1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL

An excessive uplift of -1851 lbs at support located at 3 1/2" failed this product.

An excessive uplift of -5976 lbs at support located at 19' 8" failed this product.

SUPPORTS HAVE SUFFICIENT UPLIFT CAPACITY



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)	6332 @ 19' 8"	6332 (1.93")	Passed (100%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	6305 @ 18' 6"	22736	Passed (28%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	24666 @ 12' 2 1/2"	65188	Passed (38%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	-0.531 @ 10' 9 5/16"	0.646	Passed (L/438)	--	0.6 D - 0.7 E (All Spans)
Total Load Defl. (in)	0.560 @ 10' 8 7/8"	0.969	Passed (L/416)	--	1.0 D + 0.7 E (All Spans)

System : Floor
Member Type : Flush Beam
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.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Seismic	Total	
1 - Hanger on 14" SPF beam	3.50"	Hanger¹	1.50"	223	2835/-2835	3058/-2835	See note ¹
2 - Hanger on 14" SPF beam	4.50"	Hanger¹	1.93"	223	8727/-8727	8950/-8727	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)	19' 5" o/c	
Bottom Edge (Lu)	19' 5" 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 - Face Mount Hanger	HHUS5.50/10	3.00"	N/A	30-10d	10-10d	
2 - Face Mount Hanger	MGU5.50-SDS H=13.938	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)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 19' 8"	N/A	23.0	--	
1 - Point (lb)	12' 2 1/2" (Front)	N/A	-	5781	Default Load
2 - Point (lb)	17' 7 1/2" (Front)	N/A	-	5781	Default Load

ForteWEB Software Operator	Job Notes
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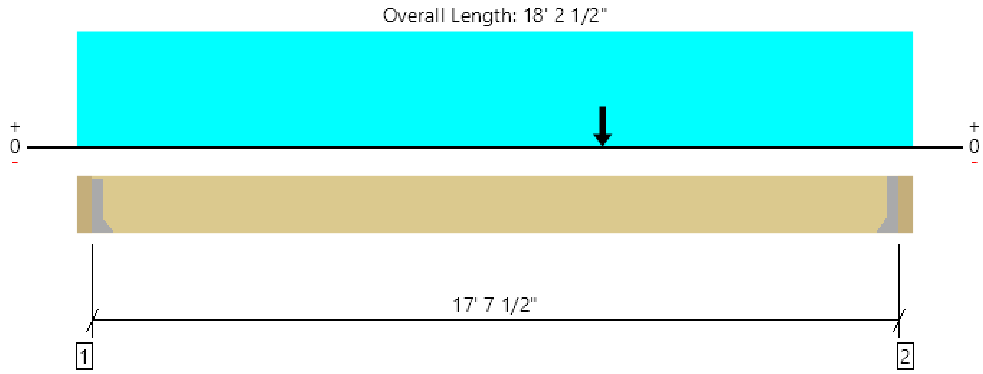
Upper Level, B12 Upper Level: Transfer Beam 3 (14" PSL)
1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL

An excessive uplift of -2203 lbs at support located at 3 1/2" failed this product.

An excessive uplift of -4077 lbs at support located at 17' 11" failed this product.

SUPPORTS HAVE SUFFICIENT UPLIFT CAPACITY



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)	5044 @ 17' 11"	5044 (1.54")	Passed (100%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	4964 @ 16' 9"	22736	Passed (22%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	31157 @ 11' 5 1/2"	65188	Passed (48%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	-0.510 @ 9' 9"	0.587	Passed (L/414)	--	0.6 D - 0.7 E (All Spans)
Total Load Defl. (in)	0.570 @ 9' 8 3/16"	0.881	Passed (L/371)	--	1.0 D + 0.7 E (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Seismic	Total	
1 - Hanger on 14" SPF beam	3.50"	Hanger[1]	1.50"	612	546	3672/-3672	4830/-3672	See note [1]
2 - Hanger on 14" SPF beam	3.50"	Hanger[1]	1.54"	612	546	6350/-6350	7508/-6350	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)	17' 8" o/c	
Bottom Edge (Lu)	17' 8" 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 - Face Mount Hanger	HHUS5.50/10	3.00"	N/A	30-10d	10-10d	
2 - Face Mount Hanger	HGUS5.50/10	4.00"	N/A	46-16d	16-16d	

- 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)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 17' 11"	N/A	23.0	--	--	
1 - Point (lb)	11' 5 1/2" (Front)	N/A	-	-	10022	Default Load
2 - Uniform (PSF)	0 to 18' 2 1/2" (Front)	1' 6"	30.0	40.0	-	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
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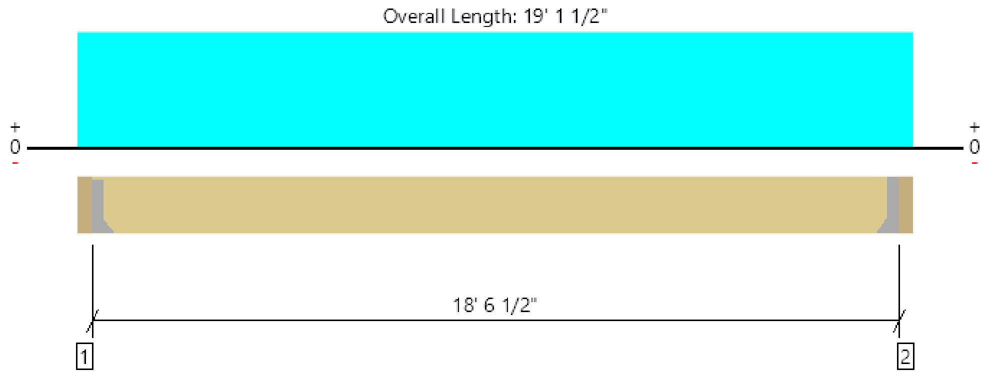
Upper Level, B13 Upper Deck: Edge Beam (11-7/8" LVL)
2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL



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)	1234 @ 3 1/2"	3938 (1.50")	Passed (31%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	1102 @ 1' 3 3/8"	9081	Passed (12%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	5718 @ 9' 6 3/4"	20525	Passed (28%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.191 @ 9' 6 3/4"	0.464	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.378 @ 9' 6 3/4"	0.927	Passed (L/588)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

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.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 11 7/8" SPF beam	3.50"	Hanger[1]	1.50"	626	343	514	1483	See note [1]
2 - Hanger on 11 7/8" SPF beam	3.50"	Hanger[1]	1.50"	626	343	514	1483	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)	18' 7" o/c	
Bottom Edge (Lu)	18' 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
1 - Face Mount Hanger	LUS410	2.00"	N/A	8-10dx1.5	6-10d	
2 - Face Mount Hanger	LUS410	2.00"	N/A	8-10dx1.5	6-10d	

- 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 18' 10"	N/A	12.1	--	--	
1 - Uniform (PSF)	0 to 19' 1 1/2" (Front)	1' 9 1/2"	30.0	20.0	30.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

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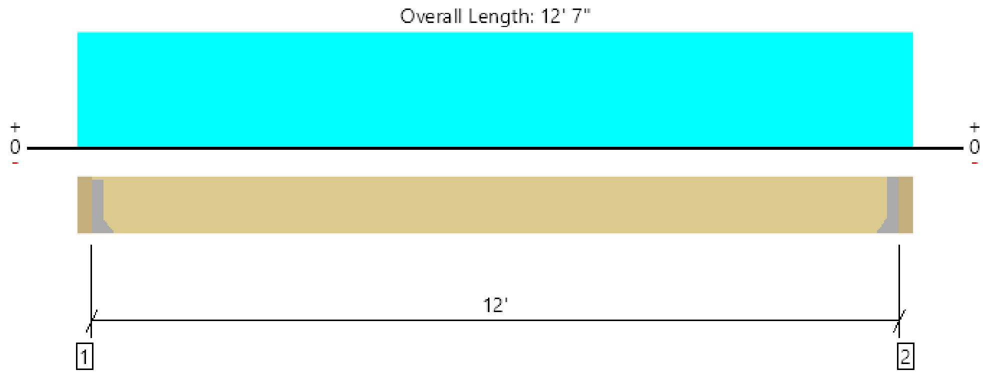
Upper Level, B13 Upper Deck: Flush Beam (11-7/8" LVL)
2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL



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)	3488 @ 3 1/2"	3938 (1.50")	Passed (89%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	2913 @ 1' 3 3/8"	9081	Passed (32%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	10464 @ 6' 3 1/2"	20525	Passed (51%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.174 @ 6' 3 1/2"	0.300	Passed (L/828)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.307 @ 6' 3 1/2"	0.600	Passed (L/470)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 11 7/8" SPF beam	3.50"	Hanger¹	1.50"	1579	1261	1506	4346	See note ¹
2 - Hanger on 11 7/8" SPF beam	3.50"	Hanger¹	1.50"	1579	1261	1506	4346	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)	12' o/c	
Bottom Edge (Lu)	12' 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 - Face Mount Hanger	HHUS48	3.00"	N/A	22-16d	8-16d	
2 - Face Mount Hanger	HHUS48	3.00"	N/A	22-16d	8-16d	

- 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 12' 3 1/2"	N/A	12.1	--	--	
1 - Uniform (PSF)	0 to 12' 7" (Front)	6' 11 1/2"	30.0	20.0	30.0	
2 - Uniform (PSF)	0 to 12' 7" (Front)	1' 1/4"	30.0	60.0	30.0	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

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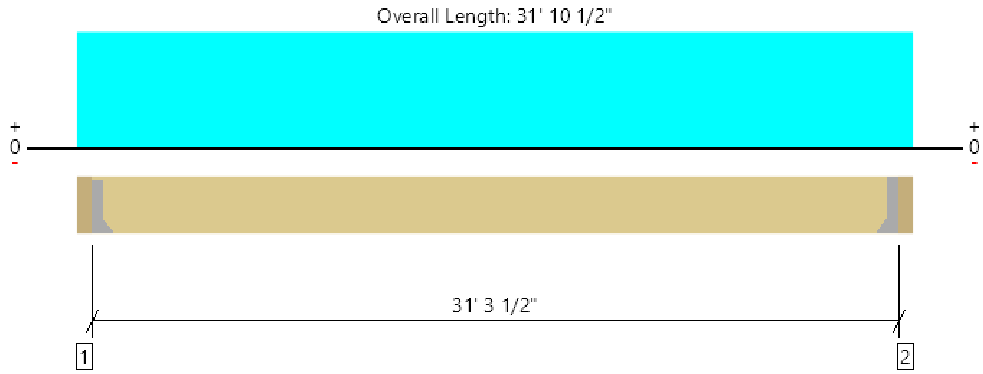
FORTE WEB

MEMBER REPORT

PASSED

Upper Level, B13 Upper Deck: Edge Beam 2 (11-7/8" LVL)
2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL



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)	828 @ 3 1/2"	3938 (1.50")	Passed (21%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	775 @ 1' 3 3/8"	9081	Passed (9%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	6475 @ 15' 11 1/4"	20525	Passed (32%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.508 @ 15' 11 1/4"	0.782	Passed (L/739)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	1.186 @ 15' 11 1/4"	1.565	Passed (L/317)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

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

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 11 7/8" SPF beam	3.50"	Hanger[1]	1.50"	479	193	289	961	See note [1]
2 - Hanger on 11 7/8" SPF beam	3.50"	Hanger[1]	1.50"	479	193	289	961	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)	23' 5" o/c	
Bottom Edge (Lu)	31' 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 - Face Mount Hanger	LUS410	2.00"	N/A	8-10dx1.5	6-10d	
2 - Face Mount Hanger	LUS410	2.00"	N/A	8-10dx1.5	6-10d	

- 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 31' 7"	N/A	12.1	--	--	
1 - Uniform (PSF)	0 to 31' 10 1/2" (Front)	7 1/4"	30.0	20.0	30.0	Default Load

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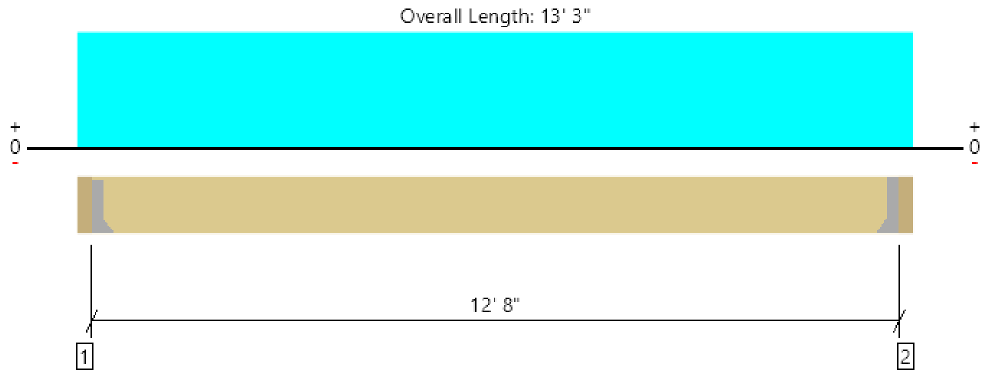
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Weyerhaeuser

Upper Level, B13 Upper Deck: Edge Beam 3 (11-7/8" LVL)
2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL



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)	2001 @ 3 1/2"	3938 (1.50")	Passed (51%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	1688 @ 1' 3 3/8"	9081	Passed (19%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	6335 @ 6' 7 1/2"	20525	Passed (31%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.109 @ 6' 7 1/2"	0.317	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.205 @ 6' 7 1/2"	0.633	Passed (L/742)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

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.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 11 7/8" SPF beam	3.50"	Hanger¹	1.50"	971	596	894	2461	See note ¹
2 - Hanger on 11 7/8" SPF beam	3.50"	Hanger¹	1.50"	971	596	894	2461	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)	12' 8" o/c	
Bottom Edge (Lu)	12' 8" 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 - Face Mount Hanger	LUS414	2.00"	N/A	10-16d	6-16d	
2 - Face Mount Hanger	LUS414	2.00"	N/A	10-16d	6-16d	

- 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 12' 11 1/2"	N/A	12.1	--	--	
1 - Uniform (PSF)	0 to 13' 3" (Front)	4' 6"	30.0	20.0	30.0	Default Load

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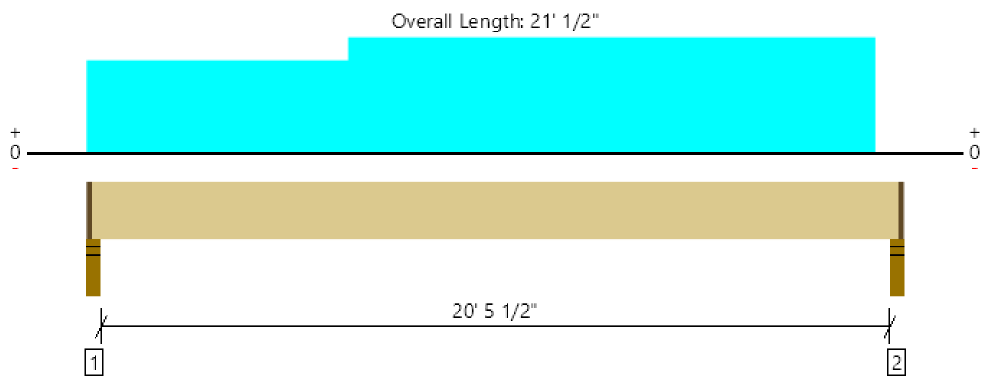
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

Upper Level, B14 Upper Deck: Long Flush Beam (11-7/8" PSL)
1 piece(s) 7" x 11 7/8" 2.2E Parallam® PSL



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)	4276 @ 20' 10 1/2"	6694 (2.25")	Passed (64%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	4015 @ 19' 9 1/8"	18481	Passed (22%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	22892 @ 10' 8 5/16"	45776	Passed (50%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.494 @ 10' 7 1/4"	0.518	Passed (L/503)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.847 @ 10' 6 13/16"	1.035	Passed (L/293)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

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.
- Member should be side-loaded from both sides of the member or braced to prevent rotation.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.50"	1864	1550	1593	5007	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.50"	1756	1875	1485	5116	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)	20' 10" o/c	
Bottom Edge (Lu)	20' 10" 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 20' 11 1/4"	N/A	26.0	--	--	
1 - Uniform (PSF)	0 to 6' 8 3/4" (Front)	5' 5/8"	30.0	20.0	30.0	Default Load
2 - Uniform (PSF)	6' 8 3/4" to 20' 3 3/4" (Front)	5' 5/8"	30.0	40.0	30.0	

Weyerhaeuser Notes

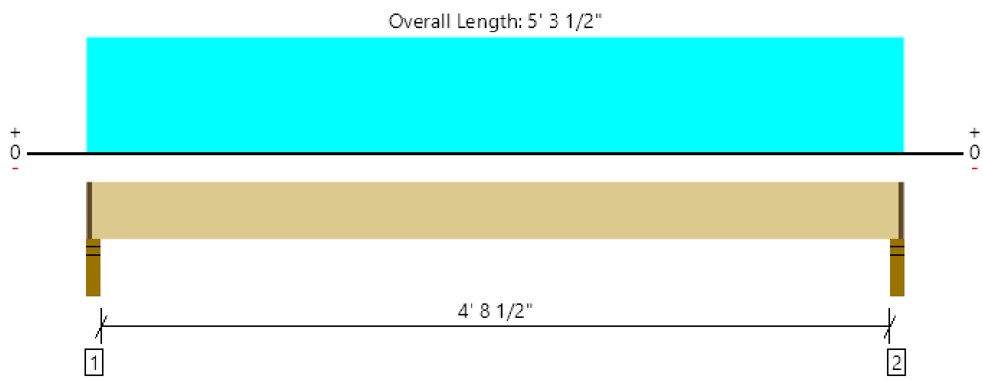
ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

MEMBER REPORT

PASSED

Upper Level, B15 Upper Deck: Short Flush Beam (11-7/8" PSL)

1 piece(s) 3 1/2" x 11 7/8" 2.2E Parallam® PSL



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)	2434 @ 2"	3347 (2.25")	Passed (73%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	1207 @ 1' 3 3/8"	8035	Passed (15%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2719 @ 2' 7 3/4"	19902	Passed (14%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.013 @ 2' 7 3/4"	0.124	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.020 @ 2' 7 3/4"	0.248	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

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.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.64"	802	1538	769	3109	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.64"	802	1538	769	3109	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)	5' 1" o/c	
Bottom Edge (Lu)	5' 1" 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 5' 2 1/4"	N/A	13.0	--	--	
1 - Uniform (PSF)	0 to 5' 3 1/2" (Front)	9' 8 1/4"	30.0	60.0	30.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Wu Fast + Epp (347) 435-2377 bwu@fastepp.com	

2.2 | STEEL FRAMING DESIGN

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Perimeter Beam Loading	**DATE:**	2021-03-02
DESIGN BY:	BJW		

NOTES: Main level west perimeter beam (B8)

GEOMETRY:

Tributary width	w_T =	2.48	ft
Beam length	L1 =	27.79	ft
Beam length	L2 =	11.67	ft

SURFACE LOADS:

Dead load	DL =	0	psf
Superimposed dead load	SDL =	30	psf
Live load	LL =	60	psf
Snow load	SL =	30	psf

LINE LOADS:

Dead load	DL =	**0**	plf	**0.00**	klf
Superimposed dead load	SDL =	**74.375**	plf	**0.07**	klf
Live load	LL =	**148.750**	plf	**0.15**	klf
Snow load	SL =	**74.375**	plf	**0.07**	klf

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Perimeter Beam Loading	**DATE:**	2021-03-02
DESIGN BY:	BJW		

NOTES: Main level south perimeter beam (B4) point load on B8

GEOMETRY:

Tributary width	w_T =	5.58	ft
Beam overhang	L =	10.31	ft

SURFACE LOADS:

Dead load	DL =	0	psf
Superimposed dead load	SDL =	30	psf
Live load	LL =	40	psf
Snow load	SL =	30	psf

LINE LOADS:

Dead load	DL =	**0**	plf	**0.00**	klf
Superimposed dead load	SDL =	**167.5**	plf	**0.17**	klf
Live load	LL =	**223.333**	plf	**0.22**	klf
Snow load	SL =	**167.500**	plf	**0.17**	klf

REACTIONS:

	RDL =	**0.00**	kips
Overhang reaction	RSDL =	**1.73**	kips
	RLL =	**2.30**	kips
	RSL =	**1.73**	kips

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Perimeter Beam Loading	**DATE:**	2021-03-02
DESIGN BY:	BJW		

NOTES: Upper level deck west perimeter beam (B8)

GEOMETRY:

Tributary width	w_T =	2.67	ft
Beam length	L1 =	27.79	ft
Beam length	L2 =	12.40	ft

SURFACE LOADS:

Dead load	DL =	0	psf
Superimposed dead load	SDL =	30	psf
Live load	LL =	20	psf
Snow load	SL =	30	psf

LINE LOADS:

Dead load	DL =	**0**	plf	**0.00**	klf
Superimposed dead load	SDL =	**80**	plf	**0.08**	klf
Live load	LL =	**53.333**	plf	**0.05**	klf
Snow load	SL =	**80.000**	plf	**0.08**	klf

REACTIONS:

	RDL =	**0.00**	kips
Girder reaction	RSDL =	**1.11**	kips
	RLL =	**0.74**	kips
	RSL =	**1.11**	kips

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Perimeter Beam Loading	**DATE:**	2021-03-02
DESIGN BY:	BJW		

NOTES: Upper level south perimeter beam (B4) point load on B8

GEOMETRY:

Tributary width	w_T =	1.90	ft
Beam overhang	L =	10.31	ft

SURFACE LOADS:

Dead load	DL =	0	psf
Superimposed dead load	SDL =	30	psf
Live load	LL =	40	psf
Snow load	SL =	30	psf

LINE LOADS:

Dead load	DL =	**0**	plf	**0.00**	klf
Superimposed dead load	SDL =	**57**	plf	**0.06**	klf
Live load	LL =	**76.000**	plf	**0.08**	klf
Snow load	SL =	**57.000**	plf	**0.06**	klf

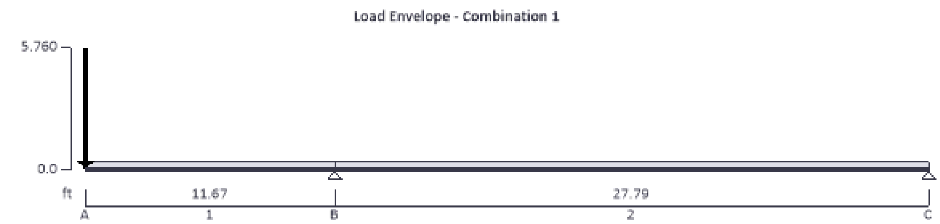
REACTIONS:

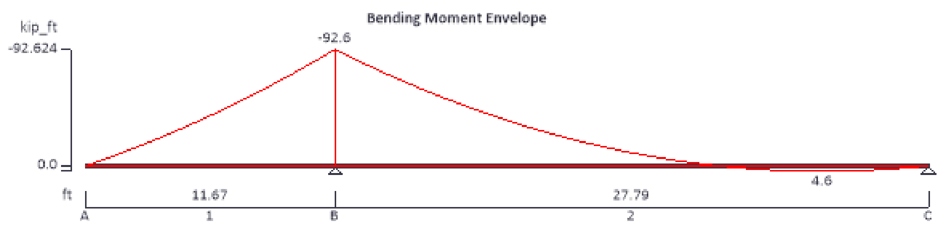
	RDL =	**0.00**	kips
Overhang reaction	RSDL =	**0.59**	kips
	RLL =	**0.78**	kips
	RSL =	**0.59**	kips

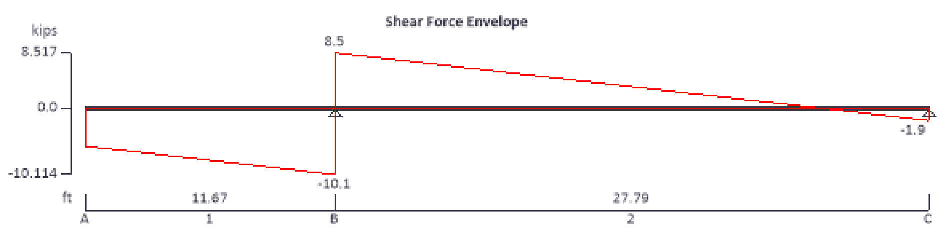
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

In accordance with AISC360-16 using the ASD method

Tedds calculation version 3.0.15







Support conditions

Support A	Vertically free
	Rotationally free
Support B	Vertically restrained
	Rotationally free
Support C	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam * 1
	Dead full UDL 0.11 kips/ft
	Live full UDL 0.15 kips/ft
	Snow full UDL 0.06 kips/ft
	Dead point load 1.73 kips at 0.00 in
	Live point load 2.3 kips at 0.00 in
	Snow point load 1.73 kips at 0.00 in

Load combinations

Load combination 1	Support A	Dead * 1.00
		Live * 1.00
		Roof live * 1.00
		Snow * 1.00
		Dead * 1.00
		Live * 1.00
		Roof live * 1.00
		Snow * 1.00
	Support B	Dead * 1.00
		Live * 1.00
		Roof live * 1.00
		Snow * 1.00
		Dead * 1.00
		Live * 1.00
		Roof live * 1.00
		Snow * 1.00
	Support C	Dead * 1.00
		Live * 1.00
		Roof live * 1.00
		Snow * 1.00

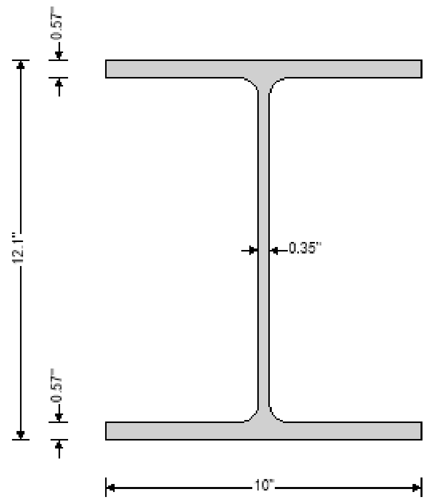
Analysis results

Maximum moment	M_{max} = **4.6** kips_ft	M_{min} = **-92.6** kips_ft
Maximum moment span 1	M_{s1_max} = **0** kips_ft	M_{s1_min} = **-92.6** kips_ft
Maximum moment span 2	M_{s2_max} = **4.6** kips_ft	M_{s2_min} = **-92.6** kips_ft
Maximum shear	V_{max} = **8.5** kips	V_{min} = **-10.1** kips
Maximum shear span 1	V_{s1_max} = **-5.8** kips	V_{s1_min} = **-10.1** kips
Maximum shear span 2	V_{s2_max} = **8.5** kips	V_{s2_min} = **-1.9** kips
Deflection	δ_{max} = **0.6** in	δ_{min} = **0.1** in
Deflection span 1	δ_{s1_max} = **0.6** in	δ_{s1_min} = **0** in
Deflection span 2	δ_{s2_max} = **0** in	δ_{s2_min} = **0.1** in
Maximum reaction at support A	R_{A_max} = **0** kips	R_{A_min} = **0** kips
Maximum reaction at support B	R_{B_max} = **18.6** kips	R_{B_min} = **18.6** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **7** kips	
Unfactored live load reaction at support B	R_{B_Live} = **7.5** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **4.1** kips	
Maximum reaction at support C	R_{C_max} = **1.9** kips	R_{C_min} = **1.9** kips
Unfactored dead load reaction at support C	R_{C_Dead} = **1.1** kips	
Unfactored live load reaction at support C	R_{C_Live} = **0.8** kips	
Unfactored snow load reaction at support C	R_{C_Snow} = **0** kips	

Section details

Section type	**W 12x53 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi

Modulus of elasticity E = **29000** ksi



Safety factors

Safety factor for tensile yielding	Ω_{ty} = **1.67**
Safety factor for tensile rupture	Ω_{tr} = **2.00**
Safety factor for compression	Ω_{c} = **1.67**
Safety factor for flexure	Ω_{b} = **1.67**

Lateral bracing

Span 1 has continuous lateral bracing
Span 2 has continuous lateral bracing
Cantilever tip is unbraced
Cantilever support is continuous with lateral and torsional restraint

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio	$b_f / (2 * t_f)$ = **8.70**	
Limiting ratio for compact section	$\lambda_{pff} = 0.38 * \sqrt{[E / F_y]}$ = **9.15**	
Limiting ratio for non-compact section	$\lambda_{rff} = 1.0 * \sqrt{[E / F_y]}$ = **24.08**	Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio	$(d - 2 * k) / t_w$ = **28.23**	
Limiting ratio for compact section	$\lambda_{pwf} = 3.76 * \sqrt{[E / F_y]}$ = **90.55**	
Limiting ratio for non-compact section	$\lambda_{rwf} = 5.70 * \sqrt{[E / F_y]}$ = **137.27**	Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength	$V_r = \max(\mathrm{abs}(V_{max}), \mathrm{abs}(V_{min}))$ = **10.114** kips
Web area	$A_w = d * t_w$ = **4.174** in^2
Web plate buckling coefficient	k_v = **5.34**
Web shear coefficient - eq G2-3	C_{v1} = **1**
Nominal shear strength – eq G6-1	$V_n = 0.6 * F_y * A_w * C_{v1}$ = **125.235** kips
Safety factor for shear	Ω_v = **1.50**

Allowable shear strength $V_c = V_n / \Omega_v =$ **83.490** kips

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure in the major axis at span 1 - Chapter F

Required flexural strength $M_r = max(abs(M_{s1\_max}), abs(M_{s1\_min})) =$ **92.624** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y * Z_x =$ **324.583** kips_ft

Nominal flexural strength $M_n = M_{nyld} =$ **324.583** kips_ft

Allowable flexural strength $M_c = M_n / \Omega_b =$ **194.361** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to live loads

Limiting deflection $\delta_{lim} = 2 * L_{s1} / 360 =$ **0.778** in

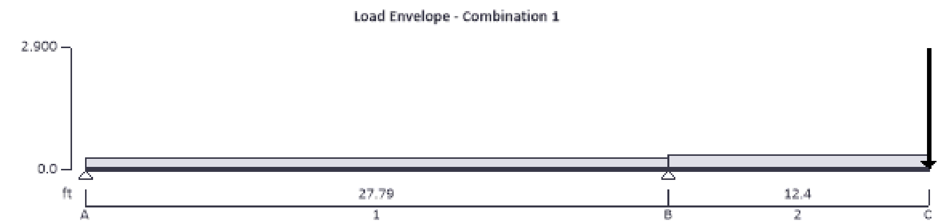
Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **0.562** in

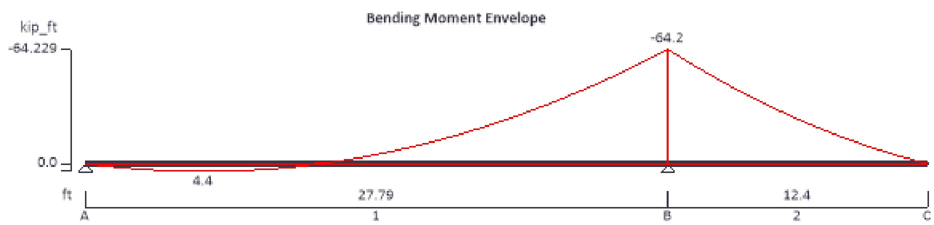
PASS - Maximum deflection does not exceed deflection limit

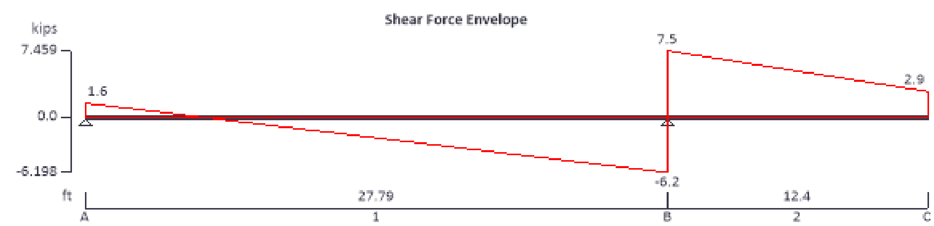
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

In accordance with AISC360-16 using the LRFD method

Tedds calculation version 3.0.15







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free
Support C	Vertically free
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam * 1
	Dead full UDL 0.08 kips/ft
	Live full UDL 0.05 kips/ft
	Snow full UDL 0.08 kips/ft
	Dead point load 0.59 kips at 482.28 in
	Live point load 0.78 kips at 482.28 in
	Snow point load 0.59 kips at 482.28 in

Load combinations

Load combination 1	Support A	Dead * 1.20
		Live * 1.60
		Snow * 0.50
		Dead * 1.20
		Live * 1.60
		Snow * 0.50
	Support B	Dead * 1.20
		Live * 1.60
		Snow * 0.50
		Dead * 1.20
		Live * 1.60
		Roof live * 1.60
		Snow * 1.60
	Support C	Dead * 1.20
		Live * 1.60
		Roof live * 1.60
		Snow * 1.60

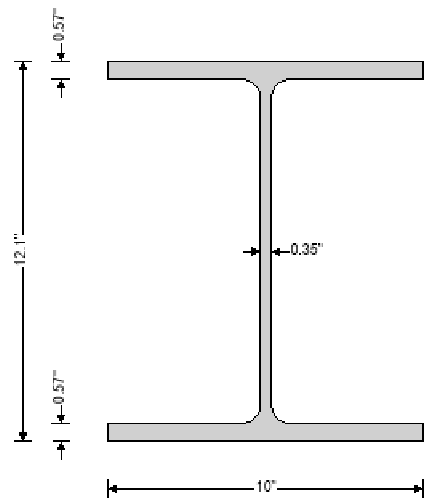
Analysis results

Maximum moment	M_{max} = **4.4** kips_ft	M_{min} = **-64.2** kips_ft
Maximum moment span 1	M_{s1_max} = **4.4** kips_ft	M_{s1_min} = **-64.2** kips_ft
Maximum moment span 2	M_{s2_max} = **0** kips_ft	M_{s2_min} = **-64.2** kips_ft
Maximum shear	V_{max} = **7.5** kips	V_{min} = **-6.2** kips
Maximum shear span 1	V_{s1_max} = **1.6** kips	V_{s1_min} = **-6.2** kips
Maximum shear span 2	V_{s2_max} = **7.5** kips	V_{s2_min} = **2.9** kips
Deflection	δ_{max} = **0.6** in	δ_{min} = **0.1** in
Deflection span 1	δ_{s1_max} = **0** in	δ_{s1_min} = **0.1** in
Deflection span 2	δ_{s2_max} = **0.6** in	δ_{s2_min} = **0** in
Maximum reaction at support A	R_{A_max} = **1.6** kips	R_{A_min} = **1.6** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **1.2** kips	
Unfactored live load reaction at support A	R_{A_Live} = **0.2** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **0.6** kips	
Maximum reaction at support B	R_{B_max} = **13.7** kips	R_{B_min} = **13.7** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **4.7** kips	
Unfactored live load reaction at support B	R_{B_Live} = **2.6** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **3.2** kips	
Maximum reaction at support C	R_{C_max} = **0** kips	R_{C_min} = **0** kips

Section details

Section type	**W 12x53 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi

Tekla Tedds Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsky Residence			Job Ref. 8119	
	Section Upper Level Deck West Perimeter Beam (B8)			Sheet no./rev. 3	
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date



Resistance factors

Resistance factor for tensile yielding ϕ_{ty} = **0.90**

Resistance factor for tensile rupture ϕ_{tr} = **0.75**

Resistance factor for compression ϕ_c = **0.90**

Resistance factor for flexure ϕ_b = **0.90**

Lateral bracing

Span 1 has continuous lateral bracing

Span 2 has continuous lateral bracing

Cantilever tip is unbraced

Cantilever support is continuous with lateral and torsional restraint

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio $b_f / (2 * t_f)$ = **8.70**

Limiting ratio for compact section $\lambda_{pff} = 0.38 * \sqrt{[E / F_y]}$ = **9.15**

Limiting ratio for non-compact section $\lambda_{rff} = 1.0 * \sqrt{[E / F_y]}$ = **24.08** Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio $(d - 2 * k) / t_w$ = **28.23**

Limiting ratio for compact section $\lambda_{pwf} = 3.76 * \sqrt{[E / F_y]}$ = **90.55**

Limiting ratio for non-compact section $\lambda_{rwf} = 5.70 * \sqrt{[E / F_y]}$ = **137.27** Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **7.459** kips

Web area $A_w = d * t_w$ = **4.174** in^2

Web plate buckling coefficient k_v = **5.34**

Web shear coefficient - eq G2-3 C_{v1} = **1**

Nominal shear strength – eq G6-1 $V_n = 0.6 * F_y * A_w * C_{v1}$ = **125.235** kips

Resistance factor for shear ϕ_v = **1.00**

Design shear strength $V_c = \phi_v * V_n$ = **125.235** kips

PASS - Design shear strength exceeds required shear strength

Design of members for flexure in the major axis at span 1 - Chapter F

Required flexural strength $M_r = max(abs(M_{s1\_max}), abs(M_{s1\_min})) =$ **64.229** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y * Z_x =$ **324.583** kips_ft

Nominal flexural strength $M_n = M_{nyld} =$ **324.583** kips_ft

Design flexural strength $M_c = \phi_b * M_n =$ **292.125** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

Limiting deflection $\delta_{lim} = 2 * L_{s2} / 240 =$ **1.24** in

Maximum deflection span 2 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **0.592** in

PASS - Maximum deflection does not exceed deflection limit

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Master Suite Transfer Beam 1	**DATE:**	2021-03-02
DESIGN BY:	BJW		

NOTES: B9 - UPPER LEVEL

GEOMETRY:

Tributary width	w_T =	8.677	ft
Beam length	L =	23.73	ft

SURFACE LOADS:

Dead load	DL =	0	psf
Superimposed dead load	SDL =	30	psf
Live load	LL =	40	psf
Snow load	SL =	0	psf

LINE LOADS:

Dead load	DL =	**0**	plf	**0**	klf
Superimposed dead load	SDL =	**260.313**	plf	**0.260**	klf
Live load	LL =	**347.083**	plf	**0.347**	klf
Snow load	SL =	**0**	plf	**0**	klf

REACTIONS:

	RDL =	**0.00**	kips
Girder reaction	RSDL =	**3.09**	kips
	RLL =	**4.12**	kips
	RSL =	**0.00**	kips

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Master Suite Transfer Beam 1	**DATE:**	2021-03-02
DESIGN BY:	BJW		

NOTES: B9 - UPPER LEVEL DECK

GEOMETRY:

Tributary width	w_T =	10.135	ft
Beam length	L =	23.73	ft

SURFACE LOADS:

Dead load	DL =	0	psf				
Superimposed dead load	SDL =	30	psf				
Live load	LL avg =	**46.9**	psf	Deck =	60	Ballast =	20
Snow load	SL =	30	psf	Distance =	**16.0**	Distance =	7.75

LINE LOADS:

Dead load	DL =	**0.0**	plf	**0.0**	klf
Superimposed dead load	SDL =	**304.1**	plf	**0.3**	klf
Live load	LL =	**475.7**	plf	**0.5**	klf
Snow load	SL =	**304.1**	plf	**0.3**	klf

REACTIONS:

	RDL =	**0.00**	kips
Girder reaction	RSDL =	**3.61**	kips
	RLL =	**5.64**	kips
	RSL =	**3.61**	kips

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Master Suite Transfer Beam 1	**DATE:**	2021-03-02
DESIGN BY:	BJW		

NOTES: B9 - HIGH ROOF

GEOMETRY:

Tributary width	w_T =	8.677	ft
Beam length	L =	23.73	ft

SURFACE LOADS:

Dead load	DL =	0	psf
Superimposed dead load	SDL =	15	psf
Live load	LL =	20	psf
Snow load	SL =	25	psf

LINE LOADS:

Dead load	DL =	**0**	plf	**0**	klf
Superimposed dead load	SDL =	**130.156**	plf	**0.130**	klf
Live load	LL =	**173.542**	plf	**0.174**	klf
Snow load	SL =	**216.927**	plf	**0.217**	klf

REACTIONS:

	RDL =	**0.00**	kips
Girder reaction	RSDL =	**1.54**	kips
	RLL =	**2.06**	kips
	RSL =	**2.57**	kips

	Line Load Total	
SDL	**0.695**	klf
LL	**0.823**	klf
RL	**0.174**	klf
SL	**0.521**	klf

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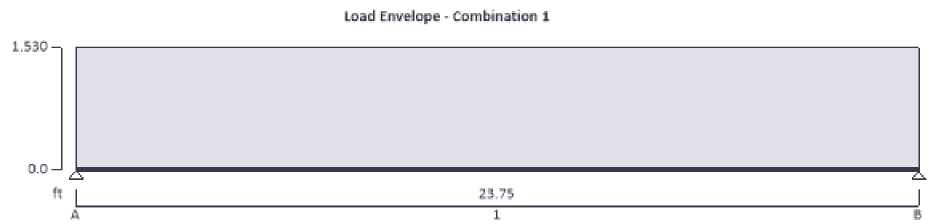
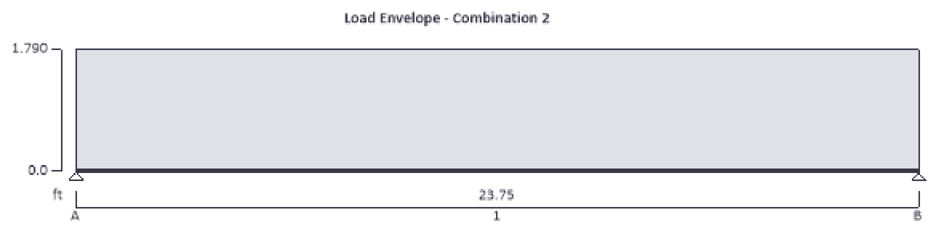
	Section Upper Level Transfer Beam 1 (B9)	Sheet no./rev. 1
	Calc. by: BJW · Date: 3/2/2021 · Chk'd by: · Date:	App'd by: · Date:

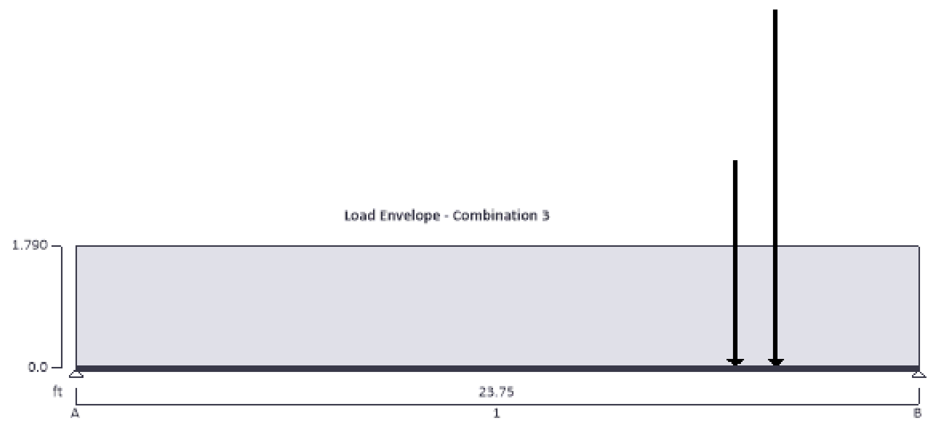
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

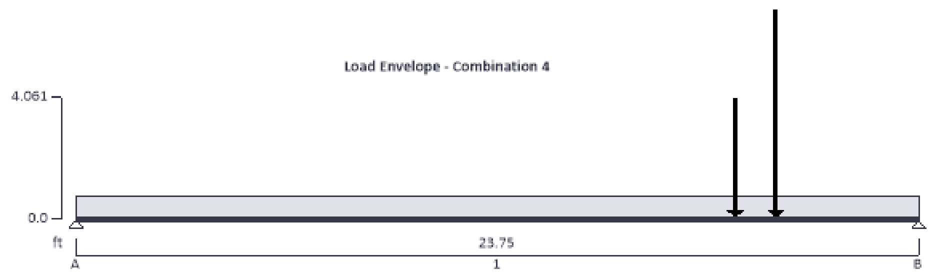
In accordance with AISC360-16 using the ASD method

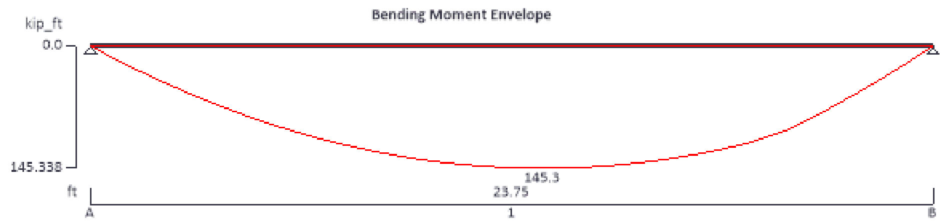
Tedds calculation version 3.0.15

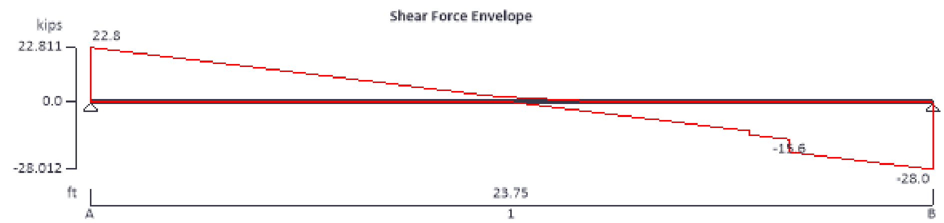












Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam * 1
	Dead full UDL 0.695 kips/ft
	Live full UDL 0.823 kips/ft
	Roof live full UDL 0.174 kips/ft
	Snow full UDL 0.521 kips/ft
	Seismic point load 5.802 kips at 223.00 in
	Seismic point load 10.022 kips at 236.75 in

Load combinations

Load combination 1 - D+0.75L+0.75Lr	Support A	Dead * 1.00
		Live * 0.75
		Roof live * 0.75
		Dead * 1.00
		Live * 0.75
		Roof live * 0.75
	Support B	Dead * 1.00
		Live * 0.75
		Roof live * 0.75
Load combination 2 - D+0.75L+0.75S	Support A	Dead * 1.00
		Live * 0.75
		Snow * 0.75

		Dead * 1.00
		Live * 0.75
		Snow * 0.75
	Support B	Dead * 1.00
		Live * 0.75
		Snow * 0.75
Load combination 3 - D+0.75L+0.525E+0.75S	Support A	Dead * 1.00
		Live * 0.75
		Snow * 0.75
		Seismic * 0.53
		Dead * 1.00
		Live * 0.75
		Snow * 0.75
		Seismic * 0.53
	Support B	Dead * 1.00
		Live * 0.75
		Snow * 0.75
		Seismic * 0.53
Load combination 4 - D+0.7E	Support A	Dead * 1.00
		Seismic * 0.70
		Dead * 1.00
		Seismic * 0.70
	Support B	Dead * 1.00
		Seismic * 0.70

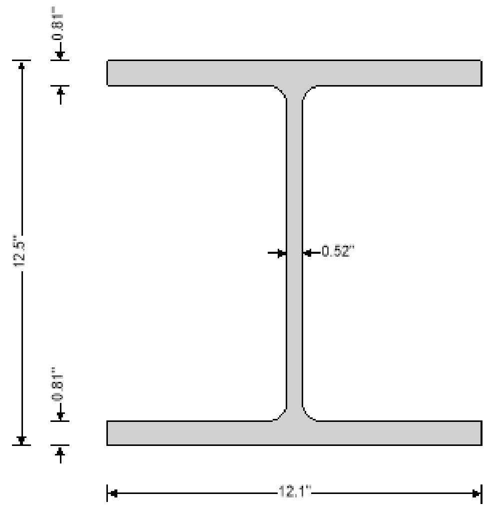
Analysis results

Maximum moment	M_{max} = **145.3** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **22.8** kips	V_{min} = **-28** kips
Deflection	δ_{max} = **0.8** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **22.8** kips	R_{A_min} = **11.4** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **9.3** kips	
Unfactored live load reaction at support A	R_{A_Live} = **9.8** kips	
Unfactored roof live load reaction at support A	$R_{A\_Roof\ live}$ = **2.1** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **6.2** kips	
Unfactored seismic load reaction at support A	$R_{A\_Seismic}$ = **3** kips	
Maximum reaction at support B	R_{B_max} = **28** kips	R_{B_min} = **18.2** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **9.3** kips	
Unfactored live load reaction at support B	R_{B_Live} = **9.8** kips	
Unfactored roof live load reaction at support B	$R_{B\_Roof\ live}$ = **2.1** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **6.2** kips	
Unfactored seismic load reaction at support B	$R_{B\_Seismic}$ = **12.9** kips	

Section details

Section type	**W 12x87 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi

Modulus of elasticity $E = \mathbf{29000}$ ksi



Safety factors

Safety factor for tensile yielding $\Omega_{ty} = \mathbf{1.67}$

Safety factor for tensile rupture $\Omega_{tr} = \mathbf{2.00}$

Safety factor for compression $\Omega_c = \mathbf{1.67}$

Safety factor for flexure $\Omega_b = \mathbf{1.67}$

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio $b_f / (2 * t_f) = \mathbf{7.47}$

Limiting ratio for compact section $\lambda_{pff} = 0.38 * \sqrt{[E / F_y]} = \mathbf{9.15}$

Limiting ratio for non-compact section $\lambda_{rff} = 1.0 * \sqrt{[E / F_y]} = \mathbf{24.08}$ Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio $(d - 2 * k) / t_w = \mathbf{18.80}$

Limiting ratio for compact section $\lambda_{pwf} = 3.76 * \sqrt{[E / F_y]} = \mathbf{90.55}$

Limiting ratio for non-compact section $\lambda_{rwf} = 5.70 * \sqrt{[E / F_y]} = \mathbf{137.27}$ Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{28.012}$ kips

Web area $A_w = d * t_w = \mathbf{6.438}$ in^2

Web plate buckling coefficient $k_v = \mathbf{5.34}$

Web shear coefficient - eq G2-3 $C_{v1} = \mathbf{1}$

Nominal shear strength – eq G6-1 $V_n = 0.6 * F_y * A_w * C_{v1} = \mathbf{193.125}$ kips

Safety factor for shear $\Omega_v = \mathbf{1.50}$

Allowable shear strength $V_c = V_n / \Omega_v = \mathbf{128.750}$ kips

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure in the major axis - Chapter F

Required flexural strength $M_r = max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **145.338** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y * Z_x$ = **550** kips_ft

Nominal flexural strength $M_n = M_{nyld}$ = **550.000** kips_ft

Allowable flexural strength $M_c = M_n / \Omega_b$ = **329.341** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 240$ = **1.188** in

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min}))$ = **0.767** in

PASS - Maximum deflection does not exceed deflection limit

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Master Suite Transfer Beam 2	**DATE:**	2021-03-02
DESIGN BY:	BJW		

NOTES: B10 - UPPER LEVEL

GEOMETRY:

				Trib 1	Trib 2
Tributary width	w_T =	**8.713**	ft	9.76	7.61
Beam length	L =	30.604	ft	15.67	**14.938**

SURFACE LOADS:

Dead load	DL =	0	psf
Superimposed dead load	SDL =	30	psf
Live load	LL =	40	psf
Snow load	SL =	0	psf

LINE LOADS:

Dead load	DL =	**0**	plf	**0**	klf
Superimposed dead load	SDL =	**261.392**	plf	**0.261**	klf
Live load	LL =	**348.523**	plf	**0.349**	klf
Snow load	SL =	**0**	plf	**0**	klf

REACTIONS:

	RDL =	**0.00**	kips
Girder reaction	RSDL =	**4.00**	kips
	RLL =	**5.33**	kips
	RSL =	**0.00**	kips

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Master Suite Transfer Beam 2	**DATE:**	2021-03-02
DESIGN BY:	BJW		

NOTES: B10 - UPPER LEVEL DECK

GEOMETRY:

Tributary width	w_T =	7.177	ft
Beam length	L =	30.604	ft

SURFACE LOADS:

Dead load	DL =	0	psf				
Superimposed dead load	SDL =	30	psf				
Live load	LL avg =	**50.7**	psf	Deck =	60	Ballast =	20
Snow load	SL =	30	psf	Distance =	**23.5**	Distance =	7.08

LINE LOADS:

Dead load	DL =	**0.0**	plf	**0.0**	klf
Superimposed dead load	SDL =	**215.3**	plf	**0.2**	klf
Live load	LL =	**364.2**	plf	**0.4**	klf
Snow load	SL =	**215.3**	plf	**0.2**	klf

REACTIONS:

	RDL =	**0.00**	kips
Girder reaction	RSDL =	**3.29**	kips
	RLL =	**5.57**	kips
	RSL =	**3.29**	kips

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Master Suite Transfer Beam 2	**DATE:**	2021-03-02
DESIGN BY:	BJW		

NOTES: B10 - HIGH ROOF

GEOMETRY:

				Trib 1	Trib 2
Tributary width	w_T =	**8.713**	ft	9.76	7.61
Beam length	L =	30.604	ft	15.67	**14.938**

SURFACE LOADS:

Dead load	DL =	0	psf
Superimposed dead load	SDL =	15	psf
Live load	LL =	20	psf
Snow load	SL =	30	psf

LINE LOADS:

Dead load	DL =	**0**	plf	**0**	klf
Superimposed dead load	SDL =	**130.696**	plf	**0.131**	klf
Live load	LL =	**174.261**	plf	**0.174**	klf
Snow load	SL =	**261.392**	plf	**0.261**	klf

REACTIONS:

	RDL =	**0.00**	kips
Girder reaction	RSDL =	**2.00**	kips
	RLL =	**2.67**	kips
	RSL =	**4.00**	kips

	Line Load Total	
SDL	**0.607**	klf
LL	**0.713**	klf
RL	**0.174**	klf
SL	**0.477**	klf

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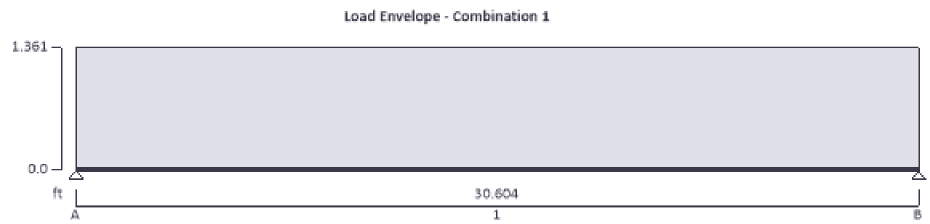
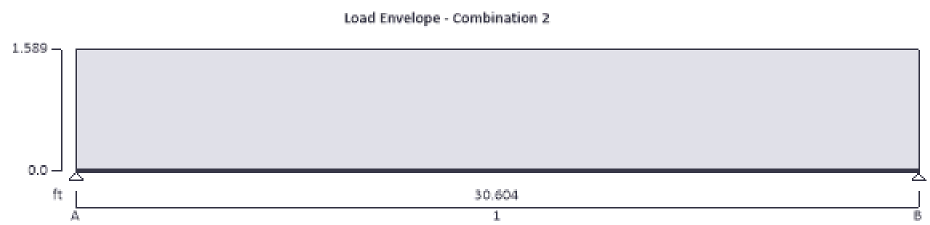
	Section Upper Level Transfer Beam 2 (B10)				Sheet no./rev. 1	
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date

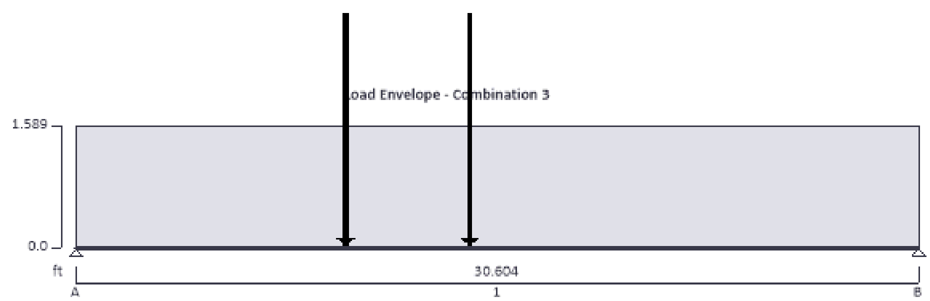
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

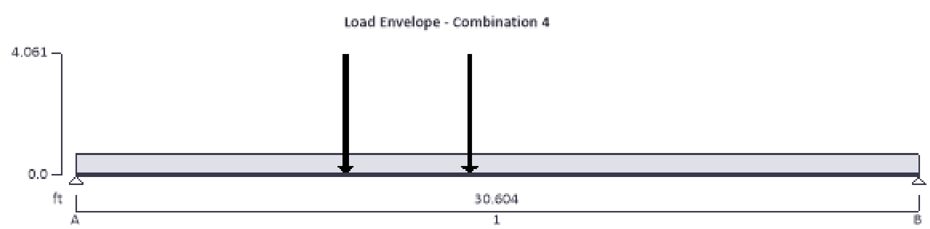
In accordance with AISC360-16 using the ASD method

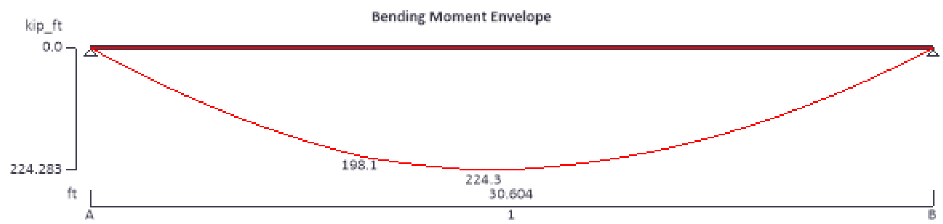
Tedds calculation version 3.0.15

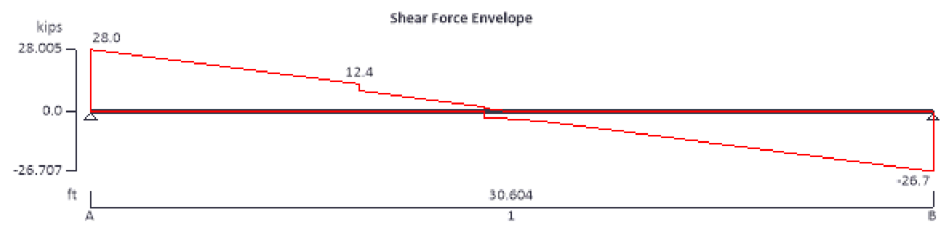












Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam * 1
	Dead full UDL 0.607 kips/ft
	Live full UDL 0.713 kips/ft
	Roof live full UDL 0.174 kips/ft
	Snow full UDL 0.477 kips/ft
	Seismic point load 5.802 kips at 117.50 in
	Seismic point load 5.802 kips at 171.50 in

Load combinations

Load combination 1 - D+0.75L+0.75Lr	Support A	Dead * 1.00
		Live * 0.75
		Roof live * 0.75
		Dead * 1.00
		Live * 0.75
		Roof live * 0.75
	Support B	Dead * 1.00
		Live * 0.75
		Roof live * 0.75
Load combination 2 - D+0.75L+0.75S	Support A	Dead * 1.00
		Live * 0.75
		Snow * 0.75
		Dead * 1.00

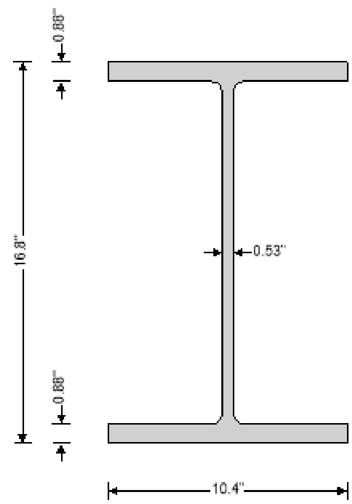
		Live * 0.75
		Snow * 0.75
	Support B	Dead * 1.00
		Live * 0.75
		Snow * 0.75
Load combination 3 - D+0.75L+0.525E+0.75S	Support A	Dead * 1.00
		Live * 0.75
		Snow * 0.75
		Seismic * 0.53
		Dead * 1.00
		Live * 0.75
		Snow * 0.75
		Seismic * 0.53
	Support B	Dead * 1.00
		Live * 0.75
		Snow * 0.75
		Seismic * 0.53
Load combination 4 - D+0.7E	Support A	Dead * 1.00
		Seismic * 0.70
		Dead * 1.00
		Seismic * 0.70
	Support B	Dead * 1.00
		Seismic * 0.70

Analysis results

Maximum moment	M_{max} = **224.3** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **28** kips	V_{min} = **-26.7** kips
Deflection	δ_{max} = **1.1** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **28** kips	R_{A_min} = **15.6** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **10.7** kips	
Unfactored live load reaction at support A	R_{A_Live} = **10.9** kips	
Unfactored roof live load reaction at support A	$R_{A\_Roof live}$ = **2.7** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **7.3** kips	
Unfactored seismic load reaction at support A	$R_{A\_Seismic}$ = **7** kips	
Maximum reaction at support B	R_{B_max} = **26.7** kips	R_{B_min} = **13.8** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **10.7** kips	
Unfactored live load reaction at support B	R_{B_Live} = **10.9** kips	
Unfactored roof live load reaction at support B	$R_{B\_Roof live}$ = **2.7** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **7.3** kips	
Unfactored seismic load reaction at support B	$R_{B\_Seismic}$ = **4.6** kips	

Section details

Section type	**W 16x89 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



Safety factors

Safety factor for tensile yielding Ω_{ty} = **1.67**

Safety factor for tensile rupture Ω_{tr} = **2.00**

Safety factor for compression Ω_c = **1.67**

Safety factor for flexure Ω_b = **1.67**

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio $b_f / (2 * t_f)$ = **5.94**

Limiting ratio for compact section $\lambda_{pff} = 0.38 * \sqrt{[E / F_y]}$ = **9.15**

Limiting ratio for non-compact section $\lambda_{rff} = 1.0 * \sqrt{[E / F_y]}$ = **24.08** Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio $(d - 2 * k) / t_w$ = **27.12**

Limiting ratio for compact section $\lambda_{pwf} = 3.76 * \sqrt{[E / F_y]}$ = **90.55**

Limiting ratio for non-compact section $\lambda_{rwf} = 5.70 * \sqrt{[E / F_y]}$ = **137.27** Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **28.005** kips

Web area $A_w = d * t_w$ = **8.82** in^2

Web plate buckling coefficient k_v = **5.34**

Web shear coefficient - eq G2-3 C_{v1} = **1**

Nominal shear strength – eq G6-1 $V_n = 0.6 * F_y * A_w * C_{v1}$ = **264.600** kips

Safety factor for shear Ω_v = **1.50**

Allowable shear strength $V_c = V_n / \Omega_v$ = **176.400** kips

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure in the major axis - Chapter F

Required flexural strength $M_r = max(abs(M_{s1\_max}), abs(M_{s1\_min})) =$ **224.283** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y * Z_x =$ **729.167** kips_ft

Nominal flexural strength $M_n = M_{nyld} =$ **729.167** kips_ft

Allowable flexural strength $M_c = M_n / \Omega_b =$ **436.627** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 240 =$ **1.53** in

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **1.079** in

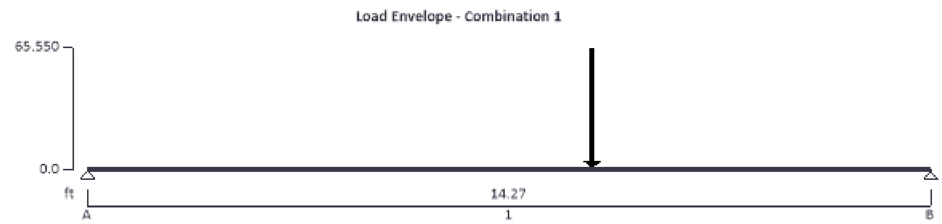
PASS - Maximum deflection does not exceed deflection limit

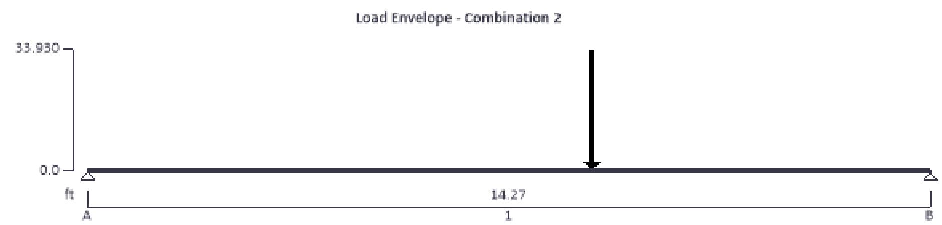
Tekla Tedds Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsky Residence				Job Ref. 8119	
	Section Upper Level Transfer Beam 3 (B11)				Sheet no./rev. 1	
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date

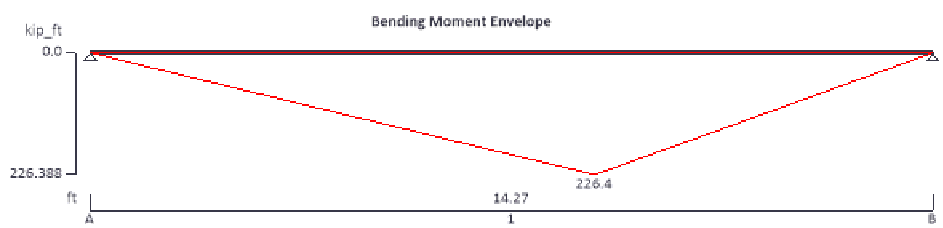
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

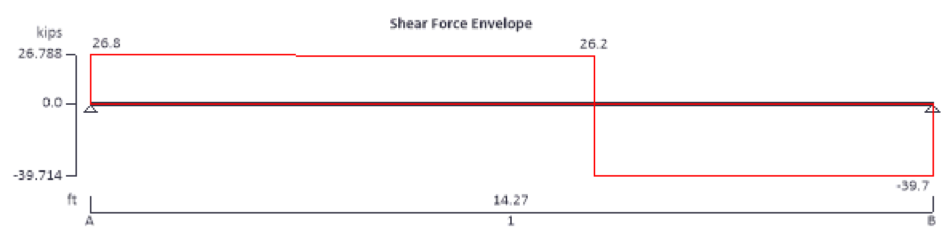
In accordance with AISC360-16 using the ASD method

Tedds calculation version 3.0.15









Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam * 1	
POINT LOADS FROM B9 AND B10 (SEE RESPECTIVE TEDDS OUTPUT)	Dead point load 20 kips at 102.50 in	
	Live point load 20.7 kips at 102.50 in	
	Roof live point load 4.8 kips at 102.50 in	
	Snow point load 13.5 kips at 102.50 in	
	Seismic point load 19.9 kips at 102.50 in	

Load combinations

Load combination 1 - D+0.75L+0.525E+0.75S	Support A	Dead * 1.00
		Live * 0.75
		Snow * 0.75
		Seismic * 1.00
		Dead * 1.00
		Live * 0.75
		Snow * 0.75
		Seismic * 1.00
	Support B	Dead * 1.00
		Live * 0.75
		Snow * 0.75
		Seismic * 1.00
Load combination 2 - D+0.7E	Support A	Dead * 1.00
		Seismic * 0.70
		Dead * 1.00
		Seismic * 0.70
	Support B	Dead * 1.00
		Seismic * 0.70

Analysis results

Maximum moment	M_{max} = **226.4** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **26.8** kips	V_{min} = **-39.7** kips
Deflection	δ_{max} = **0.2** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **26.8** kips	R_{A_min} = **14.1** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **8.5** kips	
Unfactored live load reaction at support A	R_{A_Live} = **8.3** kips	
Unfactored roof live load reaction at support A	$R_{A\_Roof\ live}$ = **1.9** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **5.4** kips	
Unfactored seismic load reaction at support A	$R_{A\_Seismic}$ = **8** kips	
Maximum reaction at support B	R_{B_max} = **39.7** kips	R_{B_min} = **20.8** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **12.4** kips	
Unfactored live load reaction at support B	R_{B_Live} = **12.4** kips	
Unfactored roof live load reaction at support B	$R_{B\_Roof\ live}$ = **2.9** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **8.1** kips	
Unfactored seismic load reaction at support B	$R_{B\_Seismic}$ = **11.9** kips	

Section details

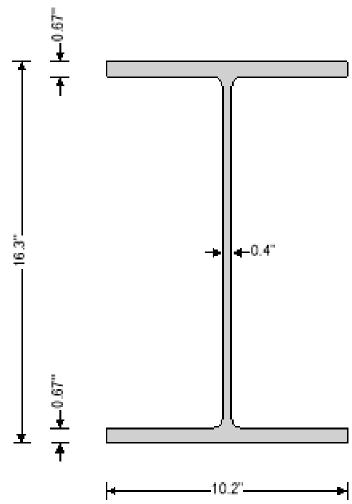
Section type **W 16x67 (AISC 15th Edn (v15.0))**

ASTM steel designation **A992**

Steel yield stress F_y = **50** ksi

Steel tensile stress F_u = **65** ksi

Modulus of elasticity E = **29000** ksi



Safety factors

Safety factor for tensile yielding Ω_{ty} = **1.67**

Safety factor for tensile rupture Ω_{tr} = **2.00**

Safety factor for compression Ω_c = **1.67**

Safety factor for flexure Ω_b = **1.67**

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio $b_f / (2 * t_f)$ = **7.67**

Limiting ratio for compact section $\lambda_{pff} = 0.38 * \sqrt{[E / F_y]}$ = **9.15**

Limiting ratio for non-compact section $\lambda_{rff} = 1.0 * \sqrt{[E / F_y]}$ = **24.08** Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio $(d - 2 * k) / t_w$ = **35.85**

Limiting ratio for compact section $\lambda_{pwf} = 3.76 * \sqrt{[E / F_y]}$ = **90.55**

Limiting ratio for non-compact section $\lambda_{rwf} = 5.70 * \sqrt{[E / F_y]}$ = **137.27** Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **39.714** kips

Web area $A_w = d * t_w$ = **6.439** in^2

Web plate buckling coefficient k_v = **5.34**

Web shear coefficient - eq G2-3 C_{v1} = **1**

Nominal shear strength – eq G6-1 $V_n = 0.6 * F_y * A_w * C_{v1}$ = **193.155** kips

Safety factor for shear Ω_v = **1.50**

Allowable shear strength $V_c = V_n / \Omega_v =$ **128.770** kips

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure in the major axis - Chapter F

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **226.388** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y * Z_x =$ **541.667** kips_ft

Nominal flexural strength $M_n = M_{nyld} =$ **541.667** kips_ft

Allowable flexural strength $M_c = M_n / \Omega_b =$ **324.351** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

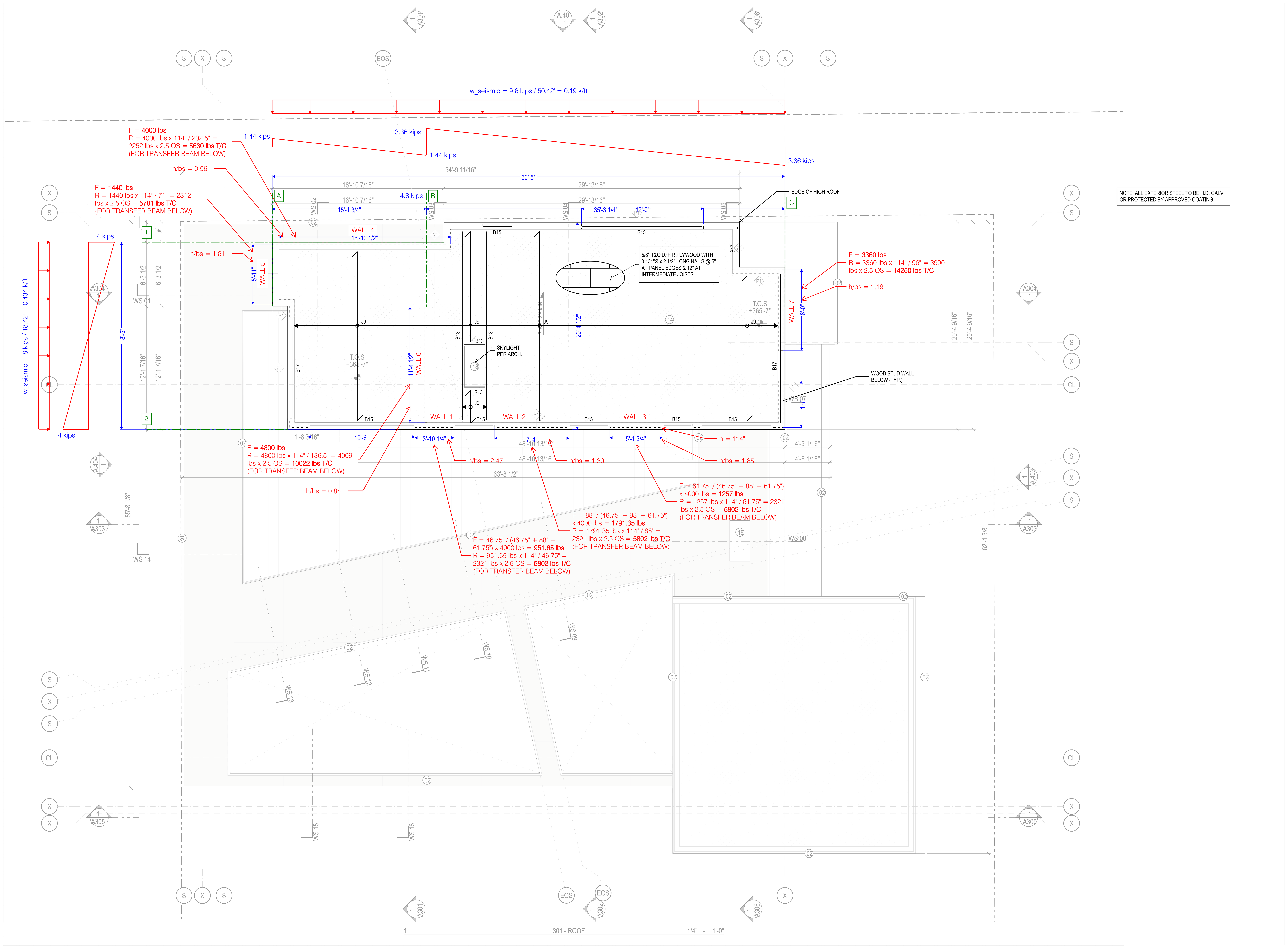
Limiting deflection $\delta_{lim} = L_{s1} / 240 =$ **0.714** in

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0.214** in

PASS - Maximum deflection does not exceed deflection limit

3 | LATERAL DESIGN

3.1 | WOOD FRAME SHEAR WALL DESIGN



NOTE: ALL EXTERIOR STEEL TO BE H.D. GALV. OR PROTECTED BY APPROVED COATING.

PROJECT ZERO & BUILDING ELEVATIONS	TRUE ELEVATION	ELEVATION TO PROJECT ZERO
PROJECT ZERO:	+000.00	+0.00
AVERAGE BUILDING ELEVATION	+000.00	-0.00
GROUND LEVEL:	+000.00	+0.00
MAXIMUM BUILDING HEIGHT:	+000.00	+000.00
MAXIMUM BUILDING HEIGHT (ROOF HEIGHT EXCEPTIONS):	+000.00	+00.00
ACTUAL BUILDING HEIGHT:	+000.00	+00.00

NOTE: ALL DIMENSION TO PROJECT ZERO, UON.

GENERAL NOTES

PERMIT DRAWINGS ARE NOT CONSTRUCTION DRAWINGS.

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True North
Project North

Revision History

Sheet Size:
24" x 36"

Drawing Title:
301 ROOF PLAN

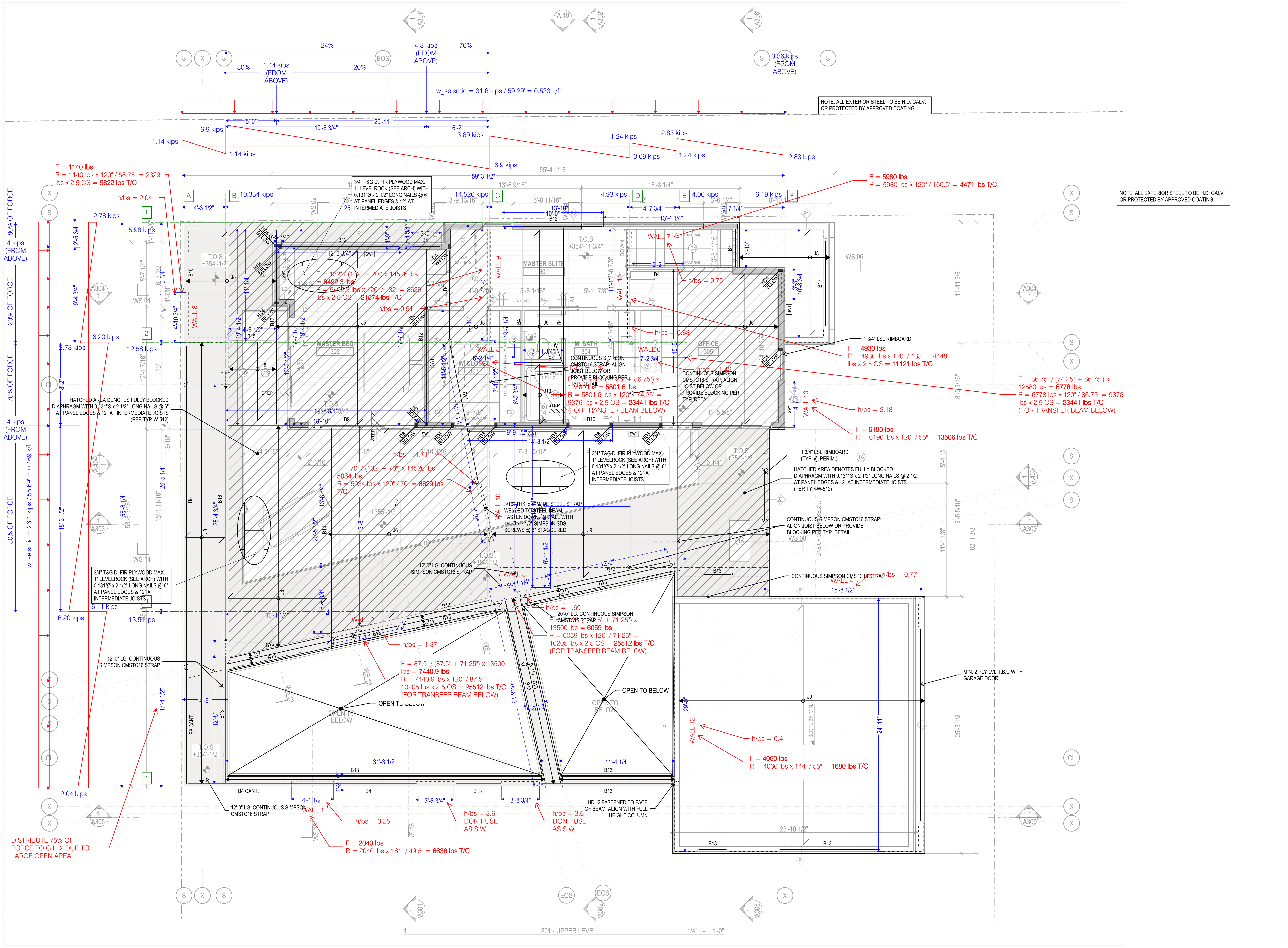
Drawing Status:
BUILDING PERMIT

Issue Date:
12/30/20

Issue:
BUILDING PERMIT

Revision:

SHEET NAME
S204



PROJECT ZERO & BUILDING ELEVATIONS	TRUE ELEVATION	ELEVATION TO PROJECT ZERO
PROJECT ZERO:	+000.00	+0.00
AVERAGE BUILDING ELEVATION	+000.00	-0.00
GROUND LEVEL:	+000.00	+0.00
MAXIMUM BUILDING HEIGHT:	+000.00	+000.00
MAXIMUM BUILDING HEIGHT (ROOF HEIGHT EXCEPTIONS):	+000.00	+00.00
ACTUAL BUILDING HEIGHT:	+000.00	+00.00

NOTE: ALL DIMENSION TO PROJECT ZERO, UON.

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True North | Project North

Revision History

Sheet Size:
24" x 36"

Drawing Title:
UPPER LEVEL PLAN

Drawing Status:
BUILDING PERMIT

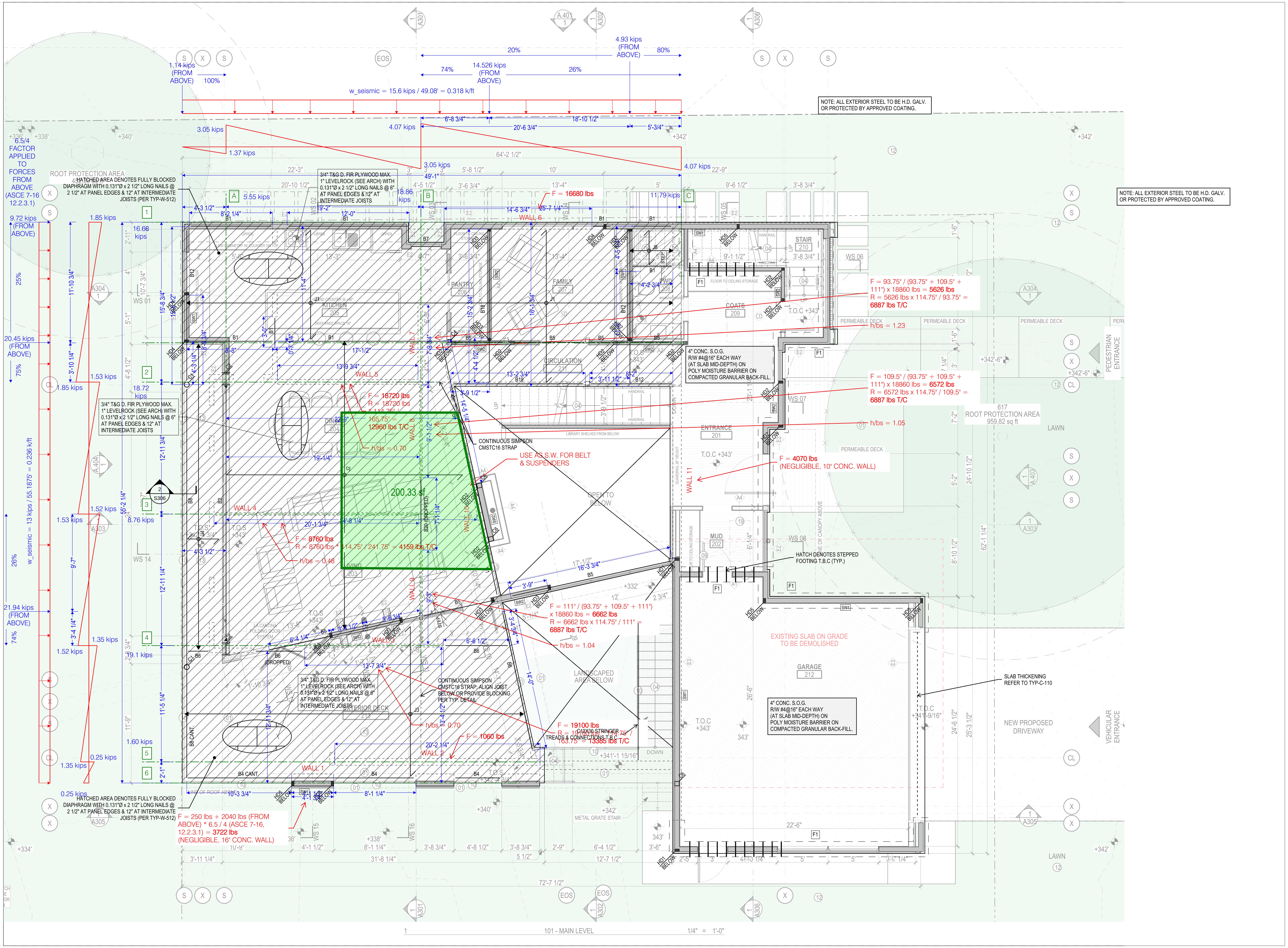
Issue Date:
12/30/20

Issue:
BUILDING PERMIT

Revision:
01

SHEET NAME

S203



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PROJECT ZERO & BUILDING ELEVATIONS	TRUE ELEVATION	ELEVATION TO PROJECT ZERO
PROJECT ZERO:	+000.00	+0.00
AVERAGE BUILDING ELEVATION	+000.00	-0.00
GROUND LEVEL:	+000.00	+0.00
MAXIMUM BUILDING HEIGHT:	+000.00	+000.00
MAXIMUM BUILDING HEIGHT (ROOF HEIGHT EXCEPTIONS):	+000.00	+00.00
ACTUAL BUILDING HEIGHT:	+000.00	+00.00

NOTE: ALL DIMENSION TO PROJECT ZERO, UON.

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True North | Project North

Revision History

Sheet Size:
24" x 36"

Drawing Title:
MAIN LEVEL PLAN

Drawing Status:
BUILDING PERMIT

Issue Date:
12/30/20

Issue:
BUILDING PERMIT

Revision:
01

SHEET NAME
S202

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

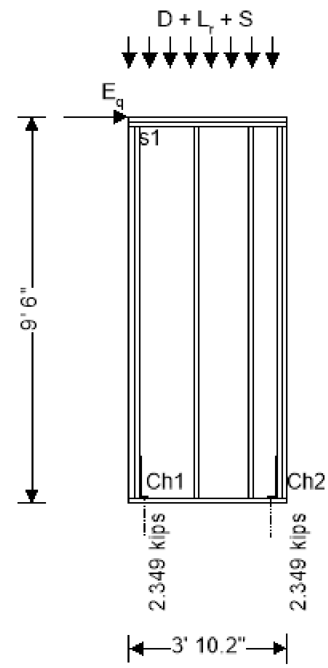
Panel details

Structural wood panel sheathing on one side

Panel height h = **9.5** ft

Panel length b = **3.85** ft

Total area of wall A = h * b = **36.575** ft^2



Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

Cross-sectional area of end posts A_e = **16.5** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **13.5** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity $G = \mathbf{0.50}$

Tension parallel to grain $F_t = \mathbf{575}$ lb/in^2

Compression parallel to grain $F_c = \mathbf{1350}$ lb/in^2

Modulus of elasticity $E = \mathbf{1600000}$ lb/in^2

Minimum modulus of elasticity $E_{min} = \mathbf{580000}$ lb/in^2

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing

Fastener type 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design $v_s = \mathbf{980}$ lb/ft

Nominal unit shear capacity for wind design $v_w = \mathbf{1370}$ lb/ft

Apparent shear wall shear stiffness $G_a = \mathbf{15}$ kips/in

Loading details

Dead load acting on top of panel $D = \mathbf{276.25}$ lb/ft

Roof live load acting on top of panel $L_r = \mathbf{369}$ lb/ft

Snow load acting on top of panel $S = \mathbf{553}$ lb/ft

Self weight of panel $S_{wt} = \mathbf{12}$ lb/ft^2

In plane seismic load acting at head of panel $E_q = \mathbf{952}$ lbs

Design spectral response accel. par., short periods $S_{DS} = \mathbf{0.944}$

From IBC 2018 cl.1605.2

Load combination no.1 $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 $0.9D + W$

Load combination no.5 $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

$K_{Ft} = \mathbf{2.70}$

Format conversion factor for compression – Table N1

$K_{Fc} = \mathbf{2.40}$

Format conversion factor for modulus of elasticity – Table N1

$K_{FE} = \mathbf{1.76}$

Resistance factor for tension – Table N2 $\phi_t = \mathbf{0.80}$

Resistance factor for compression – Table N2 $\phi_c = \mathbf{0.90}$

Resistance factor for modulus of elasticity – Table N2

$\phi_s = \mathbf{0.85}$

Time effect factor – Table N3 $\lambda = \mathbf{1.00}$

Sheathing resistance factor $\phi_D = \mathbf{0.80}$

Size factor for tension – Table 4A $C_{Ft} = \mathbf{1.30}$

Size factor for compression – Table 4A $C_{Fc} = \mathbf{1.10}$

Wet service factor for tension – Table 4A $C_{Mt} = \mathbf{1.00}$

Wet service factor for compression – Table 4A $C_{Mc} = \mathbf{1.00}$

Wet service factor for modulus of elasticity – Table 4A

	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	
	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	
	C_{tE} = **1.00**
Incising factor – cl.4.3.8	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2	C_T = **1.00**
Adjusted modulus of elasticity	$E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1665** psi
Reference compression design value	$F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi
For sawn lumber	c = **0.8**
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.45**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Shear wall length	b = **3.85** ft
Shear wall aspect ratio	h / b = **2.468**

Segmented shear wall capacity

Maximum shear force under seismic loading	$V_{s\_max} = E_q$ = **0.952** kips
Shear capacity for seismic loading	$V_s = \phi_D * v_s * b * (1.25 - 0.125 * h / b_s)$ = **2.842** kips
	V_{s_max} / V_s = **0.335**
	PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio	h / b = **2.468**
Load combination 5	
Shear force for maximum tension	$V = E_q$ = **0.952** kips
Axial force for maximum tension	P = 0 kips = **0** kips
Maximum tensile force in chord	T = V * h / (b) - P = **2.349** kips
Maximum applied tensile stress	$f_t = T / A_{en}$ = **174** lb/in^2
Design tensile stress	$F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2
	f_t / F_t' = **0.108**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Load combination 3	
Shear force for maximum compression	$V = E_q$ = **0.952** kips
Axial force for maximum compression	$P = (1.2 * (D + S_{wt} * h) + 0.2 * S_{DS} * (D + S_{wt} * h) + + 0.7 * S) * s / 2$ = **0.619** kips
Maximum compressive force in chord	C = V * h / (b) + P = **2.968** kips
Maximum applied compressive stress	$f_c = C / A_e$ = **180** lb/in^2
Design compressive stress	$F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1433** lb/in^2
	f_c / F_c' = **0.126**
	PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 — T_1 = **2.349** kips

Chord 2 — T_2 = **2.349** kips

Seismic deflection

Design shear force — $V_{\delta s} = E_q$ = **0.952** kips

Deflection limit — Δ_{s_allow} = 0.020 * h = **2.28** in

Induced unit shear — $v_{\delta s} = V_{\delta s}$ / b = **247.27** lb/ft

Anchor tension force — T_δ = max(0 kips, $v_{\delta s}$ * h) = **2.349** kips

Shear wall elastic deflection – Eqn. 4.3-1 — δ_{swse} = 2 * $v_{\delta s}$ * h^3 / (3 * E * A_e * b) + $v_{\delta s}$ * h / (G_a) + h * T_δ / (k_a * b) = **0.367** in

Deflection ampification factor — $C_{d\delta}$ = **4**

Seismic importance factor — I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 — $\delta_{sws} = C_{d\delta}$ * δ_{swse} / I_e = **1.466** in

δ_{sws} / Δ_{s_allow} = **0.643**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

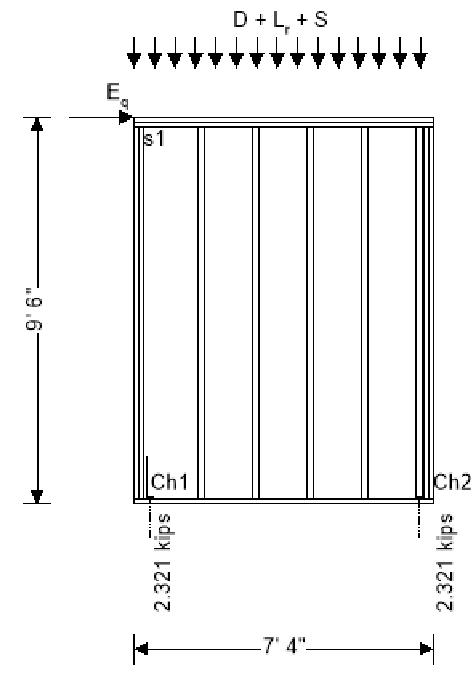
Panel details

Structural wood panel sheathing on one side

Panel height h = **9.5** ft

Panel length b = **7.333** ft

Total area of wall A = h * b = **69.666** ft^2



Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

Cross-sectional area of end posts A_e = **16.5** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **13.5** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity G = **0.50**

Tension parallel to grain F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Modulus of elasticity E = **1600000** lb/in^2

Minimum modulus of elasticity E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing

Fastener type 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **980** lb/ft

Nominal unit shear capacity for wind design v_w = **1370** lb/ft

Apparent shear wall shear stiffness G_a = **15** kips/in

Loading details

Dead load acting on top of panel D = **306** lb/ft

Roof live load acting on top of panel L_r = **408** lb/ft

Snow load acting on top of panel S = **611.25** lb/ft

Self weight of panel S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel E_q = **1792** lbs

Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 $0.9D + W$

Load combination no.5 $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2 ϕ_t = **0.80**

Resistance factor for compression – Table N2 ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3 λ = **1.00**

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.30**

Size factor for compression – Table 4A C_{Fc} = **1.10**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1665** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.45**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **7.333** ft

Shear wall aspect ratio h / b = **1.295**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **1.792** kips

Shear capacity for seismic loading $V_s = \phi_D * v_s * b$ = **5.749** kips

V_{s_max} / V_s = **0.312**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **1.295**

Load combination 5

Shear force for maximum tension $V = E_q$ = **1.792** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **2.321** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **172** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **1.792** kips

Axial force for maximum compression $P = (1.2 * (D + S_{wt} * h) + 0.2 * S_{DS} * (D + S_{wt} * h) + + 0.7 * S) * s / 2$ = **0.674** kips

Maximum compressive force in chord C = V * h / (b) + P = **2.996** kips

Maximum applied compressive stress $f_c = C / A_e$ = **182** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1433** lb/in^2

f_c / F_c' = **0.127**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 — T_1 = **2.321** kips

Chord 2 — T_2 = **2.321** kips

Seismic deflection

Design shear force — $V_{\delta s} = E_q$ = **1.792** kips

Deflection limit — Δ_{s_allow}= 0.020 * h = **2.28** in

Induced unit shear — $v_{\delta s} = V_{\delta s}$ / b = **244.36** lb/ft

Anchor tension force — T_{δ} = max(0 kips,$v_{\delta s}$ * h) = **2.321** kips

Shear wall elastic deflection – Eqn. 4.3-1 — δ_{swse} = 2 * $v_{\delta s}$ * h^3 / (3 * E * A_e * b) + $v_{\delta s}$ * h / (G_a) + h * T_{δ} / (k_a * b) = **0.264** in

Deflection ampification factor — $C_{d\delta}$ = **4**

Seismic importance factor — I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 — $\delta_{sws} = C_{d\delta}$ * δ_{swse} / I_e = **1.055** in

δ_{sws} / Δ_{s_allow} = **0.463**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

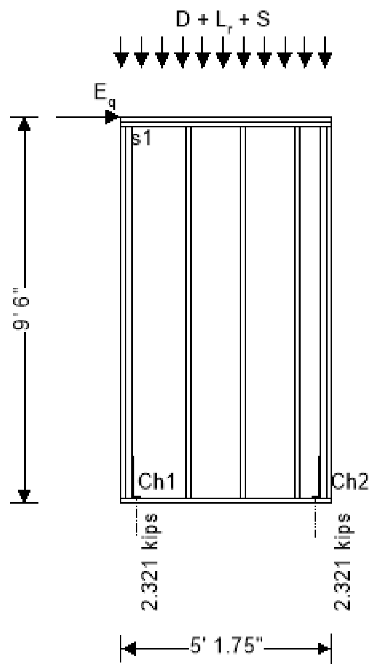
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

Panel details

Structural wood panel sheathing on one side

Panel height	h = **9.5** ft
Panel length	b = **5.146** ft
Total area of wall	A = h * b = **48.885** ft^2



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification — Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity	G = **0.50**
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Minimum modulus of elasticity	E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material	15/32" wood panel 3-ply plywood sheathing
Fastener type	8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = **980** lb/ft
Nominal unit shear capacity for wind design	v_w = **1370** lb/ft
Apparent shear wall shear stiffness	G_a = **15** kips/in

Loading details

Dead load acting on top of panel	D = **306** lb/ft
Roof live load acting on top of panel	L_r = **408** lb/ft
Snow load acting on top of panel	S = **611.25** lb/ft
Self weight of panel	S_{wt} = **12** lb/ft^2
In plane seismic load acting at head of panel	E_q = **1257** lbs
Design spectral response accel. par., short periods	S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1	1.2D + 1.6(L_r or S or R) + 0.5W
Load combination no.2	1.2D + W + 0.5L_f + 0.5(L_r or S or R)
Load combination no.3	1.2D + E + 0.5L_f + 0.7S
Load combination no.4	0.9D + W
Load combination no.5	0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2	ϕ_t = **0.80**
Resistance factor for compression – Table N2	ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3	λ = **1.00**
Sheathing resistance factor	ϕ_D = **0.80**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1665** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.45**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **5.146** ft

Shear wall aspect ratio h / b = **1.846**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **1.257** kips

Shear capacity for seismic loading $V_s = \phi_D * v_s * b$ = **4.034** kips

V_{s_max} / V_s = **0.312**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **1.846**

Load combination 5

Shear force for maximum tension $V = E_q$ = **1.257** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **2.321** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **172** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.106**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **1.257** kips

Axial force for maximum compression $P = (1.2 * (D + S_{wt} * h) + 0.2 * S_{DS} * (D + S_{wt} * h) + + 0.7 * S) * s / 2$ = **0.674** kips

Maximum compressive force in chord C = V * h / (b) + P = **2.995** kips

Maximum applied compressive stress $f_c = C / A_e$ = **181** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1433** lb/in^2

f_c / F_c' = **0.127**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **2.321** kips

Chord 2 T_2 = **2.321** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q$ = **1.257** kips

Deflection limit Δ_{s_allow}= 0.020 * h = **2.28** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b$ = **244.28** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta s} * h)$ = **2.321** kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_\delta / (k_a * b)$ = **0.31** in

Deflection ampification factor $C_{d\delta}$ = **4**

Seismic importance factor I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e$ = **1.239** in

$\delta_{sws} / \Delta_{s\_allow}$ = **0.544**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

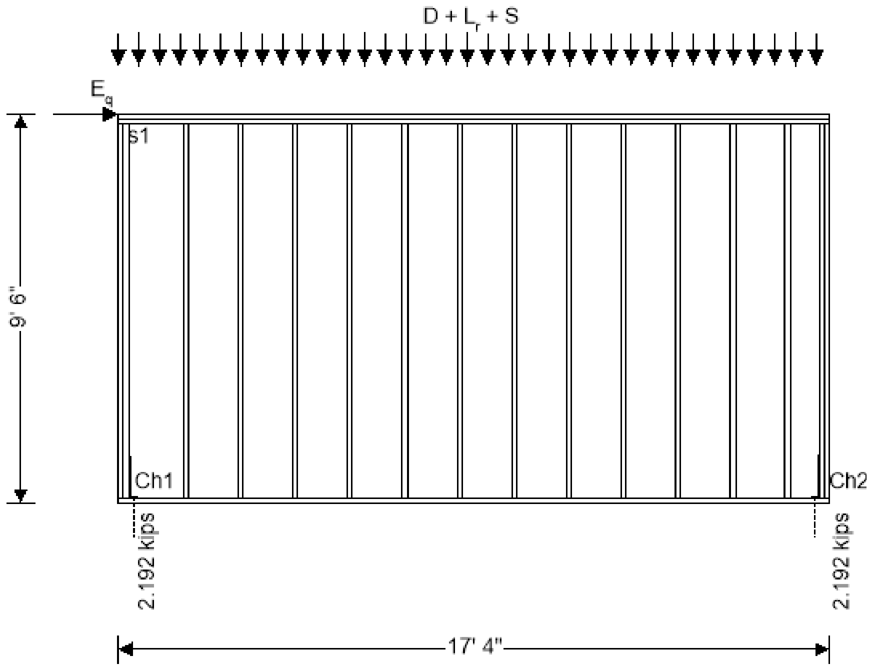
Panel details

Structural wood panel sheathing on one side

Panel height $h = \mathbf{9.5}$ ft

Panel length $b = \mathbf{17.333}$ ft

Total area of wall $A = h * b = \mathbf{164.666}$ ft^2



Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs $A_s = \mathbf{8.25}$ in^2

Stud spacing $s = \mathbf{16}$ in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

Cross-sectional area of end posts $A_e = \mathbf{16.5}$ in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts $A_{en} = \mathbf{13.5}$ in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness $k_a = \mathbf{30000}$ lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification — Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity — G = **0.50**

Tension parallel to grain — F_t = **575** lb/in^2

Compression parallel to grain — F_c = **1350** lb/in^2

Modulus of elasticity — E = **1600000** lb/in^2

Minimum modulus of elasticity — E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material — 15/32" wood panel 3-ply plywood sheathing

Fastener type — 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **980** lb/ft

Nominal unit shear capacity for wind design — v_w = **1370** lb/ft

Apparent shear wall shear stiffness — G_a = **15** kips/in

Loading details

Dead load acting on top of panel — D = **276.25** lb/ft

Roof live load acting on top of panel — L_r = **369** lb/ft

Snow load acting on top of panel — S = **553** lb/ft

Self weight of panel — S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel — E_q = **4000** lbs

Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 — $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 — $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 — $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 — $0.9D + W$

Load combination no.5 — $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2 — ϕ_t = **0.80**

Resistance factor for compression – Table N2 — ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3 — λ = **1.00**

Sheathing resistance factor — ϕ_D = **0.80**

Size factor for tension – Table 4A — C_{Ft} = **1.30**

Size factor for compression – Table 4A — C_{Fc} = **1.10**

Wet service factor for tension – Table 4A — C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1665** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.45**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **17.333** ft

Shear wall aspect ratio h / b = **0.548**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **4** kips

Shear capacity for seismic loading $V_s = \phi_D * v_s * b$ = **13.589** kips

V_{s_max} / V_s = **0.294**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **0.548**

Load combination 5

Shear force for maximum tension $V = E_q$ = **4** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **2.192** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **162** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.101**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **4** kips

Axial force for maximum compression $P = (1.2 * (D + S_{wt} * h) + 0.2 * S_{DS} * (D + S_{wt} * h) + + 0.7 * S) * s / 2$ = **0.619** kips

Maximum compressive force in chord C = V * h / (b) + P = **2.812** kips

Maximum applied compressive stress $f_c = C / A_e$ = **170** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1433** lb/in^2

f_c / F_c' = **0.119**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **2.192** kips

Chord 2 T_2 = **2.192** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q$ = **4** kips

Deflection limit $\Delta_{s\_allow} = 0.020 * h$ = **2.28** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b$ = **230.77** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta s} * h)$ = **2.192** kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_\delta / (k_a * b)$ = **0.19** in

Deflection ampification factor $C_{d\delta}$ = **4**

Seismic importance factor I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e$ = **0.759** in

$\delta_{sws} / \Delta_{s\_allow}$ = **0.333**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

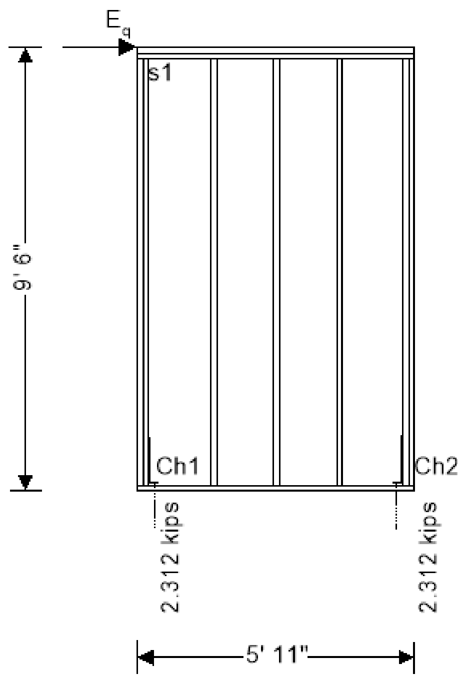
Panel details

Structural wood panel sheathing on one side

Panel height h = **9.5** ft

Panel length b = **5.917** ft

Total area of wall A = h * b = **56.209** ft^2



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification: Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity: G = **0.50**

Tension parallel to grain: F_t = **575** lb/in^2

Compression parallel to grain: F_c = **1350** lb/in^2

Modulus of elasticity: E = **1600000** lb/in^2

Minimum modulus of elasticity: E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material: 15/32" wood panel 3-ply plywood sheathing

Fastener type: 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design: v_s = **980** lb/ft

Nominal unit shear capacity for wind design: v_w = **1370** lb/ft

Apparent shear wall shear stiffness: G_a = **15** kips/in

Loading details

Self weight of panel: S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel: E_q = **1440** lbs

Design spectral response accel. par., short periods: S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1: $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2: $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3: $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4: $0.9D + W$

Load combination no.5: $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1: K_{Ft} = **2.70**

Format conversion factor for compression – Table N1: K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1: K_{FE} = **1.76**

Resistance factor for tension – Table N2: ϕ_t = **0.80**

Resistance factor for compression – Table N2: ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2: ϕ_s = **0.85**

Time effect factor – Table N3: λ = **1.00**

Sheathing resistance factor: ϕ_D = **0.80**

Size factor for tension – Table 4A: C_{Ft} = **1.30**

Size factor for compression – Table 4A: C_{Fc} = **1.10**

Wet service factor for tension – Table 4A: C_{Mt} = **1.00**

Wet service factor for compression – Table 4A: C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A: C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1665** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.45**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **5.917** ft

Shear wall aspect ratio h / b = **1.606**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **1.44** kips

Shear capacity for seismic loading $V_s = \phi_D * v_s * b$ = **4.639** kips

V_{s_max} / V_s = **0.31**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **1.606**

Load combination 5

Shear force for maximum tension $V = E_q$ = **1.44** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **2.312** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **171** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.106**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **1.44** kips

Axial force for maximum compression $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.106** kips

Maximum compressive force in chord C = V * h / (b) + P = **2.418** kips

Maximum applied compressive stress $f_c = C / A_e$ = **147** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1433** lb/in^2

f_c / F_c' = **0.102**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **2.312** kips

Chord 2 T_2 = **2.312** kips

Seismic deflection

Design shear force	$V_{\delta s} = E_q =$ **1.44** kips
Deflection limit	$\Delta_{s\_allow} = 0.020 * h =$ **2.28** in
Induced unit shear	$v_{\delta s} = V_{\delta s} / b =$ **243.38** lb/ft
Anchor tension force	$T_{\delta} = \max(0 \text{ kips}, v_{\delta s} * h) =$ **2.312** kips
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_{\delta} / (k_a * b) =$ **0.211** in
Deflection ampification factor	$C_{d\delta} =$ **4**
Seismic importance factor	$I_e =$ **1**
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e =$ **0.845** in
	$\delta_{sws} / \Delta_{s\_allow} =$ **0.371**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

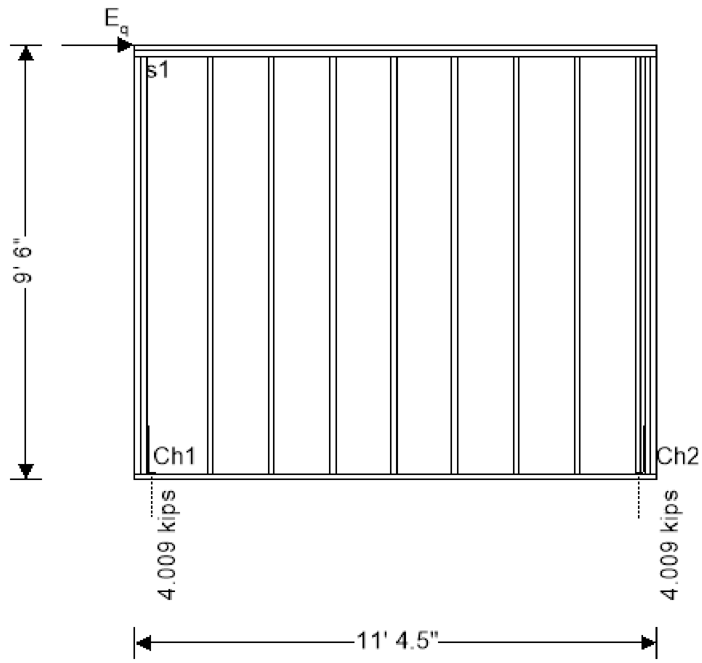
Panel details

Structural wood panel sheathing on one side

Panel height h = **9.5** ft

Panel length b = **11.375** ft

Total area of wall A = h * b = **108.063** ft^2



Panel construction

Nominal stud size 2" x 4"

Dressed stud size 1.5" x 3.5"

Cross-sectional area of studs A_s = **5.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 4"

Dressed end post size 2 x 1.5" x 3.5"

Cross-sectional area of end posts A_e = **10.5** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **7.5** in^2

Nominal collector size 2 x 2" x 4"

Dressed collector size 2 x 1.5" x 3.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity G = **0.50**

Tension parallel to grain F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Modulus of elasticity E = **1600000** lb/in^2

Minimum modulus of elasticity E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing

Fastener type 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **980** lb/ft

Nominal unit shear capacity for wind design v_w = **1370** lb/ft

Apparent shear wall shear stiffness G_a = **15** kips/in

Loading details

Self weight of panel S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel E_q = **4800** lbs

Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 1.2D + 1.6(L_r or S or R) + 0.5W

Load combination no.2 1.2D + W + 0.5L_f + 0.5(L_r or S or R)

Load combination no.3 1.2D + E + 0.5L_f + 0.7S

Load combination no.4 0.9D + W

Load combination no.5 0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2 ϕ_t = **0.80**

Resistance factor for compression – Table N2 ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3 λ = **1.00**

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.50**

Size factor for compression – Table 4A C_{Fc} = **1.15**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **674** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3353** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.19**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **11.375** ft

Shear wall aspect ratio h / b = **0.835**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **4.8** kips

Shear capacity for seismic loading $V_s = \phi_D * v_s * b$ = **8.918** kips

V_{s_max} / V_s = **0.538**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **0.835**

Load combination 5

Shear force for maximum tension $V = E_q$ = **4.8** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **4.009** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **535** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1863** lb/in^2

f_t / F_t' = **0.287**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **4.8** kips

Axial force for maximum compression $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.106** kips

Maximum compressive force in chord C = V * h / (b) + P = **4.114** kips

Maximum applied compressive stress $f_c = C / A_e$ = **392** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **644** lb/in^2

f_c / F_c' = **0.609**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **4.009** kips

Chord 2 T_2 = **4.009** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q$ = **4.8** kips

Deflection limit Δ_{s_allow}= 0.020 * h = **2.28** in

Induced unit shear $v_{\delta s}$ = $V_{\delta s}$ / b = **421.98** lb/ft

Anchor tension force T_{δ} = max(0 kips,$v_{\delta s}$ * h) = **4.009** kips

Shear wall elastic deflection – Eqn. 4.3-1 δ_{swse} = 2 * $v_{\delta s}$ * h^3 / (3 * E * A_e * b) + $v_{\delta s}$ * h / (G_a) + h * T_{δ} / (k_a * b) = **0.324** in

Deflection ampification factor $C_{d\delta}$ = **4**

Seismic importance factor I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 δ_{sws} = $C_{d\delta}$ * δ_{swse} / I_e = **1.297** in

δ_{sws} / Δ_{s_allow} = **0.569**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

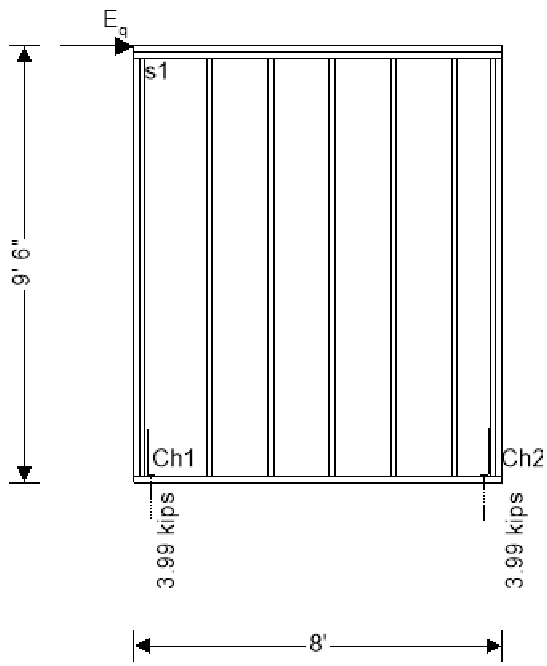
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

Panel details

Structural wood panel sheathing on one side

Panel height	h = **9.5** ft
Panel length	b = **8** ft
Total area of wall	A = h * b = **76** ft^2



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity G = **0.50**

Tension parallel to grain F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Modulus of elasticity E = **1600000** lb/in^2

Minimum modulus of elasticity E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing

Fastener type 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **980** lb/ft

Nominal unit shear capacity for wind design v_w = **1370** lb/ft

Apparent shear wall shear stiffness G_a = **15** kips/in

Loading details

Self weight of panel S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel E_q = **3360** lbs

Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 1.2D + 1.6(L_r or S or R) + 0.5W

Load combination no.2 1.2D + W + 0.5L_f + 0.5(L_r or S or R)

Load combination no.3 1.2D + E + 0.5L_f + 0.7S

Load combination no.4 0.9D + W

Load combination no.5 0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2 ϕ_t = **0.80**

Resistance factor for compression – Table N2 ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3 λ = **1.00**

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.30**

Size factor for compression – Table 4A C_{Fc} = **1.10**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1665** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.45**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **8** ft

Shear wall aspect ratio h / b = **1.188**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **3.36** kips

Shear capacity for seismic loading $V_s = \phi_D * v_s * b$ = **6.272** kips

V_{s_max} / V_s = **0.536**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **1.188**

Load combination 5

Shear force for maximum tension $V = E_q$ = **3.36** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **3.990** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **296** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.183**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **3.36** kips

Axial force for maximum compression $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.106** kips

Maximum compressive force in chord C = V * h / (b) + P = **4.096** kips

Maximum applied compressive stress $f_c = C / A_e$ = **248** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1433** lb/in^2

f_c / F_c' = **0.173**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **3.99** kips

Chord 2 T_2 = **3.99** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q$ = **3.36** kips

Deflection limit — Δ_{s_allow}= 0.020 * h = **2.28** in

Induced unit shear — $v_{\delta s}$ = $V_{\delta s}$ / b = **420** lb/ft

Anchor tension force — T_{δ} = max(0 kips,$v_{\delta s}$ * h) = **3.990** kips

Shear wall elastic deflection – Eqn. 4.3-1 — δ_{swse} = 2 * $v_{\delta s}$ * h^3 / (3 * E * A_e * b) + $v_{\delta s}$ * h / (G_a) + h * T_{δ} / (k_a * b) = **0.339** in

Deflection ampification factor — $C_{d\delta}$ = **4**

Seismic importance factor — I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 — δ_{sws} = $C_{d\delta}$ * δ_{swse} / I_e = **1.355** in

δ_{sws} / Δ_{s_allow} = **0.595**

PASS - Shear wall deflection is less than deflection limit

Tekla Tedds Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsky Residence				Job Ref. 8119	
	Section Wood Shear Wall - Supp. Upper Level Wall 1				Sheet no./rev. 1	
	Calc. by BJW	Date 2/19/2021	Chk'd by	Date	App'd by	Date

WOOD SHEAR WALL DESIGN (NDS)

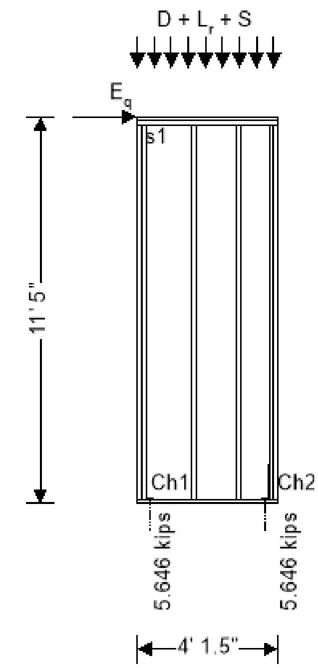
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

Panel details

Structural wood panel sheathing on one side

Panel height	h = **11.417** ft
Panel length	b = **4.125** ft
Total area of wall	A = h * b = **47.094** ft^2



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity G = **0.50**

Tension parallel to grain F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Modulus of elasticity E = **1600000** lb/in^2

Minimum modulus of elasticity E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing

Fastener type 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **980** lb/ft

Nominal unit shear capacity for wind design v_w = **1370** lb/ft

Apparent shear wall shear stiffness G_a = **15** kips/in

Loading details

Dead load acting on top of panel D = **19** lb/ft

Roof live load acting on top of panel L_r = **13** lb/ft

Snow load acting on top of panel S = **19** lb/ft

Self weight of panel S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel E_q = **2040** lbs

Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 1.2D + 1.6(L_r or S or R) + 0.5W

Load combination no.2 1.2D + W + 0.5L_f + 0.5(L_r or S or R)

Load combination no.3 1.2D + E + 0.5L_f + 0.7S

Load combination no.4 0.9D + W

Load combination no.5 0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2 ϕ_t = **0.80**

Resistance factor for compression – Table N2 ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3 λ = **1.00**

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.30**

Size factor for compression – Table 4A C_{Fc} = **1.10**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1153** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.33**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **4.125** ft

Shear wall aspect ratio h / b = **2.768**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **2.04** kips

Shear capacity for seismic loading $V_s = \phi_D * v_s * b * (1.25 - 0.125 * h / b_s)$ = **2.924** kips

V_{s_max} / V_s = **0.698**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **2.768**

Load combination 5

Shear force for maximum tension $V = E_q$ = **2.04** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **5.646** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **418** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.259**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **2.04** kips

Axial force for maximum compression $P = (1.2 * (D + S_{wt} * h) + 0.2 * S_{DS} * (D + S_{wt} * h) + + 0.7 * S) * s / 2$ = **0.153** kips

Maximum compressive force in chord C = V * h / (b) + P = **5.799** kips

Maximum applied compressive stress $f_c = C / A_e$ = **351** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1050** lb/in^2

f_c / F_c' = **0.335**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **5.646** kips

Chord 2 T_2 = **5.646** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q$ = **2.04** kips

Deflection limit Δ_{s_allow}= 0.020 * h = **2.74** in

Induced unit shear $v_{\delta s} = V_{\delta s}$ / b = **494.55** lb/ft

Anchor tension force T_δ = max(0 kips,$v_{\delta s}$ * h) = **5.646** kips

Shear wall elastic deflection – Eqn. 4.3-1 δ_{swse} = 2 * $v_{\delta s}$ * h^3 / (3 * E * A_e * b) + $v_{\delta s}$ * h / (G_a) + h * T_δ / (k_a * b) = **0.626** in

Deflection ampification factor $C_{d\delta}$ = **4**

Seismic importance factor I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta}$ * δ_{swse} / I_e = **2.503** in

δ_{sws} / Δ_{s_allow} = **0.914**

PASS - Shear wall deflection is less than deflection limit

Tekla Tedds Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsky Residence				Job Ref. 8119	
	Section Wood Shear Wall - Supp. Upper Level Wall 2				Sheet no./rev. 1	
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

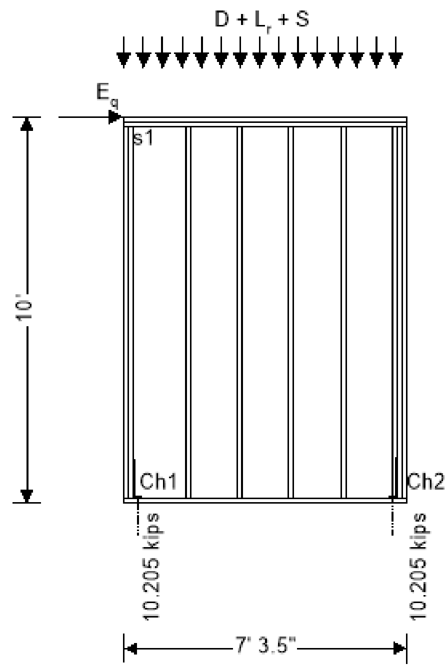
Panel details

Structural wood panel sheathing on both sides

Panel height h = **10** ft

Panel length b = **7.292** ft

Total area of wall A = h * b = **72.917** ft^2



Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

Cross-sectional area of end posts A_e = **16.5** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **13.5** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity G = **0.50**

Tension parallel to grain F_t = **575** lb/in²

Compression parallel to grain F_c = **1350** lb/in²

Modulus of elasticity E = **1600000** lb/in²

Minimum modulus of elasticity E_{min} = **580000** lb/in²

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing

Fastener type 8d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **1280** lb/ft

Nominal unit shear capacity for wind design v_w = **1790** lb/ft

Apparent shear wall shear stiffness G_a = **20** kips/in

Combined unit shear capacities

Combined nominal unit shear capacity for seismic design

$v_{sc} = 2 * v_s$ = **2560** lb/ft

Combined nominal unit shear capacity for wind design

$v_{wc} = 2 * v_w$ = **3580** lb/ft

Combined apparent shear wall shear stiffness $G_{ac} = G_{a1} + G_{a2}$ = **40** kips/in

Loading details

Dead load acting on top of panel D = **295** lb/ft

Roof live load acting on top of panel L_r = **200** lb/ft

Snow load acting on top of panel S = **295** lb/ft

Self weight of panel S_{wt} = **12** lb/ft²

In plane seismic load acting at head of panel E_q = **7441** lbs

Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 $0.9D + W$

Load combination no.5 $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2 ϕ_t = **0.80**

Resistance factor for compression – Table N2 ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3 λ = **1.00**

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.30**

Size factor for compression – Table 4A C_{Fc} = **1.10**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1502** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **7.292** ft

Shear wall aspect ratio h / b = **1.371**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **7.441** kips

Shear capacity for seismic loading $V_s = \phi_D * v_{sc} * b$ = **14.933** kips

V_{s_max} / V_s = **0.498**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **1.371**

Load combination 5

Shear force for maximum tension $V = E_q$ = **7.441** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **10.205** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **756** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.468**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **7.441** kips

Axial force for maximum compression $P = (1.2 * (D + S_{wt} * h) + 0.2 * S_{DS} * (D + S_{wt} * h) + + 0.7 * S) * s / 2$ = **0.522** kips

Maximum compressive force in chord $C = V * h / (b) + P$ = **10.727** kips

Maximum applied compressive stress $f_c = C / A_e$ = **650** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1318** lb/in^2

f_c / F_c' = **0.493**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **10.205** kips

Chord 2 T_2 = **10.205** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q$ = **7.441** kips

Deflection limit $\Delta_{s\_allow} = 0.020 * h$ = **2.4** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b$ = **1020.48** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta s} * h)$ = **10.205** kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_{ac}) + h * T_\delta / (k_a * b)$ = **0.472** in

Deflection ampification factor $C_{d\delta}$ = **4**

Seismic importance factor I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e$ = **1.89** in

$\delta_{sws} / \Delta_{s\_allow}$ = **0.787**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

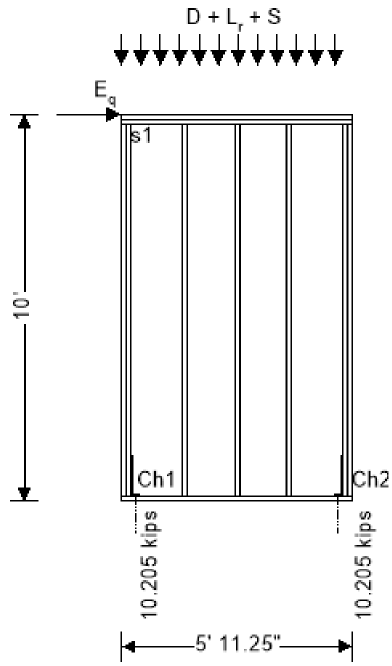
Panel details

Structural wood panel sheathing on both sides

Panel height h = **10** ft

Panel length b = **5.938** ft

Total area of wall A = h * b = **59.375** ft^2



Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

Cross-sectional area of end posts A_e = **16.5** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **13.5** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity $G = \mathbf{0.50}$

Tension parallel to grain $F_t = \mathbf{575}$ lb/in²

Compression parallel to grain $F_c = \mathbf{1350}$ lb/in²

Modulus of elasticity $E = \mathbf{1600000}$ lb/in²

Minimum modulus of elasticity $E_{min} = \mathbf{580000}$ lb/in²

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing

Fastener type 8d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design $v_s = \mathbf{1280}$ lb/ft

Nominal unit shear capacity for wind design $v_w = \mathbf{1790}$ lb/ft

Apparent shear wall shear stiffness $G_a = \mathbf{20}$ kips/in

Combined unit shear capacities

Combined nominal unit shear capacity for seismic design

$v_{sc} = 2 * v_s = \mathbf{2560}$ lb/ft

Combined nominal unit shear capacity for wind design

$v_{wc} = 2 * v_w = \mathbf{3580}$ lb/ft

Combined apparent shear wall shear stiffness $G_{ac} = G_{a1} + G_{a2} = \mathbf{40}$ kips/in

Loading details

Dead load acting on top of panel $D = \mathbf{245}$ lb/ft

Roof live load acting on top of panel $L_r = \mathbf{164}$ lb/ft

Snow load acting on top of panel $S = \mathbf{245}$ lb/ft

Self weight of panel $S_{wt} = \mathbf{12}$ lb/ft²

In plane seismic load acting at head of panel $E_q = \mathbf{6059}$ lbs

Design spectral response accel. par., short periods $S_{DS} = \mathbf{0.944}$

From IBC 2018 cl.1605.2

Load combination no.1 $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 $0.9D + W$

Load combination no.5 $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

$K_{Ft} = \mathbf{2.70}$

Format conversion factor for compression – Table N1

$K_{Fc} = \mathbf{2.40}$

Format conversion factor for modulus of elasticity – Table N1

$K_{FE} = \mathbf{1.76}$

Resistance factor for tension – Table N2 $\phi_t = \mathbf{0.80}$

Resistance factor for compression – Table N2 $\phi_c = \mathbf{0.90}$

Resistance factor for modulus of elasticity – Table N2

$\phi_s = \mathbf{0.85}$

Time effect factor – Table N3 λ = **1.00**

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.30**

Size factor for compression – Table 4A C_{Fc} = **1.10**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1502** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **5.938** ft

Shear wall aspect ratio h / b = **1.684**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **6.059** kips

Shear capacity for seismic loading $V_s = \phi_D * v_{sc} * b$ = **12.16** kips

V_{s_max} / V_s = **0.498**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **1.684**

Load combination 5

Shear force for maximum tension $V = E_q$ = **6.059** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **10.205** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **756** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.468**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **6.059** kips

Axial force for maximum compression — $P = (1.2 * (D + S_{wt} * h) + 0.2 * S_{DS} * (D + S_{wt} * h) + + 0.7 * S) * s / 2$ = **0.452** kips

Maximum compressive force in chord — $C = V * h / (b) + P$ = **10.657** kips

Maximum applied compressive stress — $f_c = C / A_e$ = **646** lb/in^2

Design compressive stress — $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1318** lb/in^2

f_c / F_c' = **0.490**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 — T_1 = **10.205** kips

Chord 2 — T_2 = **10.205** kips

Seismic deflection

Design shear force — $V_{\delta s} = E_q$ = **6.059** kips

Deflection limit — $\Delta_{s\_allow} = 0.020 * h$ = **2.4** in

Induced unit shear — $v_{\delta s} = V_{\delta s} / b$ = **1020.46** lb/ft

Anchor tension force — $T_\delta = \max(0 \text{ kips}, v_{\delta s} * h)$ = **10.205** kips

Shear wall elastic deflection – Eqn. 4.3-1 — $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_{ac}) + h * T_\delta / (k_a * b)$ = **0.522** in

Deflection ampification factor — $C_{d\delta}$ = **4**

Seismic importance factor — I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 — $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e$ = **2.088** in

$\delta_{sws} / \Delta_{s\_allow}$ = **0.87**

PASS - Shear wall deflection is less than deflection limit

Tekla Tedds Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsky Residence				Job Ref. 8119	
	Section Wood Shear Wall - Supp. Upper Level Wall 5				Sheet no./rev. 1	
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

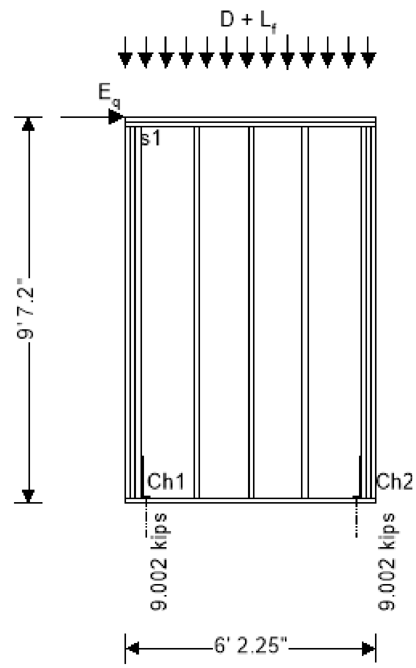
Panel details

Structural wood panel sheathing on both sides

Panel height	h = **9.6** ft

Panel length	b = **6.188** ft

Total area of wall	$A = h * b$ = **59.4** ft^2



Panel construction

Nominal stud size	2" x 4"

Dressed stud size	1.5" x 3.5"

Cross-sectional area of studs	A_s = **5.25** in^2

Stud spacing	s = **16** in

Nominal end post size	3 x 2" x 4"

Dressed end post size	3 x 1.5" x 3.5"

Cross-sectional area of end posts	A_e = **15.75** in^2

Hole diameter	Dia = **1** in

Net cross-sectional area of end posts	A_{en} = **11.25** in^2

Nominal collector size	2 x 2" x 4"

Dressed collector size	2 x 1.5" x 3.5"

Service condition	Dry

Temperature	100 degF or less

Vertical anchor stiffness	k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity G = **0.50**

Tension parallel to grain F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Modulus of elasticity E = **1600000** lb/in^2

Minimum modulus of elasticity E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing

Fastener type 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **980** lb/ft

Nominal unit shear capacity for wind design v_w = **1370** lb/ft

Apparent shear wall shear stiffness G_a = **15** kips/in

Combined unit shear capacities

Combined nominal unit shear capacity for seismic design

v_{sc} = 2 * v_s = **1960** lb/ft

Combined nominal unit shear capacity for wind design

v_{wc} = 2 * v_w = **2740** lb/ft

Combined apparent shear wall shear stiffness G_{ac} = G_{a1} + G_{a2} = **30** kips/in

Loading details

Dead load acting on top of panel D = **294** lb/ft

Floor live load acting on top of panel L_f = **392** lb/ft

Self weight of panel S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel E_q = **5802** lbs

Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 1.2D + 1.6(L_r or S or R) + 0.5W

Load combination no.2 1.2D + W + 0.5L_f + 0.5(L_r or S or R)

Load combination no.3 1.2D + E + 0.5L_f + 0.7S

Load combination no.4 0.9D + W

Load combination no.5 0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2 ϕ_t = **0.80**

Resistance factor for compression – Table N2 ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3 λ = **1.00**

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.50**

Size factor for compression – Table 4A C_{Fc} = **1.15**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **660** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3353** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.19**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **6.188** ft

Shear wall aspect ratio h / b = **1.552**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **5.802** kips

Shear capacity for seismic loading $V_s = \phi_D * v_{sc} * b$ = **9.702** kips

V_{s_max} / V_s = **0.598**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **1.552**

Load combination 5

Shear force for maximum tension $V = E_q$ = **5.802** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **9.002** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **800** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1863** lb/in^2

f_t / F_t' = **0.430**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **5.802** kips

Axial force for maximum compression $P = (1.2 * (D + S_{wt} * h) + 0.2 * S_{DS} * (D + S_{wt} * h) + 0.5 * L_f) * s / 2$ = **0.51** kips

Maximum compressive force in chord $C = V * h / (b) + P =$ **9.511** kips

Maximum applied compressive stress $f_c = C / A_e =$ **604** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P =$ **631** lb/in^2

$f_c / F_c' =$ **0.957**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 $T_1 =$ **9.002** kips

Chord 2 $T_2 =$ **9.002** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q =$ **5.802** kips

Deflection limit $\Delta_{s\_allow} = 0.020 * h =$ **2.304** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b =$ **937.7** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta s} * h) =$ **9.002** kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_{ac}) + h * T_\delta / (k_a * b) =$ **0.517** in

Deflection ampification factor $C_{d\delta} =$ **4**

Seismic importance factor $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e =$ **2.069** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.898**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

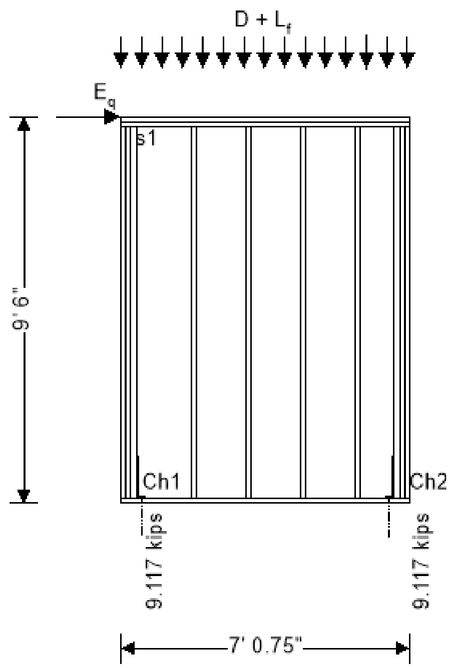
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

Panel details

Structural wood panel sheathing on both sides

Panel height	h = **9.5** ft
Panel length	b = **7.062** ft
Total area of wall	A = h * b = **67.094** ft^2



Panel construction

Nominal stud size	2" x 4"
Dressed stud size	1.5" x 3.5"
Cross-sectional area of studs	A_s = **5.25** in^2
Stud spacing	s = **16** in
Nominal end post size	3 x 2" x 4"
Dressed end post size	3 x 1.5" x 3.5"
Cross-sectional area of end posts	A_e = **15.75** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **11.25** in^2
Nominal collector size	2 x 2" x 4"
Dressed collector size	2 x 1.5" x 3.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity G = **0.50**

Tension parallel to grain F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Modulus of elasticity E = **1600000** lb/in^2

Minimum modulus of elasticity E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing

Fastener type 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **980** lb/ft

Nominal unit shear capacity for wind design v_w = **1370** lb/ft

Apparent shear wall shear stiffness G_a = **15** kips/in

Combined unit shear capacities

Combined nominal unit shear capacity for seismic design

$v_{sc} = 2 * v_s$ = **1960** lb/ft

Combined nominal unit shear capacity for wind design

$v_{wc} = 2 * v_w$ = **2740** lb/ft

Combined apparent shear wall shear stiffness $G_{ac} = G_{a1} + G_{a2}$ = **30** kips/in

Loading details

Dead load acting on top of panel D = **231.25** lb/ft

Floor live load acting on top of panel L_f = **309** lb/ft

Self weight of panel S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel E_q = **6778** lbs

Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 $0.9D + W$

Load combination no.5 $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2 ϕ_t = **0.80**

Resistance factor for compression – Table N2 ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3 λ = **1.00**

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.50**

Size factor for compression – Table 4A C_{Fc} = **1.15**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3 C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3 C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **674** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3353** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.19**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **7.062** ft

Shear wall aspect ratio h / b = **1.345**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **6.778** kips

Shear capacity for seismic loading $V_s = \phi_D * v_{sc} * b$ = **11.074** kips

V_{s_max} / V_s = **0.612**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **1.345**

Load combination 5

Shear force for maximum tension $V = E_q$ = **6.778** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **9.117** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **810** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1863** lb/in^2

f_t / F_t' = **0.435**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **6.778** kips

Axial force for maximum compression $P = (1.2 * (D + S_{wt} * h) + 0.2 * S_{DS} * (D + S_{wt} * h) + 0.5 * L_f) * s / 2$ = **0.423** kips

Maximum compressive force in chord — $C = V * h / (b) + P =$ **9.540** kips

Maximum applied compressive stress — $f_c = C / A_e =$ **606** lb/in^2

Design compressive stress — $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P =$ **644** lb/in^2

$f_c / F_c' =$ **0.941**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 — $T_1 =$ **9.117** kips

Chord 2 — $T_2 =$ **9.117** kips

Seismic deflection

Design shear force — $V_{\delta s} = E_q =$ **6.778** kips

Deflection limit — $\Delta_{s\_allow} = 0.020 * h =$ **2.28** in

Induced unit shear — $v_{\delta s} = V_{\delta s} / b =$ **959.72** lb/ft

Anchor tension force — $T_\delta = \max(0 \text{ kips}, v_{\delta s} * h) =$ **9.117** kips

Shear wall elastic deflection – Eqn. 4.3-1 — $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_{ac}) + h * T_\delta / (k_a * b) =$ **0.494** in

Deflection ampification factor — $C_{d\delta} =$ **4**

Seismic importance factor — $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 — $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e =$ **1.977** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.867**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

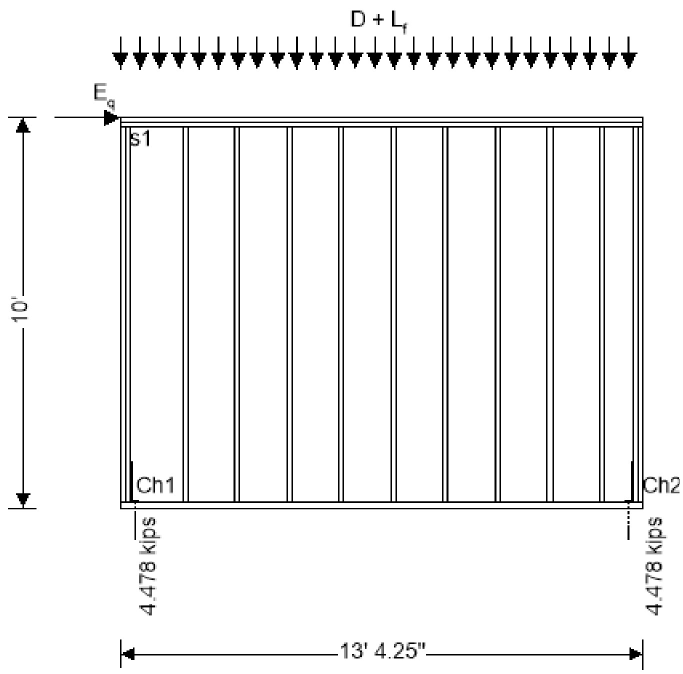
Panel details

Structural wood panel sheathing on one side

Panel height h = **10** ft

Panel length b = **13.354** ft

Total area of wall A = h * b = **133.542** ft^2



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity G = **0.50**
Tension parallel to grain F_t = **575** lb/in^2
Compression parallel to grain F_c = **1350** lb/in^2
Modulus of elasticity E = **1600000** lb/in^2
Minimum modulus of elasticity E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing
Fastener type 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **980** lb/ft
Nominal unit shear capacity for wind design v_w = **1370** lb/ft
Apparent shear wall shear stiffness G_a = **15** kips/in

Loading details

Dead load acting on top of panel D = **60** lb/ft
Floor live load acting on top of panel L_f = **80** lb/ft
Self weight of panel S_{wt} = **12** lb/ft^2
In plane seismic load acting at head of panel E_q = **5980** lbs
Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 1.2D + 1.6(L_r or S or R) + 0.5W
Load combination no.2 1.2D + W + 0.5L_f + 0.5(L_r or S or R)
Load combination no.3 1.2D + E + 0.5L_f + 0.7S
Load combination no.4 0.9D + W
Load combination no.5 0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1
K_{Ft} = **2.70**
Format conversion factor for compression – Table N1
K_{Fc} = **2.40**
Format conversion factor for modulus of elasticity – Table N1
K_{FE} = **1.76**
Resistance factor for tension – Table N2 ϕ_t = **0.80**
Resistance factor for compression – Table N2 ϕ_c = **0.90**
Resistance factor for modulus of elasticity – Table N2
ϕ_s = **0.85**
Time effect factor – Table N3 λ = **1.00**
Sheathing resistance factor ϕ_D = **0.80**
Size factor for tension – Table 4A C_{Ft} = **1.30**
Size factor for compression – Table 4A C_{Fc} = **1.10**
Wet service factor for tension – Table 4A C_{Mt} = **1.00**
Wet service factor for compression – Table 4A C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A
C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3 C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3 C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1502** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **13.354** ft

Shear wall aspect ratio h / b = **0.749**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **5.98** kips

Shear capacity for seismic loading $V_s = \phi_D * v_s * b$ = **10.47** kips

V_{s_max} / V_s = **0.571**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **0.749**

Load combination 5

Shear force for maximum tension $V = E_q$ = **5.98** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **4.478** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **332** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.205**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **5.98** kips

Axial force for maximum compression $P = (1.2 * (D + S_{wt} * h) + 0.2 * S_{DS} * (D + S_{wt} * h) + 0.5 * L_f) * s / 2$ = **0.193** kips

Maximum compressive force in chord C = V * h / (b) + P = **4.671** kips

Maximum applied compressive stress $f_c = C / A_e$ = **283** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1318** lb/in^2

f_c / F_c' = **0.215**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **4.478** kips

Chord 2 T_2 = **4.478** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q$ = **5.98** kips

Deflection limit Δ_{s_allow}= 0.020 * h = **2.4** in

Induced unit shear $v_{\delta s} = V_{\delta s}$ / b = **447.8** lb/ft

Anchor tension force T_δ = max(0 kips,$v_{\delta s}$ * h) = **4.478** kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_\delta / (k_a * b)$ = **0.351** in

Deflection ampification factor $C_{d\delta}$ = **4**

Seismic importance factor I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e$ = **1.402** in

$\delta_{sws} / \Delta_{s\_allow}$ = **0.584**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

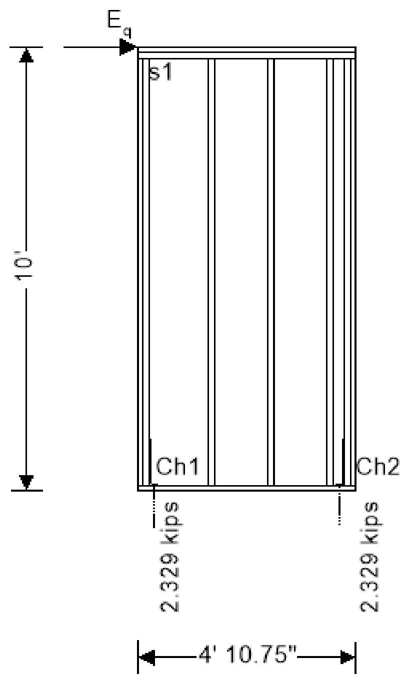
Panel details

Structural wood panel sheathing on one side

Panel height h = **10** ft

Panel length b = **4.896** ft

Total area of wall $A = h * b$ = **48.958** ft^2



Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

Cross-sectional area of end posts A_e = **16.5** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **13.5** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification — Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity — $G = \mathbf{0.50}$

Tension parallel to grain — $F_t = \mathbf{575}$ lb/in^2

Compression parallel to grain — $F_c = \mathbf{1350}$ lb/in^2

Modulus of elasticity — $E = \mathbf{1600000}$ lb/in^2

Minimum modulus of elasticity — $E_{min} = \mathbf{580000}$ lb/in^2

Sheathing details

Sheathing material — 15/32" wood panel 3-ply plywood sheathing

Fastener type — 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design — $v_s = \mathbf{980}$ lb/ft

Nominal unit shear capacity for wind design — $v_w = \mathbf{1370}$ lb/ft

Apparent shear wall shear stiffness — $G_a = \mathbf{15}$ kips/in

Loading details

Self weight of panel — $S_{wt} = \mathbf{12}$ lb/ft^2

In plane seismic load acting at head of panel — $E_q = \mathbf{1140}$ lbs

Design spectral response accel. par., short periods — $S_{DS} = \mathbf{0.944}$

From IBC 2018 cl.1605.2

Load combination no.1 — $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 — $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 — $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 — $0.9D + W$

Load combination no.5 — $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

$K_{Ft} = \mathbf{2.70}$

Format conversion factor for compression – Table N1

$K_{Fc} = \mathbf{2.40}$

Format conversion factor for modulus of elasticity – Table N1

$K_{FE} = \mathbf{1.76}$

Resistance factor for tension – Table N2 — $\phi_t = \mathbf{0.80}$

Resistance factor for compression – Table N2 — $\phi_c = \mathbf{0.90}$

Resistance factor for modulus of elasticity – Table N2

$\phi_s = \mathbf{0.85}$

Time effect factor – Table N3 — $\lambda = \mathbf{1.00}$

Sheathing resistance factor — $\phi_D = \mathbf{0.80}$

Size factor for tension – Table 4A — $C_{Ft} = \mathbf{1.30}$

Size factor for compression – Table 4A — $C_{Fc} = \mathbf{1.10}$

Wet service factor for tension – Table 4A — $C_{Mt} = \mathbf{1.00}$

Wet service factor for compression – Table 4A — $C_{Mc} = \mathbf{1.00}$

Wet service factor for modulus of elasticity – Table 4A

$C_{ME} = \mathbf{1.00}$

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3 C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3 C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1502** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **4.896** ft

Shear wall aspect ratio h / b = **2.043**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **1.14** kips

Shear capacity for seismic loading $V_s = \phi_D * v_s * b * (1.25 - 0.125 * h / b_s)$ = **3.818** kips

V_{s_max} / V_s = **0.299**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **2.043**

Load combination 5

Shear force for maximum tension $V = E_q$ = **1.14** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **2.329** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **172** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **1.14** kips

Axial force for maximum compression $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.111** kips

Maximum compressive force in chord C = V * h / (b) + P = **2.440** kips

Maximum applied compressive stress $f_c = C / A_e$ = **148** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1318** lb/in^2

f_c / F_c' = **0.112**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **2.329** kips

Chord 2 T_2 = **2.329** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q =$ **1.14** kips

Deflection limit $\Delta_{s\_allow} = 0.020 * h =$ **2.4** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b =$ **232.85** lb/ft

Anchor tension force $T_{\delta} = \max(0 \text{ kips}, v_{\delta s} * h) =$ **2.329** kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_{\delta} / (k_a * b) =$ **0.229** in

Deflection ampification factor $C_{d\delta} =$ **4**

Seismic importance factor $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e =$ **0.916** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.382**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

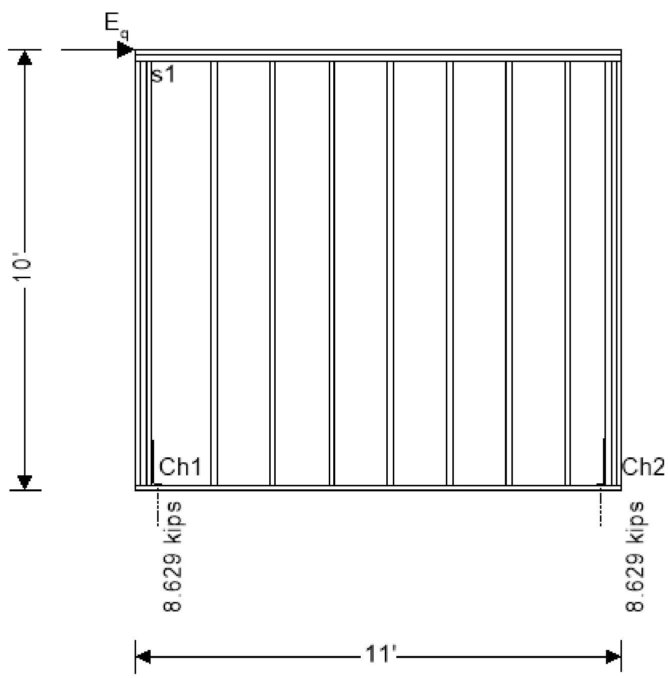
Panel details

Structural wood panel sheathing on both sides

Panel height h = **10** ft

Panel length b = **11** ft

Total area of wall A = h * b = **110** ft^2



Panel construction

Nominal stud size 2" x 4"

Dressed stud size 1.5" x 3.5"

Cross-sectional area of studs A_s = **5.25** in^2

Stud spacing s = **16** in

Nominal end post size 3 x 2" x 4"

Dressed end post size 3 x 1.5" x 3.5"

Cross-sectional area of end posts A_e = **15.75** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **11.25** in^2

Nominal collector size 2 x 2" x 4"

Dressed collector size 2 x 1.5" x 3.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification — Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity — $G = \mathbf{0.50}$

Tension parallel to grain — $F_t = \mathbf{575}$ lb/in^2

Compression parallel to grain — $F_c = \mathbf{1350}$ lb/in^2

Modulus of elasticity — $E = \mathbf{1600000}$ lb/in^2

Minimum modulus of elasticity — $E_{min} = \mathbf{580000}$ lb/in^2

Sheathing details

Sheathing material — 15/32" wood panel 3-ply plywood sheathing

Fastener type — 8d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design — $v_s = \mathbf{1280}$ lb/ft

Nominal unit shear capacity for wind design — $v_w = \mathbf{1790}$ lb/ft

Apparent shear wall shear stiffness — $G_a = \mathbf{20}$ kips/in

Combined unit shear capacities

Combined nominal unit shear capacity for seismic design

$v_{sc} = 2 * v_s = \mathbf{2560}$ lb/ft

Combined nominal unit shear capacity for wind design

$v_{wc} = 2 * v_w = \mathbf{3580}$ lb/ft

Combined apparent shear wall shear stiffness — $G_{ac} = G_{a1} + G_{a2} = \mathbf{40}$ kips/in

Loading details

Self weight of panel — $S_{wt} = \mathbf{12}$ lb/ft^2

In plane seismic load acting at head of panel — $E_q = \mathbf{9492.3}$ lbs

Design spectral response accel. par., short periods — $S_{DS} = \mathbf{0.944}$

From IBC 2018 cl.1605.2

Load combination no.1 — $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 — $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 — $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 — $0.9D + W$

Load combination no.5 — $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

$K_{Ft} = \mathbf{2.70}$

Format conversion factor for compression – Table N1

$K_{Fc} = \mathbf{2.40}$

Format conversion factor for modulus of elasticity – Table N1

$K_{FE} = \mathbf{1.76}$

Resistance factor for tension – Table N2 — $\phi_t = \mathbf{0.80}$

Resistance factor for compression – Table N2 — $\phi_c = \mathbf{0.90}$

Resistance factor for modulus of elasticity – Table N2

$\phi_s = \mathbf{0.85}$

Time effect factor – Table N3 — $\lambda = \mathbf{1.00}$

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.50**

Size factor for compression – Table 4A C_{Fc} = **1.15**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **608** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3353** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **11** ft

Shear wall aspect ratio h / b = **0.909**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **9.492** kips

Shear capacity for seismic loading $V_s = \phi_D * v_{sc} * b$ = **22.528** kips

V_{s_max} / V_s = **0.421**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **0.909**

Load combination 5

Shear force for maximum tension $V = E_q$ = **9.492** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **8.629** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **767** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1863** lb/in^2

f_t / F_t' = **0.412**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **9.492** kips

Axial force for maximum compression $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.111** kips

Maximum compressive force in chord C = V * h / (b) + P = **8.740** kips

Maximum applied compressive stress $f_c = C / A_e =$ **555** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P =$ **584** lb/in^2

$f_c / F_c' =$ **0.951**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 $T_1 =$ **8.629** kips

Chord 2 $T_2 =$ **8.629** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q =$ **9.492** kips

Deflection limit $\Delta_{s\_allow} = 0.020 * h =$ **2.4** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b =$ **862.94** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta s} * h) =$ **8.629** kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_{ac}) + h * T_\delta / (k_a * b) =$ **0.339** in

Deflection ampification factor $C_{d\delta} =$ **4**

Seismic importance factor $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e =$ **1.355** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.564**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

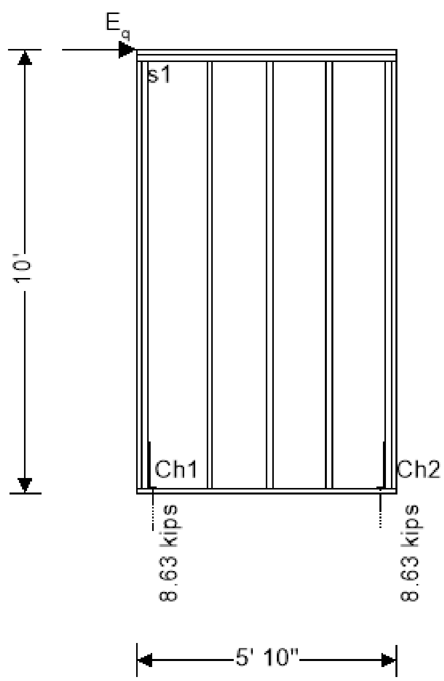
Panel details

Structural wood panel sheathing on both sides

Panel height h = **10** ft

Panel length b = **5.833** ft

Total area of wall A = h * b = **58.333** ft^2



Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

Cross-sectional area of end posts A_e = **16.5** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **13.5** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification — Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity — G = **0.50**

Tension parallel to grain — F_t = **575** lb/in^2

Compression parallel to grain — F_c = **1350** lb/in^2

Modulus of elasticity — E = **1600000** lb/in^2

Minimum modulus of elasticity — E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material — 15/32" wood panel 3-ply plywood sheathing

Fastener type — 8d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design — v_s = **1280** lb/ft

Nominal unit shear capacity for wind design — v_w = **1790** lb/ft

Apparent shear wall shear stiffness — G_a = **20** kips/in

Combined unit shear capacities

Combined nominal unit shear capacity for seismic design

$v_{sc} = 2 * v_s$ = **2560** lb/ft

Combined nominal unit shear capacity for wind design

$v_{wc} = 2 * v_w$ = **3580** lb/ft

Combined apparent shear wall shear stiffness — $G_{ac} = G_{a1} + G_{a2}$ = **40** kips/in

Loading details

Self weight of panel — S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel — E_q = **5034** lbs

Design spectral response accel. par., short periods — S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 — $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 — $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 — $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 — $0.9D + W$

Load combination no.5 — $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2 — ϕ_t = **0.80**

Resistance factor for compression – Table N2 — ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3 — λ = **1.00**

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.30**

Size factor for compression – Table 4A C_{Fc} = **1.10**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3 C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3 C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1502** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **5.833** ft

Shear wall aspect ratio h / b = **1.714**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **5.034** kips

Shear capacity for seismic loading $V_s = \phi_D * v_{sc} * b$ = **11.947** kips

V_{s_max} / V_s = **0.421**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **1.714**

Load combination 5

Shear force for maximum tension $V = E_q$ = **5.034** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **8.630** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **639** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.396**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **5.034** kips

Axial force for maximum compression $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.111** kips

Maximum compressive force in chord C = V * h / (b) + P = **8.741** kips

Maximum applied compressive stress $f_c = C / A_e =$ **530** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P =$ **1318** lb/in^2

$f_c / F_c' =$ **0.402**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 $T_1 =$ **8.63** kips

Chord 2 $T_2 =$ **8.63** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q =$ **5.034** kips

Deflection limit $\Delta_{s\_allow} = 0.020 * h =$ **2.4** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b =$ **862.98** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta s} * h) =$ **8.630** kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_{ac}) + h * T_\delta / (k_a * b) =$ **0.445** in

Deflection ampification factor $C_{d\delta} =$ **4**

Seismic importance factor $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e =$ **1.782** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.742**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

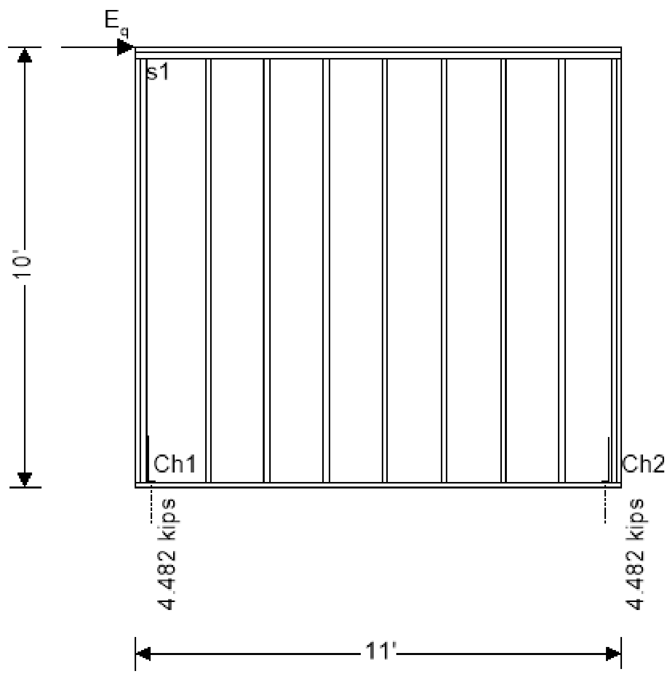
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

Panel details

Structural wood panel sheathing on one side

Panel height	h = **10** ft
Panel length	b = **11** ft
Total area of wall	A = h * b = **110** ft^2



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification — Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity — G = **0.50**

Tension parallel to grain — F_t = **575** lb/in^2

Compression parallel to grain — F_c = **1350** lb/in^2

Modulus of elasticity — E = **1600000** lb/in^2

Minimum modulus of elasticity — E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material — 15/32" wood panel 3-ply plywood sheathing

Fastener type — 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design — v_s = **980** lb/ft

Nominal unit shear capacity for wind design — v_w = **1370** lb/ft

Apparent shear wall shear stiffness — G_a = **15** kips/in

Loading details

Self weight of panel — S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel — E_q = **4930** lbs

Design spectral response accel. par., short periods — S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 — 1.2D + 1.6(L_r or S or R) + 0.5W

Load combination no.2 — 1.2D + W + 0.5L_f + 0.5(L_r or S or R)

Load combination no.3 — 1.2D + E + 0.5L_f + 0.7S

Load combination no.4 — 0.9D + W

Load combination no.5 — 0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2 — ϕ_t = **0.80**

Resistance factor for compression – Table N2 — ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3 — λ = **1.00**

Sheathing resistance factor — ϕ_D = **0.80**

Size factor for tension – Table 4A — C_{Ft} = **1.30**

Size factor for compression – Table 4A — C_{Fc} = **1.10**

Wet service factor for tension – Table 4A — C_{Mt} = **1.00**

Wet service factor for compression – Table 4A — C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1502** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **11** ft

Shear wall aspect ratio h / b = **0.909**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **4.93** kips

Shear capacity for seismic loading $V_s = \phi_D * v_s * b$ = **8.624** kips

V_{s_max} / V_s = **0.572**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **0.909**

Load combination 5

Shear force for maximum tension $V = E_q$ = **4.93** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **4.482** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **332** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.206**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **4.93** kips

Axial force for maximum compression $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.111** kips

Maximum compressive force in chord C = V * h / (b) + P = **4.593** kips

Maximum applied compressive stress $f_c = C / A_e$ = **278** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1318** lb/in^2

f_c / F_c' = **0.211**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **4.482** kips

Chord 2 T_2 = **4.482** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q =$ **4.93** kips

Deflection limit $\Delta_{s\_allow} = 0.020 * h =$ **2.4** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b =$ **448.18** lb/ft

Anchor tension force $T_{\delta} = \max(0 \text{ kips}, v_{\delta s} * h) =$ **4.482** kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_{\delta} / (k_a * b) =$ **0.362** in

Deflection ampification factor $C_{d\delta} =$ **4**

Seismic importance factor $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e =$ **1.448** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.603**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

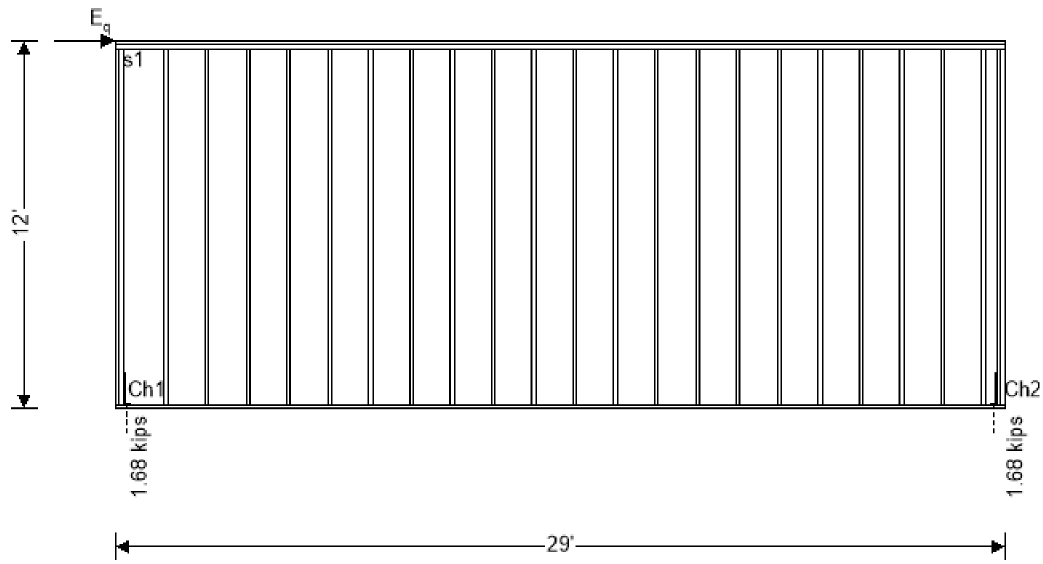
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

Panel details

Structural wood panel sheathing on one side

Panel height	h = **12** ft
Panel length	b = **29** ft
Total area of wall	A = h * b = **348** ft^2



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity	G = **0.50**
Tension parallel to grain	F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2
Modulus of elasticity E = **1600000** lb/in^2
Minimum modulus of elasticity E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing
Fastener type 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **980** lb/ft
Nominal unit shear capacity for wind design v_w = **1370** lb/ft
Apparent shear wall shear stiffness G_a = **15** kips/in

Loading details

Self weight of panel S_{wt} = **12** lb/ft^2
In plane seismic load acting at head of panel E_q = **4060** lbs
Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 1.2D + 1.6(L_r or S or R) + 0.5W
Load combination no.2 1.2D + W + 0.5L_f + 0.5(L_r or S or R)
Load combination no.3 1.2D + E + 0.5L_f + 0.7S
Load combination no.4 0.9D + W
Load combination no.5 0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1
K_{Ft} = **2.70**
Format conversion factor for compression – Table N1
K_{Fc} = **2.40**
Format conversion factor for modulus of elasticity – Table N1
K_{FE} = **1.76**
Resistance factor for tension – Table N2 ϕ_t = **0.80**
Resistance factor for compression – Table N2 ϕ_c = **0.90**
Resistance factor for modulus of elasticity – Table N2
ϕ_s = **0.85**
Time effect factor – Table N3 λ = **1.00**
Sheathing resistance factor ϕ_D = **0.80**
Size factor for tension – Table 4A C_{Ft} = **1.30**
Size factor for compression – Table 4A C_{Fc} = **1.10**
Wet service factor for tension – Table 4A C_{Mt} = **1.00**
Wet service factor for compression – Table 4A C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A
C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3
C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3

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	C_{tE} = **1.00**
Incising factor – cl.4.3.8	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2	C_T = **1.00**
Adjusted modulus of elasticity	$E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1043** psi
Reference compression design value	$F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi
For sawn lumber	c = **0.8**
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.30**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Shear wall length	b = **29** ft
Shear wall aspect ratio	h / b = **0.414**

Segmented shear wall capacity

Maximum shear force under seismic loading	$V_{s\_max} = E_q$ = **4.06** kips
Shear capacity for seismic loading	$V_s = \phi_D * v_s * b$ = **22.736** kips
	V_{s_max} / V_s = **0.179**
	PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio	h / b = **0.414**
Load combination 5	
Shear force for maximum tension	$V = E_q$ = **4.06** kips
Axial force for maximum tension	P = 0 kips = **0** kips
Maximum tensile force in chord	T = V * h / (b) - P = **1.680** kips
Maximum applied tensile stress	$f_t = T / A_{en}$ = **124** lb/in^2
Design tensile stress	$F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2
	f_t / F_t' = **0.077**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Load combination 3	
Shear force for maximum compression	$V = E_q$ = **4.06** kips
Axial force for maximum compression	$P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.133** kips
Maximum compressive force in chord	C = V * h / (b) + P = **1.813** kips
Maximum applied compressive stress	$f_c = C / A_e$ = **110** lb/in^2
Design compressive stress	$F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **961** lb/in^2
	f_c / F_c' = **0.114**
	PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1	T_1 = **1.68** kips
Chord 2	T_2 = **1.68** kips

Seismic deflection

Design shear force	$V_{\delta s} = E_q$ = **4.06** kips
Deflection limit	Δ_{s_allow}= 0.020 * h = **2.88** in

Induced unit shear	$v_{\delta s} = V_{\delta s} / b =$ **140** lb/ft
Anchor tension force	$T_{\delta} = \max(0 \text{ kips}, v_{\delta s} * h) =$ **1.680** kips
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_{\delta} / (k_a * b) =$ **0.123** in
Deflection ampification factor	$C_{d\delta} =$ **4**
Seismic importance factor	$I_e =$ **1**
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e =$ **0.493** in
	$\delta_{sws} / \Delta_{s\_allow} =$ **0.171**

PASS - Shear wall deflection is less than deflection limit

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WOOD SHEAR WALL DESIGN (NDS)

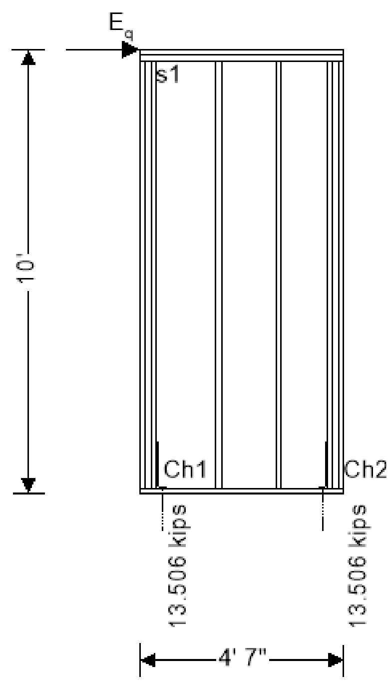
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

Panel details

Structural wood panel sheathing on both sides

Panel height	h = **10** ft
Panel length	b = **4.583** ft
Total area of wall	A = h * b = **45.833** ft^2



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	3 x 2" x 6"
Dressed end post size	3 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **24.75** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **20.25** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **84000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification — Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity — $G = \mathbf{0.50}$

Tension parallel to grain — $F_t = \mathbf{575}$ lb/in^2

Compression parallel to grain — $F_c = \mathbf{1350}$ lb/in^2

Modulus of elasticity — $E = \mathbf{1600000}$ lb/in^2

Minimum modulus of elasticity — $E_{min} = \mathbf{580000}$ lb/in^2

Sheathing details

Sheathing material — 15/32" wood panel 3-ply plywood sheathing

Fastener type — 10d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design — $v_s = \mathbf{1540}$ lb/ft

Nominal unit shear capacity for wind design — $v_w = \mathbf{2155}$ lb/ft

Apparent shear wall shear stiffness — $G_a = \mathbf{23}$ kips/in

Combined unit shear capacities

Combined nominal unit shear capacity for seismic design

$v_{sc} = 2 * v_s = \mathbf{3080}$ lb/ft

Combined nominal unit shear capacity for wind design

$v_{wc} = 2 * v_w = \mathbf{4310}$ lb/ft

Combined apparent shear wall shear stiffness — $G_{ac} = G_{a1} + G_{a2} = \mathbf{46}$ kips/in

Loading details

Self weight of panel — $S_{wt} = \mathbf{12}$ lb/ft^2

In plane seismic load acting at head of panel — $E_q = \mathbf{6190}$ lbs

Design spectral response accel. par., short periods — $S_{DS} = \mathbf{0.944}$

From IBC 2018 cl.1605.2

Load combination no.1 — $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 — $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 — $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 — $0.9D + W$

Load combination no.5 — $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

$K_{Ft} = \mathbf{2.70}$

Format conversion factor for compression – Table N1

$K_{Fc} = \mathbf{2.40}$

Format conversion factor for modulus of elasticity – Table N1

$K_{FE} = \mathbf{1.76}$

Resistance factor for tension – Table N2 — $\phi_t = \mathbf{0.80}$

Resistance factor for compression – Table N2 — $\phi_c = \mathbf{0.90}$

Resistance factor for modulus of elasticity – Table N2

$\phi_s = \mathbf{0.85}$

Time effect factor – Table N3 — $\lambda = \mathbf{1.00}$

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.30**

Size factor for compression – Table 4A C_{Fc} = **1.10**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1502** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **4.583** ft

Shear wall aspect ratio h / b = **2.182**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **6.19** kips

Shear capacity for seismic loading $V_s = \phi_D * v_{sc} * b * (1.25 - 0.125 * h / b_s)$ = **11.037** kips

V_{s_max} / V_s = **0.561**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **2.182**

Load combination 5

Shear force for maximum tension $V = E_q$ = **6.19** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **13.506** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **667** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.413**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **6.19** kips

Axial force for maximum compression $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.111** kips

Maximum compressive force in chord C = V * h / (b) + P = **13.617** kips

Maximum applied compressive stress $f_c = C / A_e =$ **550** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P =$ **1318** lb/in^2

$f_c / F_c' =$ **0.417**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 $T_1 =$ **13.506** kips

Chord 2 $T_2 =$ **13.506** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q =$ **6.19** kips

Deflection limit $\Delta_{s\_allow} = 0.020 * h =$ **2.4** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b =$ **1350.56** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta s} * h) =$ **13.506** kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_{ac}) + h * T_\delta / (k_a * b) =$ **0.704** in

Deflection ampification factor $C_{d\delta} =$ **4**

Seismic importance factor $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e =$ **2.816** in

$\delta_{sws} / \Delta_{s\_allow} =$ **1.173**

FAIL - Shear wall deflection exceeds deflection limit

Tekla Tedds Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsky Residence			Job Ref. 8119	
	Section Wood Shear Wall - Supp. Main Level Wall 3			Sheet no./rev. 1	
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

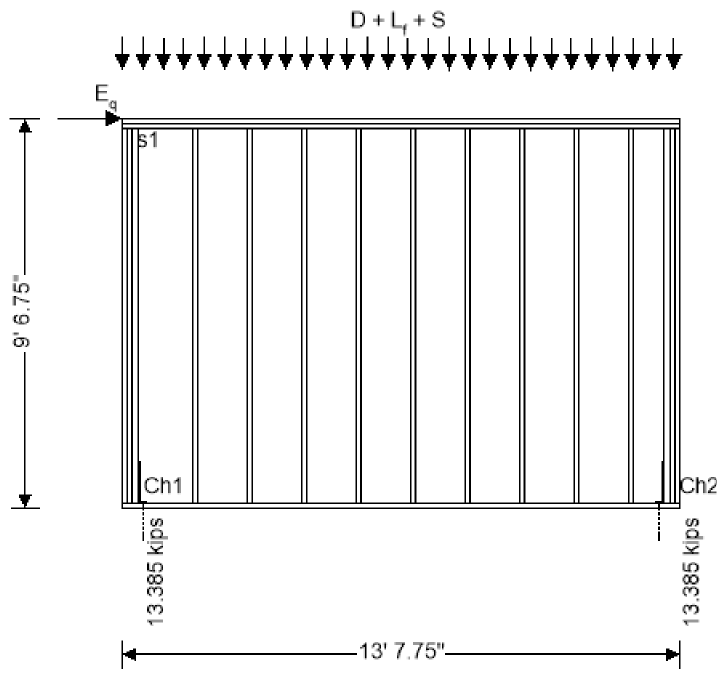
Panel details

Structural wood panel sheathing on both sides

Panel height h = **9.563** ft

Panel length b = **13.646** ft

Total area of wall A = h * b = **130.488** ft^2



Panel construction

Nominal stud size 2" x 4"

Dressed stud size 1.5" x 3.5"

Cross-sectional area of studs A_s = **5.25** in^2

Stud spacing s = **16** in

Nominal end post size 3 x 2" x 4"

Dressed end post size 3 x 1.5" x 3.5"

Cross-sectional area of end posts A_e = **15.75** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **11.25** in^2

Nominal collector size 2 x 2" x 4"

Dressed collector size 2 x 1.5" x 3.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity $G = \mathbf{0.50}$

Tension parallel to grain $F_t = \mathbf{575}$ lb/in²

Compression parallel to grain $F_c = \mathbf{1350}$ lb/in²

Modulus of elasticity $E = \mathbf{1600000}$ lb/in²

Minimum modulus of elasticity $E_{min} = \mathbf{580000}$ lb/in²

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing

Fastener type 8d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design $v_s = \mathbf{1280}$ lb/ft

Nominal unit shear capacity for wind design $v_w = \mathbf{1790}$ lb/ft

Apparent shear wall shear stiffness $G_a = \mathbf{20}$ kips/in

Combined unit shear capacities

Combined nominal unit shear capacity for seismic design

$v_{sc} = 2 * v_s = \mathbf{2560}$ lb/ft

Combined nominal unit shear capacity for wind design

$v_{wc} = 2 * v_w = \mathbf{3580}$ lb/ft

Combined apparent shear wall shear stiffness $G_{ac} = G_{a1} + G_{a2} = \mathbf{40}$ kips/in

Loading details

Dead load acting on top of panel $D = \mathbf{200}$ lb/ft

Floor live load acting on top of panel $L_f = \mathbf{400}$ lb/ft

Snow load acting on top of panel $S = \mathbf{200}$ lb/ft

Self weight of panel $S_{wt} = \mathbf{12}$ lb/ft²

In plane seismic load acting at head of panel $E_q = \mathbf{19100}$ lbs

Design spectral response accel. par., short periods $S_{DS} = \mathbf{0.944}$

From IBC 2018 cl.1605.2

Load combination no.1 $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 $0.9D + W$

Load combination no.5 $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

$K_{Ft} = \mathbf{2.70}$

Format conversion factor for compression – Table N1

$K_{Fc} = \mathbf{2.40}$

Format conversion factor for modulus of elasticity – Table N1

$K_{FE} = \mathbf{1.76}$

Resistance factor for tension – Table N2 $\phi_t = \mathbf{0.80}$

Resistance factor for compression – Table N2 $\phi_c = \mathbf{0.90}$

Resistance factor for modulus of elasticity – Table N2

$\phi_s = \mathbf{0.85}$

Time effect factor – Table N3 λ = **1.00**

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.50**

Size factor for compression – Table 4A C_{Fc} = **1.15**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3 C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3 C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **665** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3353** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.19**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **13.646** ft

Shear wall aspect ratio h / b = **0.701**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **19.1** kips

Shear capacity for seismic loading $V_s = \phi_D * v_{sc} * b$ = **27.947** kips

V_{s_max} / V_s = **0.683**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **0.701**

Load combination 5

Shear force for maximum tension $V = E_q$ = **19.1** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **13.385** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **1190** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1863** lb/in^2

f_t / F_t' = **0.639**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **19.1** kips

Axial force for maximum compression — $P = (1.2 * (D + S_{wt} * h) + 0.2 * S_{DS} * (D + S_{wt} * h) + 0.5 * L_f + 0.7 * S) * s / 2 =$ **0.518** kips

Maximum compressive force in chord — $C = V * h / (b) + P =$ **13.903** kips

Maximum applied compressive stress — $f_c = C / A_e =$ **883** lb/in²

Design compressive stress — $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P =$ **636** lb/in²

$f_c / F_c' =$ **1.389**

FAIL - Design compressive stress is less than maximum applied compressive stress

Hold down force

Chord 1 — $T_1 =$ **13.385** kips

Chord 2 — $T_2 =$ **13.385** kips

Seismic deflection

Design shear force — $V_{\delta s} = E_q =$ **19.1** kips

Deflection limit — $\Delta_{s\_allow} = 0.020 * h =$ **2.295** in

Induced unit shear — $v_{\delta s} = V_{\delta s} / b =$ **1399.7** lb/ft

Anchor tension force — $T_\delta = \max(0 \text{ kips}, v_{\delta s} * h) =$ **13.385** kips

Shear wall elastic deflection – Eqn. 4.3-1 — $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_{ac}) + h * T_\delta / (k_a * b) =$ **0.48** in

Deflection ampification factor — $C_{d\delta} =$ **4**

Seismic importance factor — $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 — $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e =$ **1.921** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.837**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

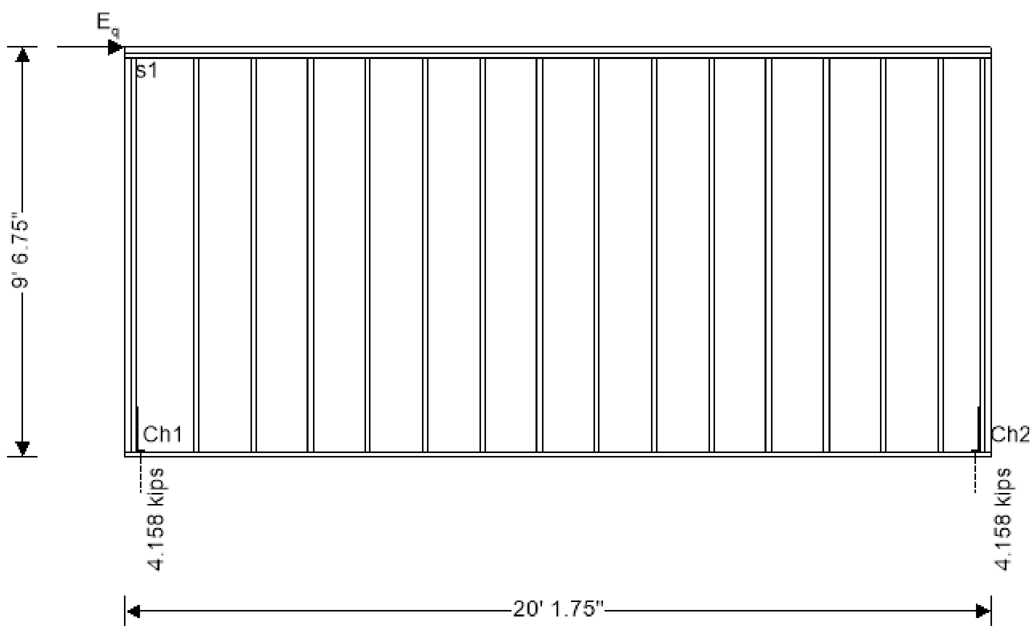
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

Panel details

Structural wood panel sheathing on one side

Panel height	h = **9.563** ft
Panel length	b = **20.146** ft
Total area of wall	A = h * b = **192.644** ft^2



Panel construction

Nominal stud size	2" x 4"
Dressed stud size	1.5" x 3.5"
Cross-sectional area of studs	A_s = **5.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 4"
Dressed end post size	2 x 1.5" x 3.5"
Cross-sectional area of end posts	A_e = **10.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **7.5** in^2
Nominal collector size	2 x 2" x 4"
Dressed collector size	2 x 1.5" x 3.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **35000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity	G = **0.50**

Tension parallel to grain F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Modulus of elasticity E = **1600000** lb/in^2

Minimum modulus of elasticity E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing

Fastener type 8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **980** lb/ft

Nominal unit shear capacity for wind design v_w = **1370** lb/ft

Apparent shear wall shear stiffness G_a = **15** kips/in

Loading details

Self weight of panel S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel E_q = **8760** lbs

Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 1.2D + 1.6(L_r or S or R) + 0.5W

Load combination no.2 1.2D + W + 0.5L_f + 0.5(L_r or S or R)

Load combination no.3 1.2D + E + 0.5L_f + 0.7S

Load combination no.4 0.9D + W

Load combination no.5 0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1 K_{Ft} = **2.70**

Format conversion factor for compression – Table N1 K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1 K_{FE} = **1.76**

Resistance factor for tension – Table N2 ϕ_t = **0.80**

Resistance factor for compression – Table N2 ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2 ϕ_s = **0.85**

Time effect factor – Table N3 λ = **1.00**

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.50**

Size factor for compression – Table 4A C_{Fc} = **1.15**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3 C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 — C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 — C_T = **1.00**

Adjusted modulus of elasticity — $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value — $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **665** psi

Reference compression design value — $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3353** psi

For sawn lumber — c = **0.8**

Column stability factor – eqn.3.7-1 — $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.19**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio — 3.5

Shear wall length — b = **20.146** ft

Shear wall aspect ratio — h / b = **0.475**

Segmented shear wall capacity

Maximum shear force under seismic loading — $V_{s\_max} = E_q$ = **8.76** kips

Shear capacity for seismic loading — $V_s = \phi_D * v_s * b$ = **15.794** kips

V_{s_max} / V_s = **0.555**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio — h / b = **0.475**

Load combination 5

Shear force for maximum tension — $V = E_q$ = **8.76** kips

Axial force for maximum tension — P = 0 kips = **0** kips

Maximum tensile force in chord — T = V * h / (b) - P = **4.158** kips

Maximum applied tensile stress — $f_t = T / A_{en}$ = **554** lb/in^2

Design tensile stress — $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1863** lb/in^2

f_t / F_t' = **0.298**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression — $V = E_q$ = **8.76** kips

Axial force for maximum compression — $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.106** kips

Maximum compressive force in chord — C = V * h / (b) + P = **4.264** kips

Maximum applied compressive stress — $f_c = C / A_e$ = **406** lb/in^2

Design compressive stress — $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **636** lb/in^2

f_c / F_c' = **0.639**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 — T_1 = **4.158** kips

Chord 2 — T_2 = **4.158** kips

Seismic deflection

Design shear force — $V_{\delta s} = E_q$ = **8.76** kips

Deflection limit — Δ_{s_allow}= 0.020 * h = **2.295** in

Induced unit shear — $v_{\delta s}$ = $V_{\delta s}$ / b = **434.83** lb/ft

Anchor tension force — T_{δ} = max(0 kips,$v_{\delta s}$ * h) = **4.158** kips

Shear wall elastic deflection – Eqn. 4.3-1 — δ_{swse} = 2 * $v_{\delta s}$ * h^3 / (3 * E * A_e * b) + $v_{\delta s}$ * h / (G_a) + h * T_{δ} / (k_a * b) = **0.343** in

Deflection ampification factor — $C_{d\delta}$ = **4**

Seismic importance factor — I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 — δ_{sws} = $C_{d\delta}$ * δ_{swse} / I_e = **1.37** in

δ_{sws} / Δ_{s_allow} = **0.597**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

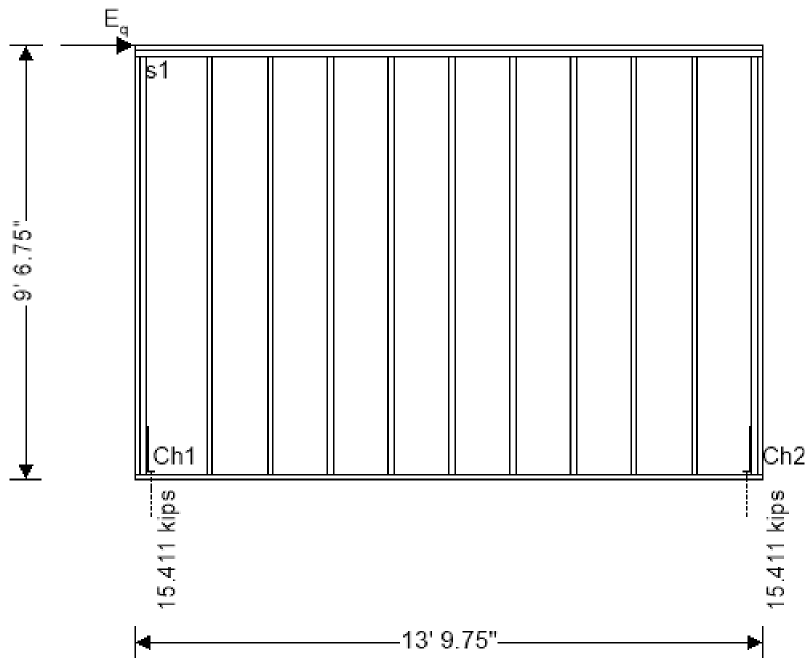
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

Panel details

Structural wood panel sheathing on both sides

Panel height	h = **9.563** ft
Panel length	b = **13.812** ft
Total area of wall	A = h * b = **132.082** ft^2



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **80000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity G = **0.50**

Tension parallel to grain F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Modulus of elasticity E = **1600000** lb/in^2

Minimum modulus of elasticity E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing

Fastener type 8d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **1280** lb/ft

Nominal unit shear capacity for wind design v_w = **1790** lb/ft

Apparent shear wall shear stiffness G_a = **20** kips/in

Combined unit shear capacities

Combined nominal unit shear capacity for seismic design

$v_{sc} = 2 * v_s$ = **2560** lb/ft

Combined nominal unit shear capacity for wind design

$v_{wc} = 2 * v_w$ = **3580** lb/ft

Combined apparent shear wall shear stiffness $G_{ac} = G_{a1} + G_{a2}$ = **40** kips/in

Loading details

Self weight of panel S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel E_q = **22260** lbs

Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 $0.9D + W$

Load combination no.5 $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2 ϕ_t = **0.80**

Resistance factor for compression – Table N2 ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3 λ = **1.00**

Sheathing resistance factor ϕ_D = **0.80**

Size factor for tension – Table 4A C_{Ft} = **1.30**

Size factor for compression – Table 4A C_{Fc} = **1.10**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1643** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.44**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **13.812** ft

Shear wall aspect ratio h / b = **0.692**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **22.26** kips

Shear capacity for seismic loading $V_s = \phi_D * v_{sc} * b$ = **28.288** kips

V_{s_max} / V_s = **0.787**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **0.692**

Load combination 5

Shear force for maximum tension $V = E_q$ = **22.26** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **15.411** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **1142** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.707**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **22.26** kips

Axial force for maximum compression $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.106** kips

Maximum compressive force in chord C = V * h / (b) + P = **15.517** kips

Maximum applied compressive stress $f_c = C / A_e$ = **940** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1418** lb/in^2

f_c / F_c' = **0.663**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 — T_1 = **15.411** kips

Chord 2 — T_2 = **15.411** kips

Seismic deflection

Design shear force — $V_{\delta s}$ = E_q = **22.26** kips

Deflection limit — Δ_{s_allow}= 0.020 * h = **2.295** in

Induced unit shear — $v_{\delta s}$ = $V_{\delta s}$ / b = **1611.58** lb/ft

Anchor tension force — T_δ = max(0 kips,$v_{\delta s}$ * h) = **15.411** kips

Shear wall elastic deflection – Eqn. 4.3-1 — δ_{swse} = 2 * $v_{\delta s}$ * h^3 / (3 * E * A_e * b) + $v_{\delta s}$ * h / (G_{ac}) + h * T_δ / (k_a * b) = **0.55** in

Deflection ampification factor — $C_{d\delta}$ = **4**

Seismic importance factor — I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 — δ_{sws} = $C_{d\delta}$ * δ_{swse} / I_e = **2.198** in

δ_{sws} / Δ_{s_allow} = **0.958**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

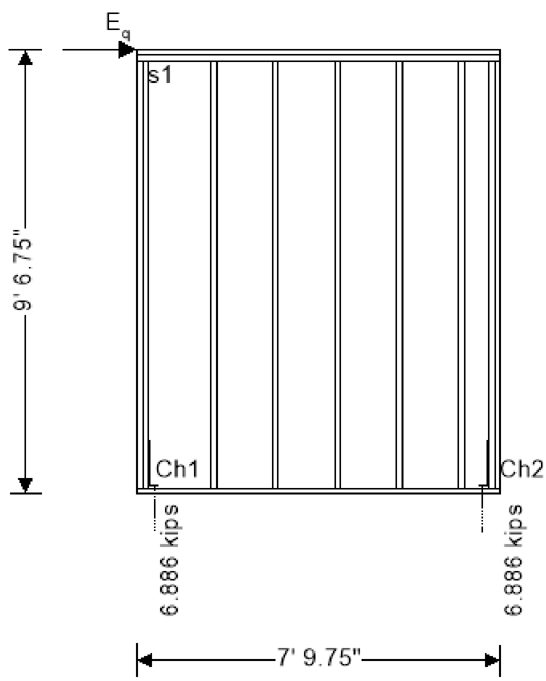
Panel details

Structural wood panel sheathing on one side

Panel height h = **9.563** ft

Panel length b = **7.813** ft

Total area of wall A = h * b = **74.707** ft^2



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **60000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity	G = **0.50**
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Minimum modulus of elasticity	E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material	15/32" wood panel 3-ply plywood sheathing
Fastener type	8d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = **1280** lb/ft
Nominal unit shear capacity for wind design	v_w = **1790** lb/ft
Apparent shear wall shear stiffness	G_a = **20** kips/in

Loading details

Self weight of panel	S_{wt} = **12** lb/ft^2
In plane seismic load acting at head of panel	E_q = **5626** lbs
Design spectral response accel. par., short periods	S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1	1.2D + 1.6(L_r or S or R) + 0.5W
Load combination no.2	1.2D + W + 0.5L_f + 0.5(L_r or S or R)
Load combination no.3	1.2D + E + 0.5L_f + 0.7S
Load combination no.4	0.9D + W
Load combination no.5	0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1	
	K_{Ft} = **2.70**
Format conversion factor for compression – Table N1	
	K_{Fc} = **2.40**
Format conversion factor for modulus of elasticity – Table N1	
	K_{FE} = **1.76**
Resistance factor for tension – Table N2	ϕ_t = **0.80**
Resistance factor for compression – Table N2	ϕ_c = **0.90**
Resistance factor for modulus of elasticity – Table N2	
	ϕ_s = **0.85**
Time effect factor – Table N3	λ = **1.00**
Sheathing resistance factor	ϕ_D = **0.80**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	
	C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1643** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3208** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.44**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **7.813** ft

Shear wall aspect ratio h / b = **1.224**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **5.626** kips

Shear capacity for seismic loading $V_s = \phi_D * v_s * b$ = **8** kips

V_{s_max} / V_s = **0.703**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **1.224**

Load combination 5

Shear force for maximum tension $V = E_q$ = **5.626** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **6.886** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **510** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1615** lb/in^2

f_t / F_t' = **0.316**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **5.626** kips

Axial force for maximum compression $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.106** kips

Maximum compressive force in chord C = V * h / (b) + P = **6.992** kips

Maximum applied compressive stress $f_c = C / A_e$ = **424** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **1418** lb/in^2

f_c / F_c' = **0.299**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **6.886** kips

Chord 2 T_2 = **6.886** kips

Seismic deflection

Design shear force	$V_{\delta s} = E_q =$ **5.626** kips
Deflection limit	$\Delta_{s\_allow} = 0.020 * h =$ **2.295** in
Induced unit shear	$v_{\delta s} = V_{\delta s} / b =$ **720.13** lb/ft
Anchor tension force	$T_\delta = \max(0 \text{ kips}, v_{\delta s} * h) =$ **6.886** kips
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_\delta / (k_a * b) =$ **0.509** in
Deflection ampification factor	$C_{d\delta} =$ **4**
Seismic importance factor	$I_e =$ **1**
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e =$ **2.037** in
	$\delta_{sws} / \Delta_{s\_allow} =$ **0.888**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

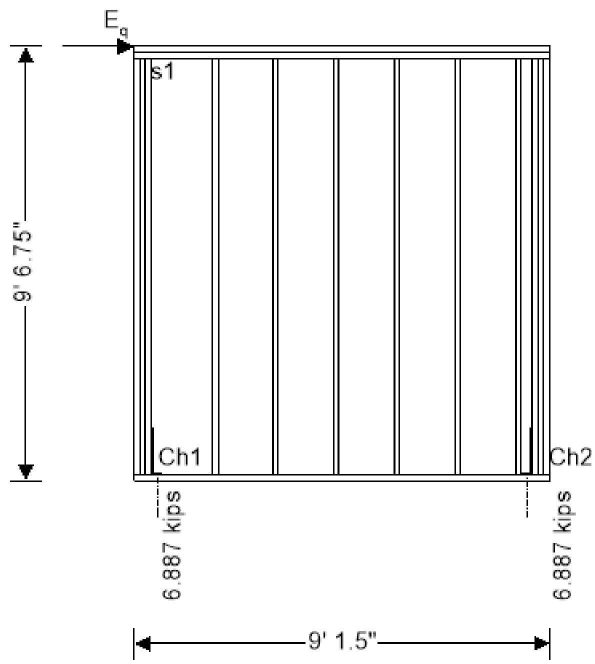
Panel details

Structural wood panel sheathing on one side

Panel height h = **9.563** ft

Panel length b = **9.125** ft

Total area of wall A = h * b = **87.258** ft^2



Panel construction

Nominal stud size	2" x 4"
Dressed stud size	1.5" x 3.5"
Cross-sectional area of studs	A_s = **5.25** in^2
Stud spacing	s = **16** in
Nominal end post size	3 x 2" x 4"
Dressed end post size	3 x 1.5" x 3.5"
Cross-sectional area of end posts	A_e = **15.75** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **11.25** in^2
Nominal collector size	2 x 2" x 4"
Dressed collector size	2 x 1.5" x 3.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **60000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity G = **0.50**
Tension parallel to grain F_t = **575** lb/in^2
Compression parallel to grain F_c = **1350** lb/in^2
Modulus of elasticity E = **1600000** lb/in^2
Minimum modulus of elasticity E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material 15/32" wood panel 3-ply plywood sheathing
Fastener type 8d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **1280** lb/ft
Nominal unit shear capacity for wind design v_w = **1790** lb/ft
Apparent shear wall shear stiffness G_a = **20** kips/in

Loading details

Self weight of panel S_{wt} = **12** lb/ft^2
In plane seismic load acting at head of panel E_q = **6572** lbs
Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 1.2D + 1.6(L_r or S or R) + 0.5W
Load combination no.2 1.2D + W + 0.5L_f + 0.5(L_r or S or R)
Load combination no.3 1.2D + E + 0.5L_f + 0.7S
Load combination no.4 0.9D + W
Load combination no.5 0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1
K_{Ft} = **2.70**
Format conversion factor for compression – Table N1
K_{Fc} = **2.40**
Format conversion factor for modulus of elasticity – Table N1
K_{FE} = **1.76**
Resistance factor for tension – Table N2 ϕ_t = **0.80**
Resistance factor for compression – Table N2 ϕ_c = **0.90**
Resistance factor for modulus of elasticity – Table N2
ϕ_s = **0.85**
Time effect factor – Table N3 λ = **1.00**
Sheathing resistance factor ϕ_D = **0.80**
Size factor for tension – Table 4A C_{Ft} = **1.50**
Size factor for compression – Table 4A C_{Fc} = **1.15**
Wet service factor for tension – Table 4A C_{Mt} = **1.00**
Wet service factor for compression – Table 4A C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A
C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 — C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 — C_T = **1.00**

Adjusted modulus of elasticity — $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value — $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **665** psi

Reference compression design value — $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3353** psi

For sawn lumber — c = **0.8**

Column stability factor – eqn.3.7-1 — $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.19**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio — 3.5

Shear wall length — b = **9.125** ft

Shear wall aspect ratio — h / b = **1.048**

Segmented shear wall capacity

Maximum shear force under seismic loading — $V_{s\_max} = E_q$ = **6.572** kips

Shear capacity for seismic loading — $V_s = \phi_D * v_s * b$ = **9.344** kips

V_{s_max} / V_s = **0.703**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio — h / b = **1.048**

Load combination 5

Shear force for maximum tension — $V = E_q$ = **6.572** kips

Axial force for maximum tension — P = 0 kips = **0** kips

Maximum tensile force in chord — T = V * h / (b) - P = **6.887** kips

Maximum applied tensile stress — $f_t = T / A_{en}$ = **612** lb/in^2

Design tensile stress — $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1863** lb/in^2

f_t / F_t' = **0.329**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression — $V = E_q$ = **6.572** kips

Axial force for maximum compression — $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.106** kips

Maximum compressive force in chord — C = V * h / (b) + P = **6.993** kips

Maximum applied compressive stress — $f_c = C / A_e$ = **444** lb/in^2

Design compressive stress — $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **636** lb/in^2

f_c / F_c' = **0.699**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 — T_1 = **6.887** kips

Chord 2 — T_2 = **6.887** kips

Seismic deflection

Design shear force — $V_{\delta s} = E_q$ = **6.572** kips

Deflection limit Δ_{s_allow}= 0.020 * h = **2.295** in

Induced unit shear $v_{\delta s}$ = $V_{\delta s}$ / b = **720.22** lb/ft

Anchor tension force T_{δ} = max(0 kips,$v_{\delta s}$ * h) = **6.887** kips

Shear wall elastic deflection – Eqn. 4.3-1 δ_{swse} = 2 * $v_{\delta s}$ * h^3 / (3 * E * A_e * b) + $v_{\delta s}$ * h / (G_a) + h * T_{δ} / (k_a * b) = **0.487** in

Deflection ampification factor $C_{d\delta}$ = **4**

Seismic importance factor I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 δ_{sws} = $C_{d\delta}$ * δ_{swse} / I_e = **1.946** in

δ_{sws} / Δ_{s_allow} = **0.848**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

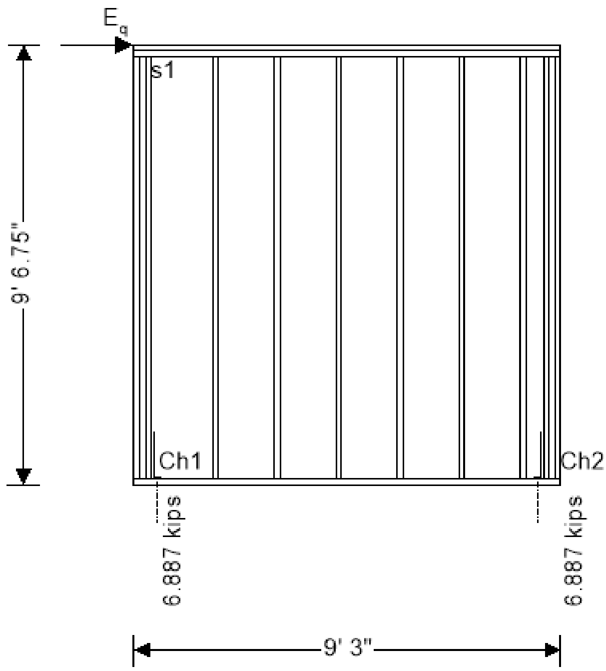
Panel details

Structural wood panel sheathing on one side

Panel height h = **9.563** ft

Panel length b = **9.25** ft

Total area of wall A = h * b = **88.453** ft²



Panel construction

Nominal stud size	2" x 4"
Dressed stud size	1.5" x 3.5"
Cross-sectional area of studs	A_s = **5.25** in²
Stud spacing	s = **16** in
Nominal end post size	3 x 2" x 4"
Dressed end post size	3 x 1.5" x 3.5"
Cross-sectional area of end posts	A_e = **15.75** in²
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **11.25** in²
Nominal collector size	2 x 2" x 4"
Dressed collector size	2 x 1.5" x 3.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **60000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification — Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity — G = **0.50**

Tension parallel to grain — F_t = **575** lb/in^2

Compression parallel to grain — F_c = **1350** lb/in^2

Modulus of elasticity — E = **1600000** lb/in^2

Minimum modulus of elasticity — E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material — 15/32" wood panel 3-ply plywood sheathing

Fastener type — 8d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = **1280** lb/ft

Nominal unit shear capacity for wind design — v_w = **1790** lb/ft

Apparent shear wall shear stiffness — G_a = **20** kips/in

Loading details

Self weight of panel — S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel — E_q = **6662** lbs

Design spectral response accel. par., short periods S_{DS} = **0.944**

From IBC 2018 cl.1605.2

Load combination no.1 — $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W$

Load combination no.2 — $1.2D + W + 0.5L_f + 0.5(L_r \text{ or } S \text{ or } R)$

Load combination no.3 — $1.2D + E + 0.5L_f + 0.7S$

Load combination no.4 — $0.9D + W$

Load combination no.5 — $0.9D + E$

Adjustment factors

Format conversion factor for tension – Table N1

K_{Ft} = **2.70**

Format conversion factor for compression – Table N1

K_{Fc} = **2.40**

Format conversion factor for modulus of elasticity – Table N1

K_{FE} = **1.76**

Resistance factor for tension – Table N2 — ϕ_t = **0.80**

Resistance factor for compression – Table N2 — ϕ_c = **0.90**

Resistance factor for modulus of elasticity – Table N2

ϕ_s = **0.85**

Time effect factor – Table N3 — λ = **1.00**

Sheathing resistance factor — ϕ_D = **0.80**

Size factor for tension – Table 4A — C_{Ft} = **1.50**

Size factor for compression – Table 4A — C_{Fc} = **1.15**

Wet service factor for tension – Table 4A — C_{Mt} = **1.00**

Wet service factor for compression – Table 4A — C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} * K_{FE} * \phi_s * C_{ME} * C_{tE} * C_i * C_T$ = **870000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **665** psi

Reference compression design value $F_c^* = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i$ = **3353** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.19**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **9.25** ft

Shear wall aspect ratio h / b = **1.034**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = E_q$ = **6.662** kips

Shear capacity for seismic loading $V_s = \phi_D * v_s * b$ = **9.472** kips

V_{s_max} / V_s = **0.703**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **1.034**

Load combination 5

Shear force for maximum tension $V = E_q$ = **6.662** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord T = V * h / (b) - P = **6.887** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **612** lb/in^2

Design tensile stress $F_t' = F_t * K_{Ft} * \phi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i$ = **1863** lb/in^2

f_t / F_t' = **0.329**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = E_q$ = **6.662** kips

Axial force for maximum compression $P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2$ = **0.106** kips

Maximum compressive force in chord C = V * h / (b) + P = **6.993** kips

Maximum applied compressive stress $f_c = C / A_e$ = **444** lb/in^2

Design compressive stress $F_c' = F_c * K_{Fc} * \phi_c * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_i * C_P$ = **636** lb/in^2

f_c / F_c' = **0.699**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **6.887** kips

Chord 2 T_2 = **6.887** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q =$ **6.662** kips

Deflection limit $\Delta_{s\_allow} = 0.020 * h =$ **2.295** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b =$ **720.22** lb/ft

Anchor tension force $T_{\delta} = \max(0 \text{ kips}, v_{\delta s} * h) =$ **6.887** kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_{\delta} / (k_a * b) =$ **0.485** in

Deflection ampification factor $C_{d\delta} =$ **4**

Seismic importance factor $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e =$ **1.939** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.845**

PASS - Shear wall deflection is less than deflection limit

3.2 | STEEL MOMENT FRAME DESIGN

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Moment Frame Gravity Loading	**DATE:**	2021-03-03
DESIGN BY:	BJW		

NOTES: MAIN LEVEL

GEOMETRY:

Tributary width	w_T =	14.208	ft
Beam length	L =	26.58	ft

SURFACE LOADS:

Dead load	DL =	0	psf		
Superimposed dead load	SDL =	30	psf	1.83	12.375
Live load	LL avg =	42.58	psf	60	40
Snow load	SL =	3.87	psf	30	0

LINE LOADS:

Dead load	DL =	**0**	plf	**0.00**	klf
Superimposed dead load	SDL =	**426.25**	plf	**0.43**	klf
Live load	LL =	**605**	plf	**0.61**	klf
Snow load	SL =	**55.000**	plf	**0.06**	klf

REACTIONS:

	RDL =	**0.00**	kips
Girder reaction	RSDL =	**5.67**	kips
	RLL =	**8.04**	kips
	RSL =	**0.73**	kips

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Moment Frame Gravity Loading	**DATE:**	2021-03-03
DESIGN BY:	BJW		

NOTES: UPPER LEVEL

GEOMETRY:

Tributary width	w_T =	6.917	ft
Beam length	L =	26.58	ft

SURFACE LOADS:

Dead load	DL =	0	psf
Superimposed dead load	SDL =	30	psf
Live load	LL =	40	psf
Snow load	SL =	30	psf

LINE LOADS:

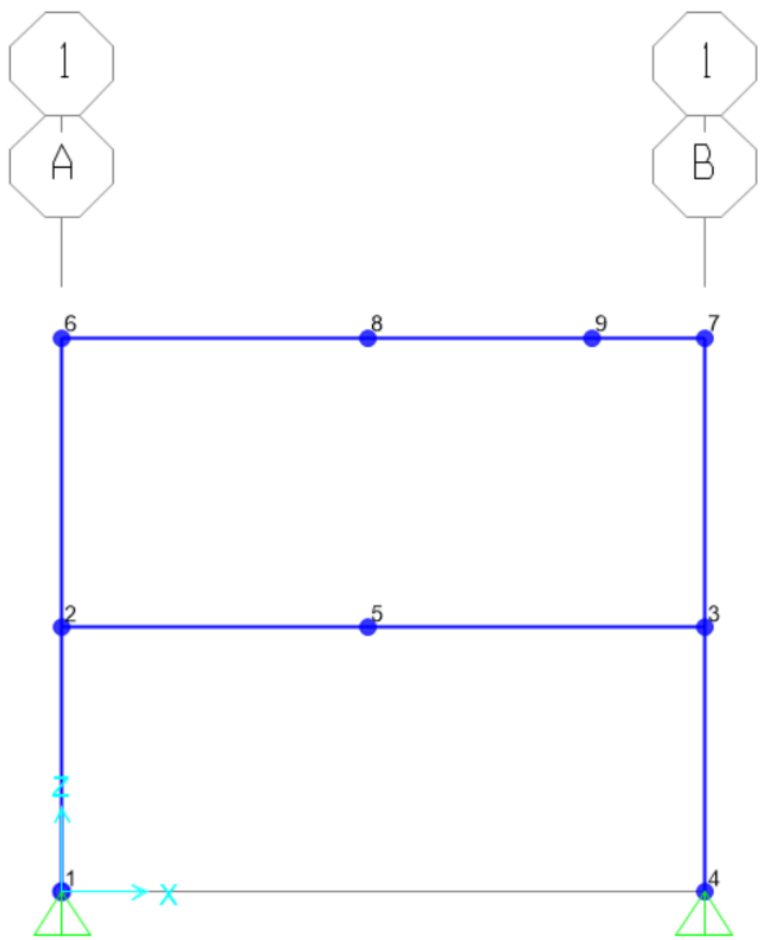
Dead load	DL =	**0**	plf	**0.00**	klf	
Superimposed dead load	SDL =	**407.5**	plf	**0.41**	klf	(Includes 200 plf for door)
Live load	LL =	**276.667**	plf	**0.28**	klf	
Snow load	SL =	**207.500**	plf	**0.21**	klf	

POINT LOADS (SEE TB1 TEDDS FILE): ACTING 4'-8" FROM RIGHT SUPPORT

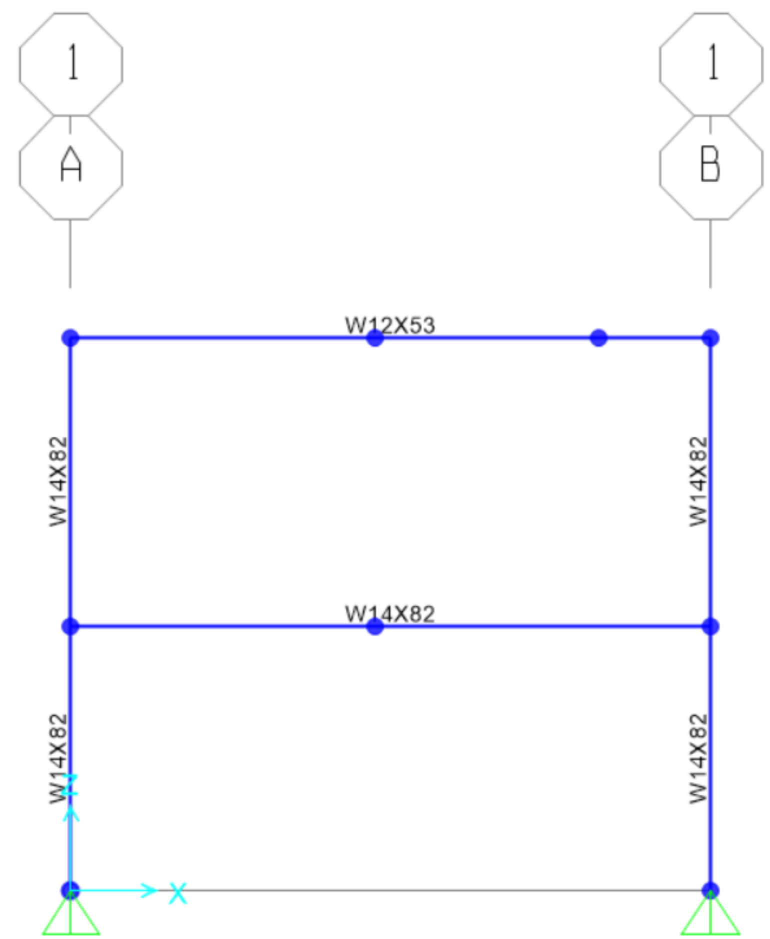
Dead load	DL =	0	kips
Superimposed dead load	SDL =	13.1	kips
Live load	LL =	9.8	kips
Snow load	SL =	5.6	kips

REACTIONS:

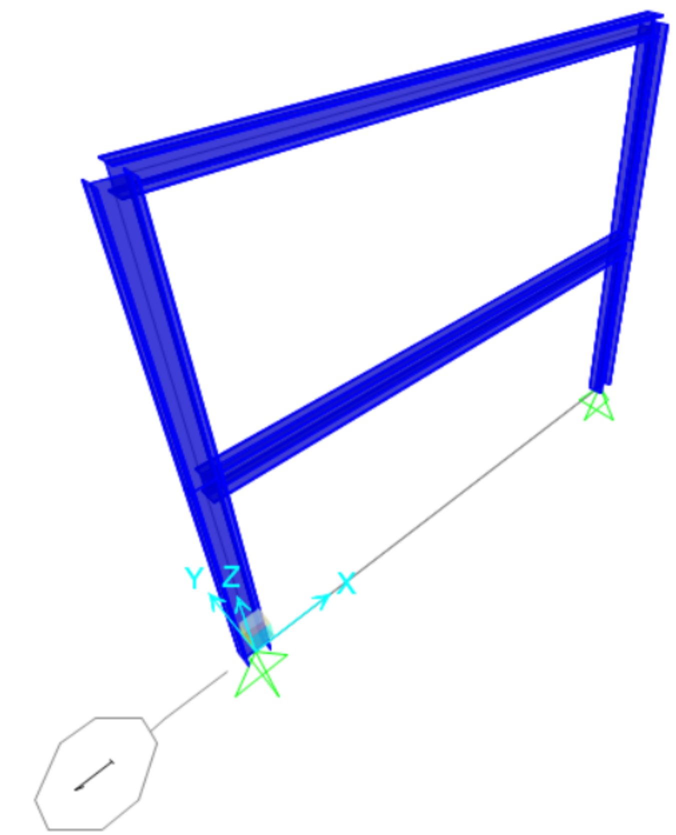
	RDL =	**0.00**	kips
Girder reaction	RSDL =	**5.42**	kips
	RLL =	**3.68**	kips
	RSL =	**2.76**	kips



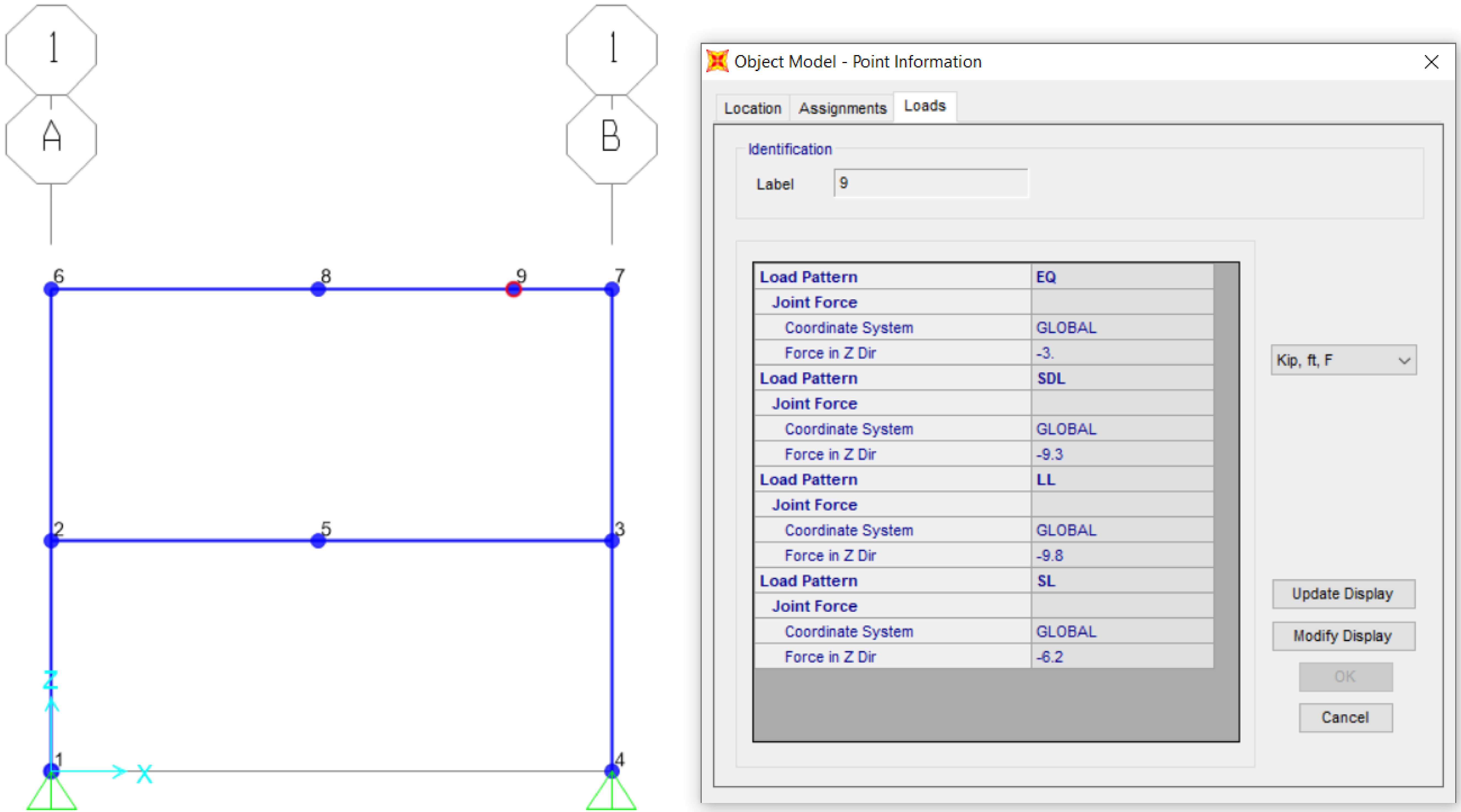
JOINT LABELS



FRAME SECTION ASSIGNMENTS

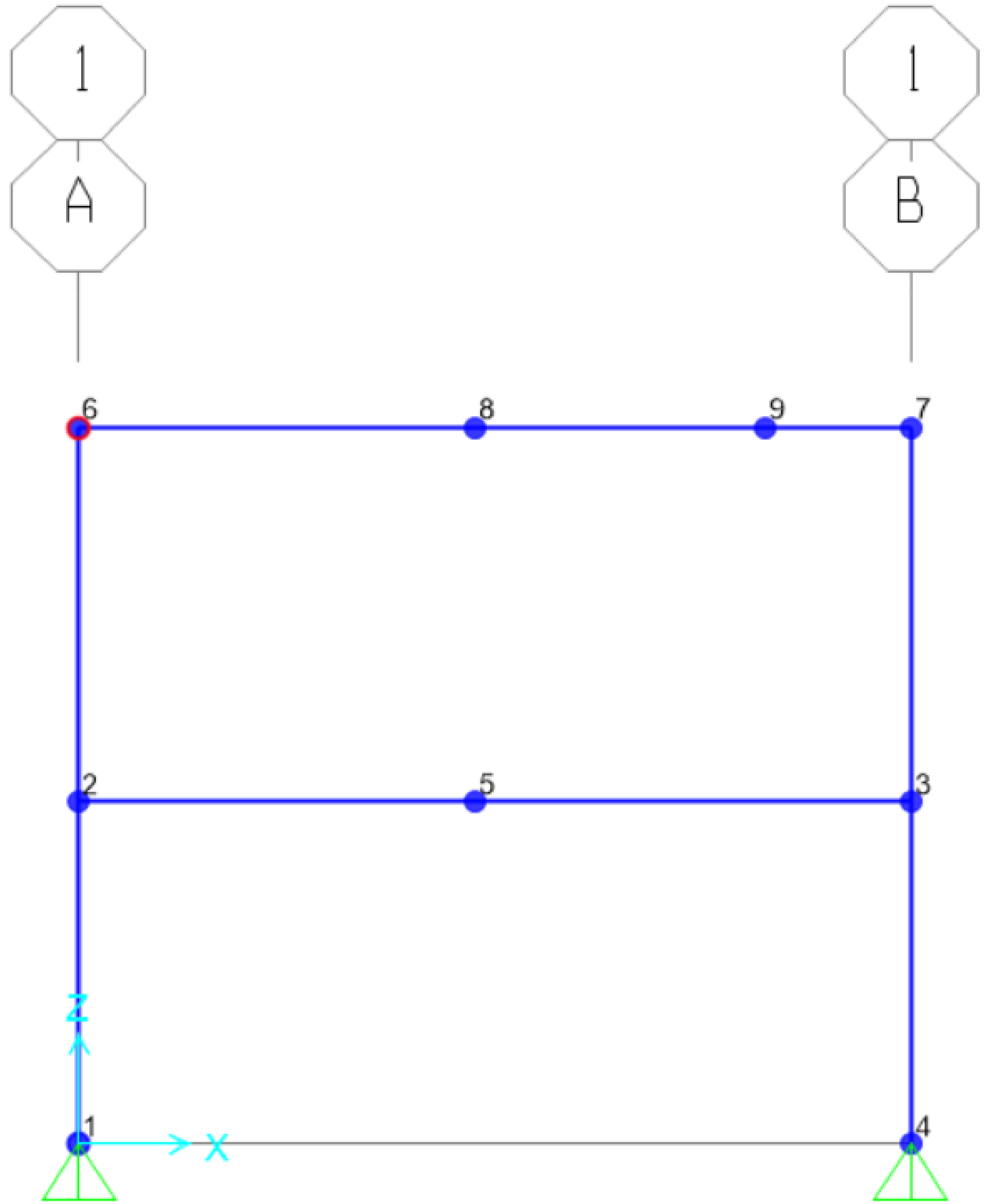


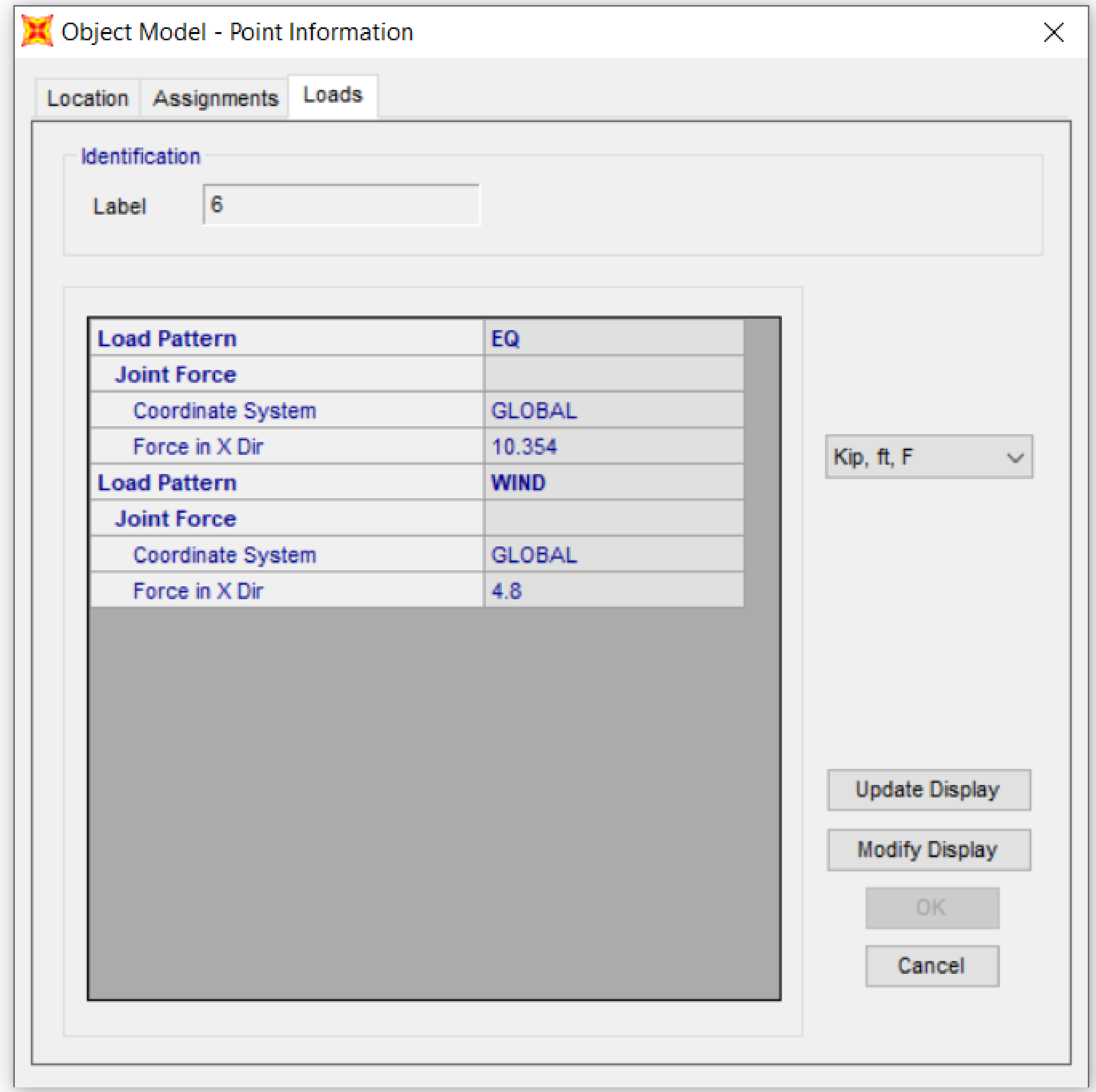
3D MODEL VIEW



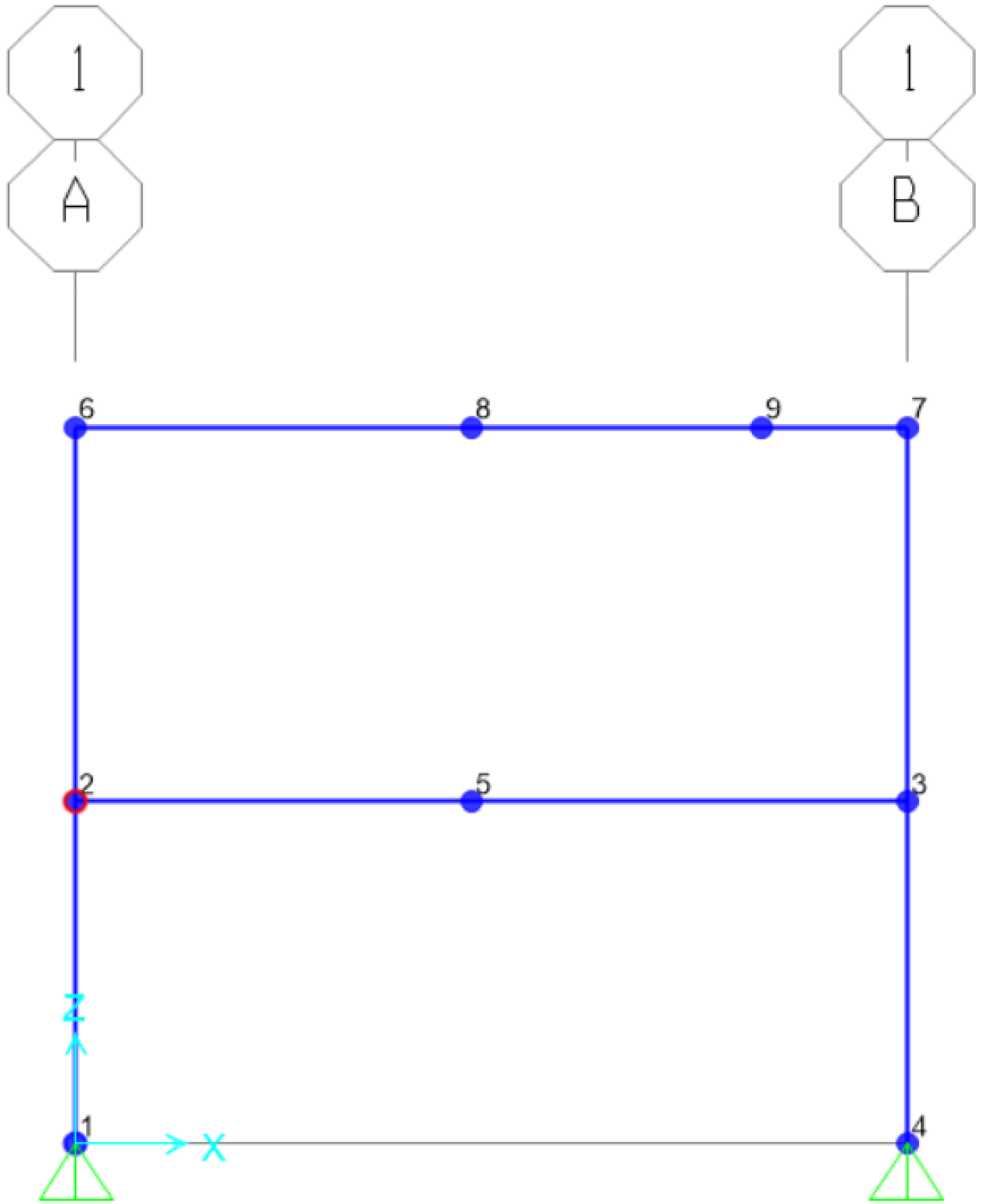
Load Pattern	EQ
Joint Force	
Coordinate System	GLOBAL
Force in Z Dir	-3.
Load Pattern	SDL
Joint Force	
Coordinate System	GLOBAL
Force in Z Dir	-9.3
Load Pattern	LL
Joint Force	
Coordinate System	GLOBAL
Force in Z Dir	-9.8
Load Pattern	SL
Joint Force	
Coordinate System	GLOBAL
Force in Z Dir	-6.2

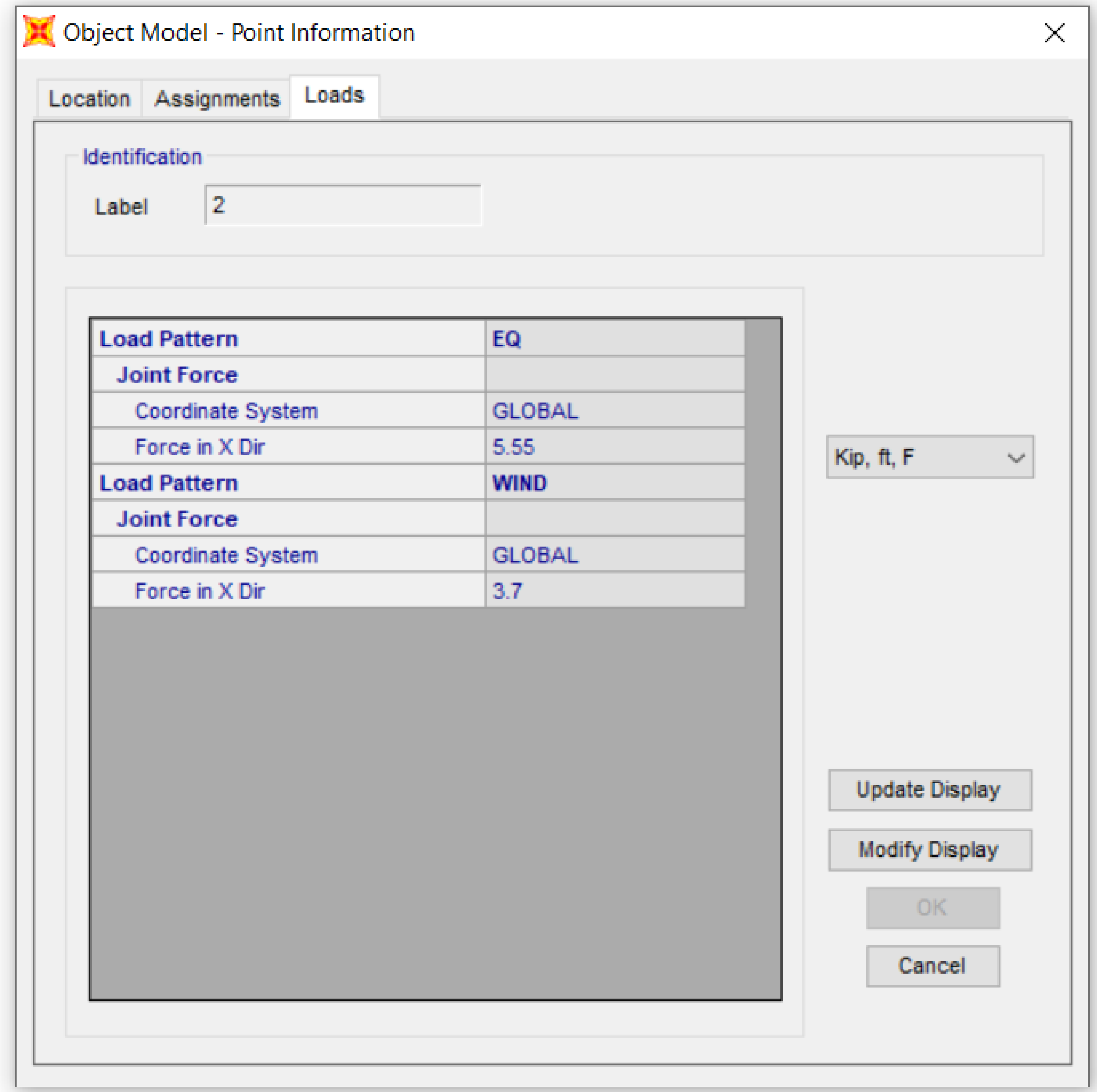
JOINT 9 LOADING - POINT LOAD FROM STEEL TRANSFER BEAM



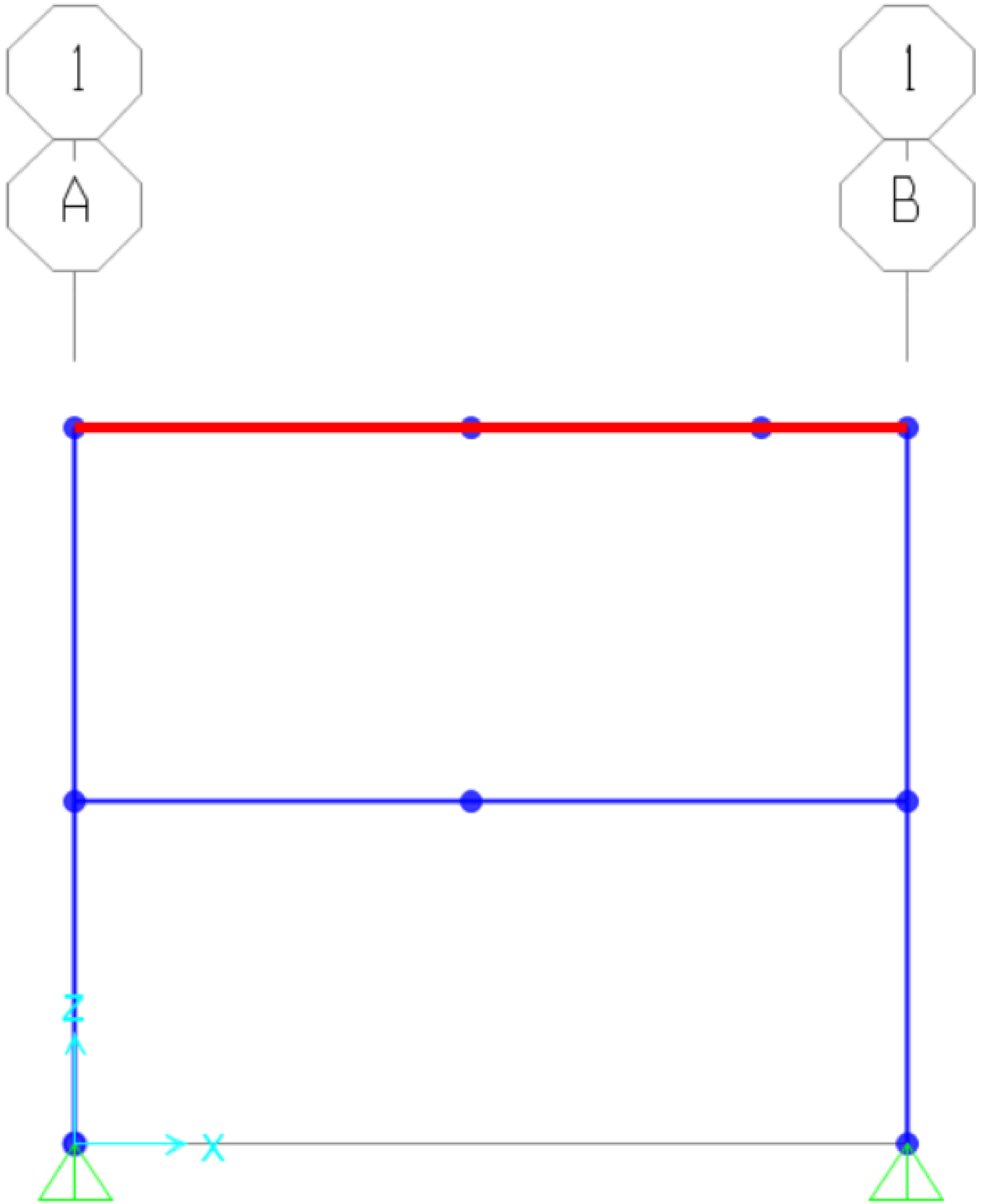


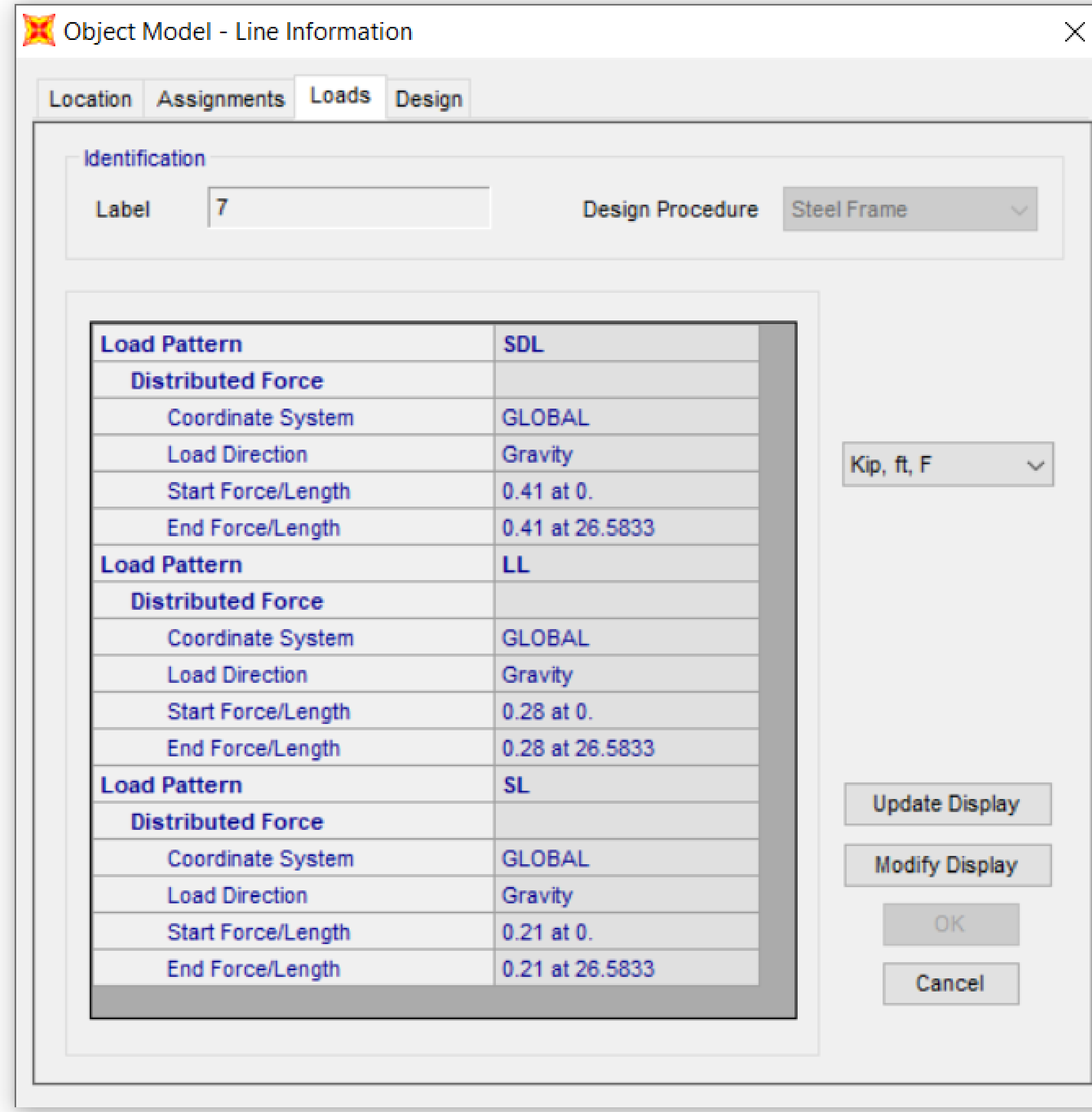
JOINT 6 LOADING





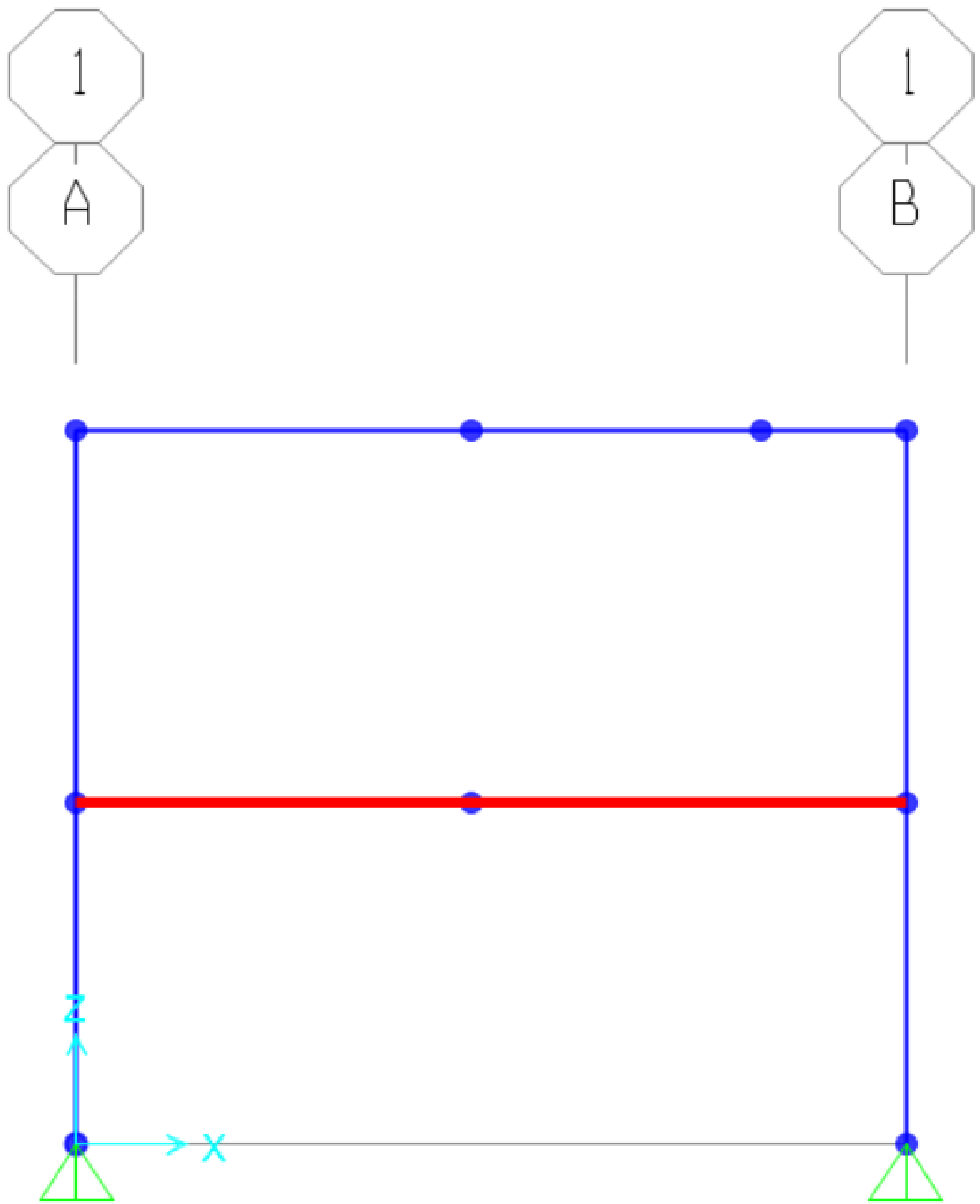
JOINT 2 LOADING

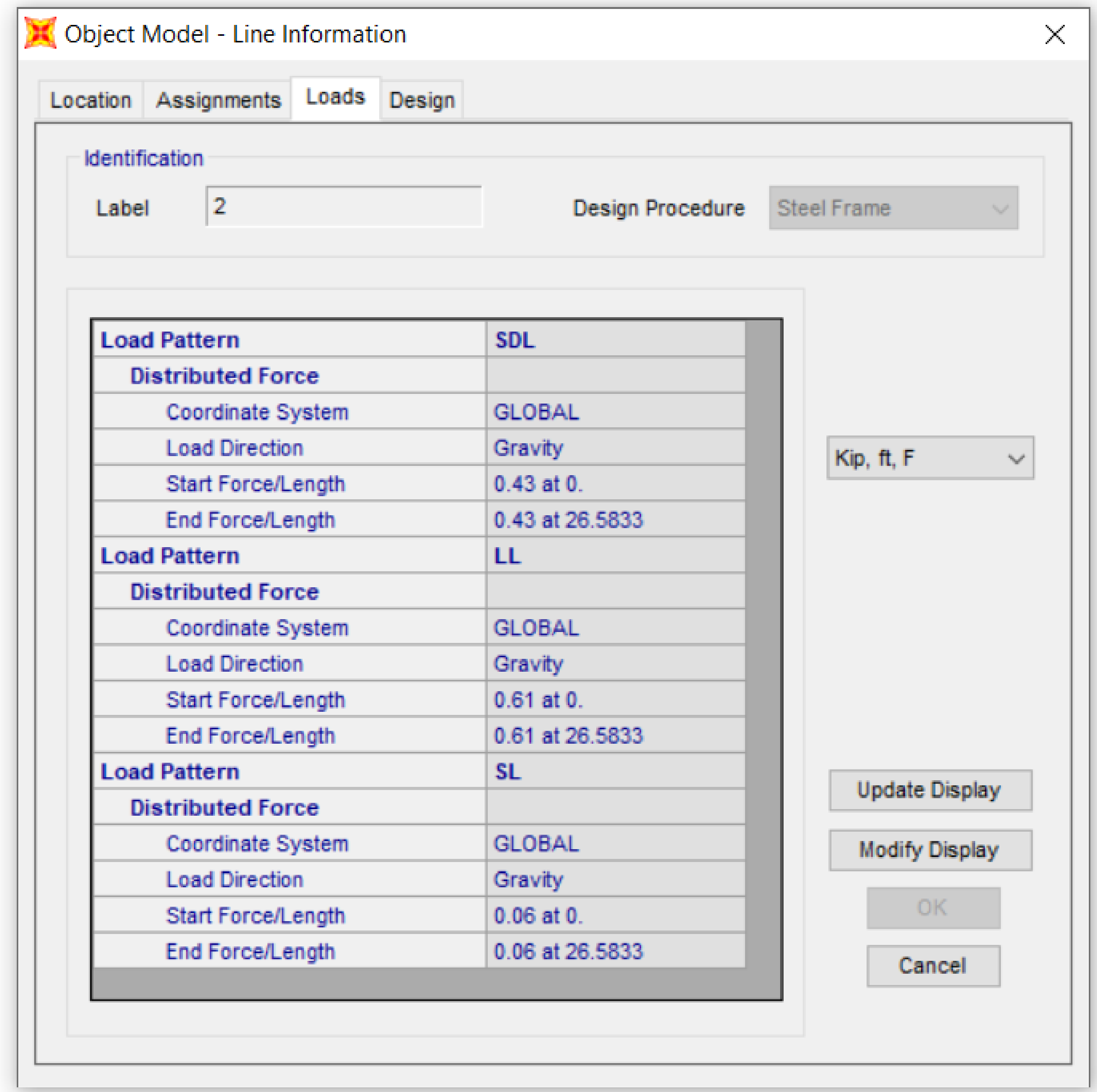




Load Pattern	**SDL**
Distributed Force	
Coordinate System	GLOBAL
Load Direction	Gravity
Start Force/Length	0.41 at 0.
End Force/Length	0.41 at 26.5833
Load Pattern	**LL**
Distributed Force	
Coordinate System	GLOBAL
Load Direction	Gravity
Start Force/Length	0.28 at 0.
End Force/Length	0.28 at 26.5833
Load Pattern	**SL**
Distributed Force	
Coordinate System	GLOBAL
Load Direction	Gravity
Start Force/Length	0.21 at 0.
End Force/Length	0.21 at 26.5833

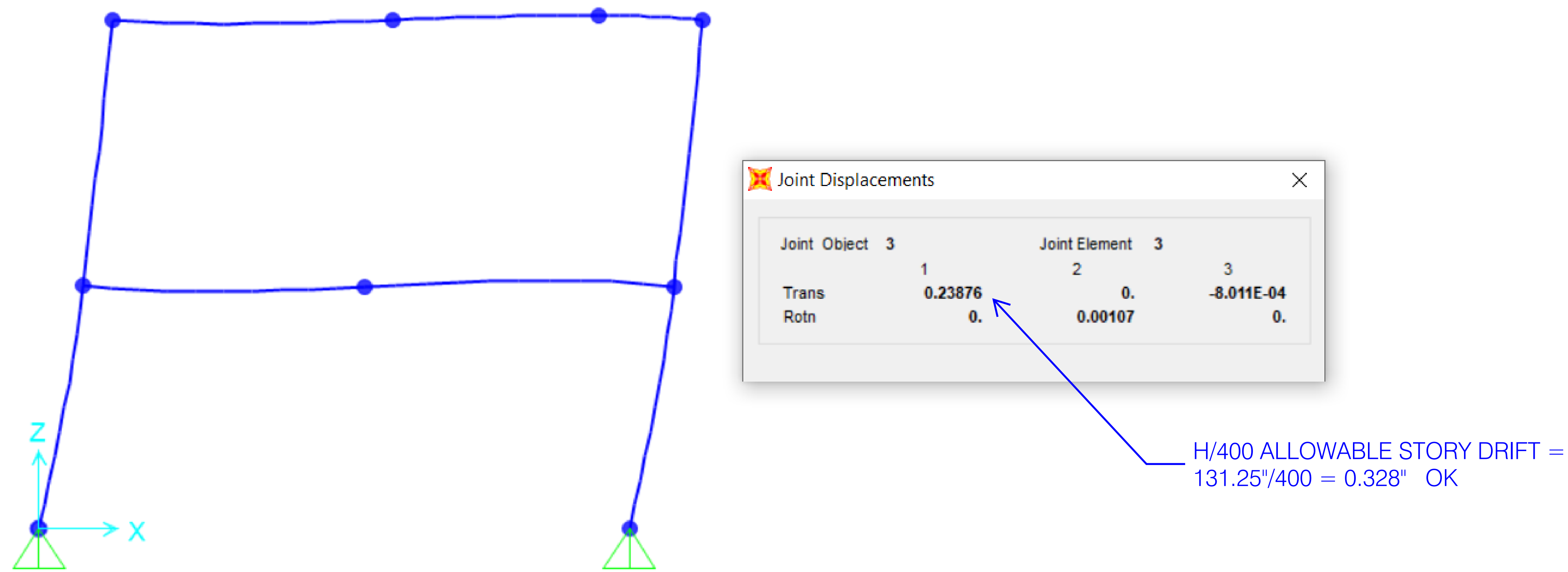
UPPER FRAME ELEMENT LOADING



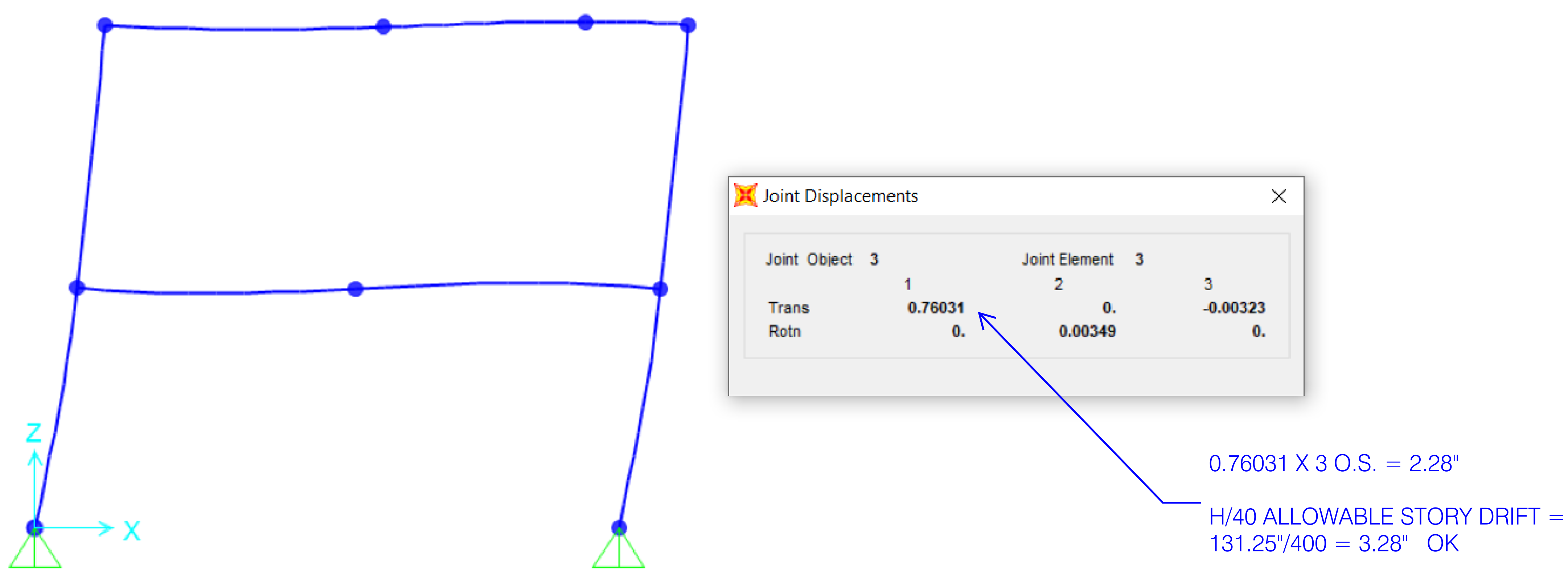


Load Pattern	**SDL**
Distributed Force	
Coordinate System	GLOBAL
Load Direction	Gravity
Start Force/Length	0.43 at 0.
End Force/Length	0.43 at 26.5833
Load Pattern	**LL**
Distributed Force	
Coordinate System	GLOBAL
Load Direction	Gravity
Start Force/Length	0.61 at 0.
End Force/Length	0.61 at 26.5833
Load Pattern	**SL**
Distributed Force	
Coordinate System	GLOBAL
Load Direction	Gravity
Start Force/Length	0.06 at 0.
End Force/Length	0.06 at 26.5833

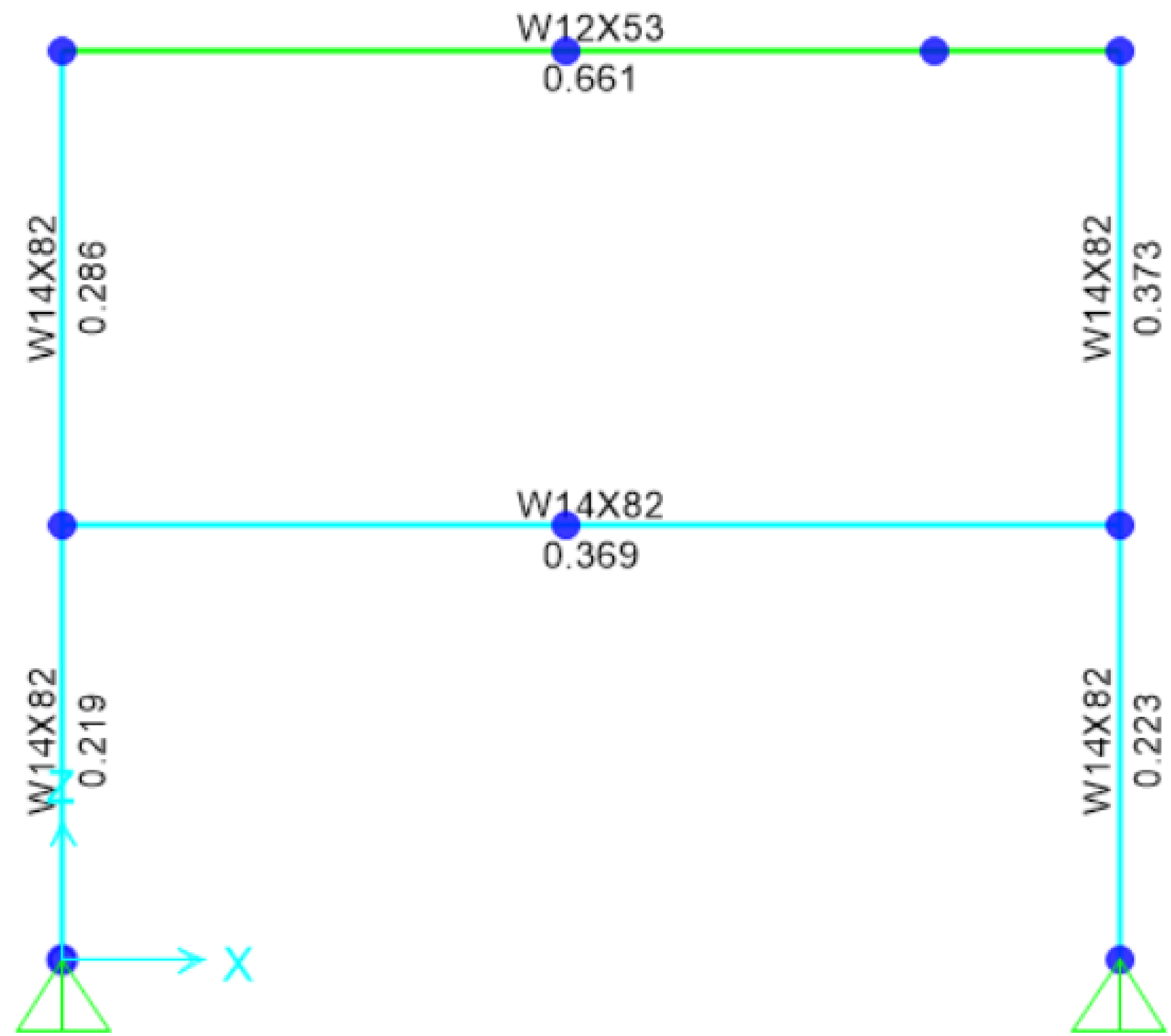
LOWER FRAME ELEMENT LOADING



DEFORMED SHAPE - 0.6 W



DEFORMED SHAPE - 1.0 EQ



STRENGTH DESIGN CHECK (PER AISC 360)

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Moment Frame Base Plate	Date:	3/3/2021
Fastening point:			

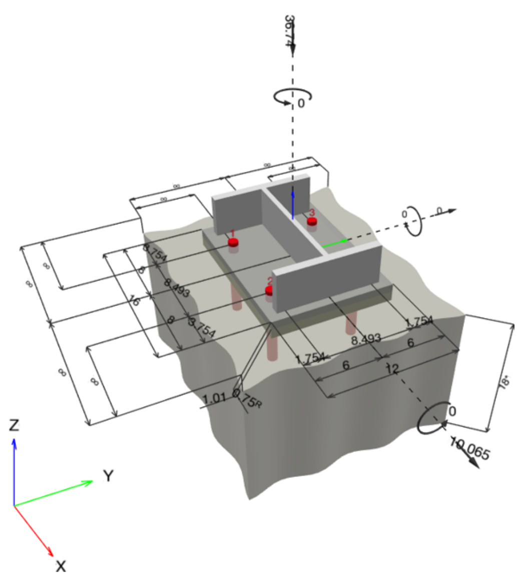
Specifier's comments:

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1**
Item number:	not available
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-11 / CIP
Stand-off installation:	without clamping (anchor); restraint level (anchor plate): 2.00; e_b = 1.010 in.; t = 0.750 in.
	Hilti Grout: CB-G EG, epoxy, $f_{c,Grout}$ = 14,939 psi
Anchor plate[R] :	l_x x l_y x t = 16.000 in. x 12.000 in. x 0.750 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W14X82; (L x W x T x FT) = 14.300 in. x 10.100 in. x 0.510 in. x 0.855 in.
Base material:	cracked concrete, 5000, f_c' = 5,000 psi; h = 18.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: no
	Shear load: yes (D.3.3.5.3 (a))

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

Geometry [in.] & Loading [kip, ft.kip]



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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Moment Frame Base Plate	Date:	3/3/2021
Fastening point:			

1.1 Unfactored loads

	Sustained load factor	Load factor f_1 or f_2	V_x [kip]	V_y [kip]	N [kip]	M_x [ft.kip]	M_y [ft.kip]	M_z [ft.kip]
D (Dead)	1.000	-	1.180	-	-30.000	-	-	-
F (Fluid)	1.000	-	-	-	-	-	-	-
T (Temperature)	1.000	-	-	-	-	-	-	-
L (Live)	1.000	0.500	1.210	-	-20.000	-	-	-
H (Lateral)	1.000	-	-	-	-	-	-	-
L_r (Roof live)	1.000	-	-	-	-	-	-	-
S (Snow)	1.000	0.200	0.220	-	-8.700	-	-	-
R (Rain)	-	-	-	-	-	-	-	-
W (Wind)	-	-	4.300	-	5.700	-	-	-
E (Earthquake)	-	-	8.000	-	11.000	-	-	-

1.2 Design results

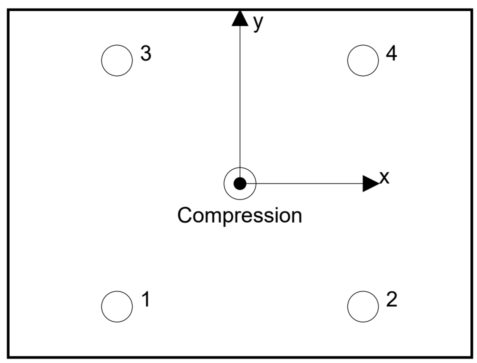
Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Load case: Design loads	N = -42.000; V_x = 1.652; V_y = 0.000; M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;	yes	14

2 Load case/Resulting anchor forces

Anchor reactions [kip]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	2.516	2.516	0.000
2	0.000	2.516	2.516	0.000
3	0.000	2.516	2.516	0.000
4	0.000	2.516	2.516	0.000



max. concrete compressive strain:	0.04 [‰]
max. concrete compressive stress:	191 [psi]
resulting tension force in (x/y)=(0.000/0.000):	0.000 [kip]
resulting compression force in (x/y)=(0.000/0.000):	36.740 [kip]

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

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

4 Shear load

	Load V_{ua} [kip]	Capacity ϕ V_n [kip]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2.516	10.966	23	OK
Steel failure (with lever arm)*	2.516	3.047	83	OK
Pryout Strength**	10.065	75.642	14	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

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

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-11 Eq. (D-29)

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-11 Table D.4.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

V_{sa} [kip]
21.089

Results

V_{sa} [kip]	ϕ_{steel}	ϕ_{eb}	$\phi\ V_{sa,eq}$ [kip]	V_{ua} [kip]
21.089	0.650	0.800	10.966	2.516

4.2 Steel failure (with lever arm)

V_s^M	$= \frac{\alpha_M \cdot M_s}{L_b}$	bending equation for stand-off
M_s	$= M_s^0 \left(1 - \frac{N_{ua}}{\phi\, N_{sa}}\right)$	resultant flexural resistance of anchor
M_s^0	$= (1.2)\ (S)\ (f_{u,min})$	characteristic flexural resistance of anchor
$\left(1 - \frac{N_{ua}}{\phi\, N_{sa}}\right)$		reduction for tensile force acting simultaneously with a shear force on the anchor
S	$= \frac{\pi (d)^3}{32}$	elastic section modulus of anchor bolt at concrete surface
L_b	$= z + (n)(d_0)$	internal lever arm adjusted for spalling of the surface concrete
$\phi\, V_s^M$	$\geq V_{ua}$	ACI 318-11 Table D.4.1.1

Variables

α_M	$f_{u,min}$ [psi]	N_{ua} [kip]	$\phi\, N_{sa}$ [kip]	z [in.]	n	d_0 [in.]
2.00	58,000	0.000	26.361	1.385	0.500	1.000

Calculations

M_s^0 [ft.kip]	$\left(1 - \frac{N_{ua}}{\phi\, N_{sa}}\right)$	M_s [ft.kip]	L_b [in.]
0.36815	1.000	0.36815	1.885

Results

V_s^M [kip]	ϕ_{steel}	$\phi\, V_s^M$ [kip]	V_{ua} [kip]
4.687	0.650	3.047	2.516

4.3 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$ ACI 318-11 Eq. (D-41)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-11 Table D.4.1.1

A_{Nc} see ACI 318-11, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-11 Eq. (D-5)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}} \right) \leq 1.0$ ACI 318-11 Eq. (D-8)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-11 Eq. (D-10)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-11 Eq. (D-12)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-11 Eq. (D-6)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	5,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
701.87	324.00	1.000	1.000	1.000	1.000	24.942

Results

V_{cpg} [kip]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [kip]	V_{ua} [kip]
108.061	0.700	1.000	1.000	75.642	10.065

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2018, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- ACI 318 does not specifically address anchor bending when a stand-off condition exists. PROFIS Engineering calculates a shear load corresponding to anchor bending when stand-off exists and includes the results as a shear Design Strength!
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-11 Appendix D. The connection design (shear) shall satisfy the provisions of Part D.3.3.5.3 (a), Part D.3.3.5.3 (b), or Part D.3.3.5.3 (c).
- Part D.3.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Part D.3.3.5.3 (b) waive the ductility requirements and requires that the anchors shall be designed for the maximum shear that can be transmitted to the anchors by a non-yielding attachment. Part D.3.3.5.3 (c) waives the ductility requirements and requires the design strength of the anchors to equal or exceed the maximum shear obtained from design load combinations that include E, with E increased by ω_0.

Fastening meets the design criteria!

6 Installation data

Profile: W shape (AISC), W14X82; (L x W x T x FT) = 14.300 in. x 10.100 in. x 0.510 in. x 0.855 in.

Hole diameter in the fixture: d_f = 1.062 in.

Plate thickness (input): 0.750 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1

Item number: not available

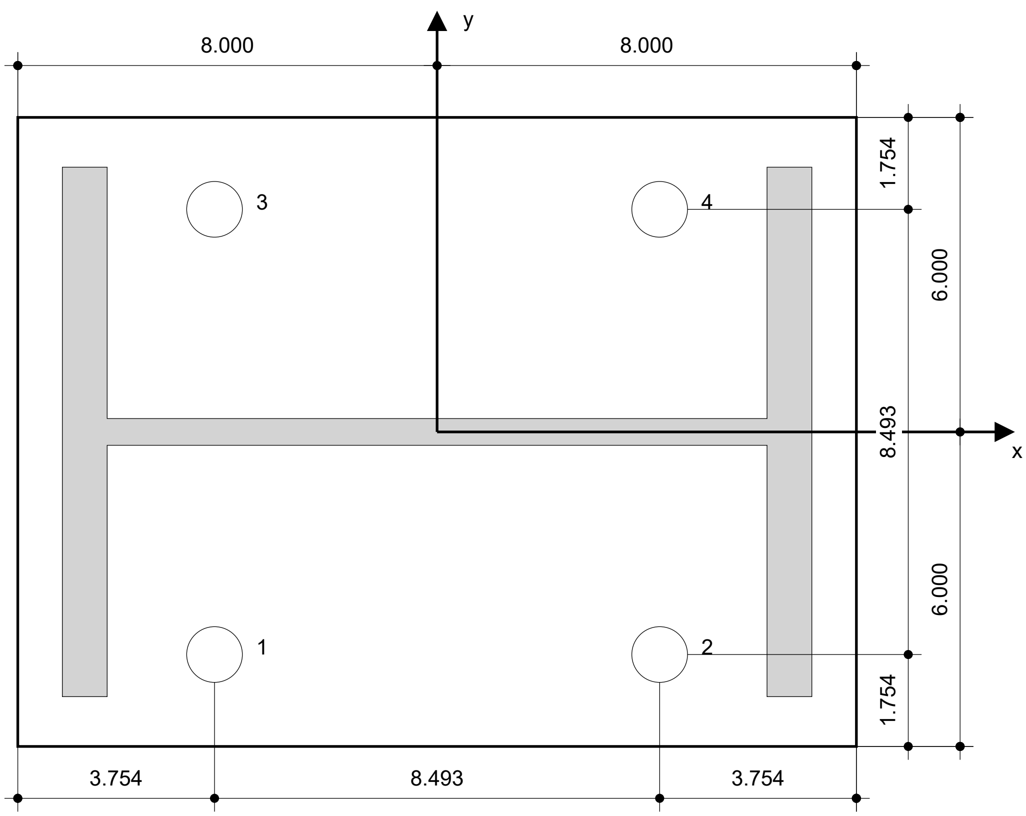
Installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 6.000 in.

Minimum thickness of the base material: 7.172 in.

Hilti Hex Head headed stud anchor with 6 in embedment, 1, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-4.246	-4.246	-	-	-	-
2	4.246	-4.246	-	-	-	-
3	-4.246	4.246	-	-	-	-
4	4.246	4.246	-	-	-	-

7 Remarks; Your Cooperation Duties

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3.3 | DIAPHRAGM DESIGN

Level: **High Roof**
H = 9.5 ft
Typ. Diaphragm Unblocked, 8d 19/32

GRID	Vu (kips)	ϕvs (k/ft)	Lreq (ft)	Lwall (ft)	Check	Notes	ϕvs req table (lb/ft)	Blocking	ϕvs table (lb/ft)	Drag Strap (min. ft)	Drag Strap Demand (kips)
A	1.44	0.384	3.75	5.92	OK	Case 1	304	-	-	-	-
B	3.36	0.384	8.75	11.38	OK	Case 1	369	-	-	-	-
C	3.36	0.384	8.75	12.58	OK	Case 1	334	-	-	-	-
1	4	0.288	13.89	16.88	OK	Case 3	296	-	-	-	-
2	4	0.288	13.89	16.38	OK	Case 3	305	-	-	-	-

Level: **Upper Level**
H = 10 ft
Typ. Diaphragm Unblocked, 8d 15/32

GRID	Vu (kips)	ϕvs (k/ft)	Lreq (ft)	Lwall (ft)	Check	Notes	ϕvs req table (lb/ft)	Blocking	ϕvs table (lb/ft)	Drag Strap (min. ft)	Drag Strap Demand (kips)
A	1.14	0.288	3.96	4.90	OK	Case 2-6	291	-	-	-	-
B	9.2	0.288	31.94	25.40	NG	Case 2-6	453	B to C, 15/32 8d @ 6"	540	-	-
C	10.836	0.288	37.63	16.83	NG	Case 2-6	805	-	-	20.79	6.0
D	3.69	0.288	12.81	11.08	NG	Case 2-6	416	-	-	1.73	0.5
E	2.83	0.288	9.83	29.00	OK	Case 2-6	122	-	-	Engage garage wall	2.8
F	6.19	0.288	21.49	7.83	NG	Case 2-6	988	E to F, 15/32 8d @ 2.5"	1060	-	-
1	2.78	0.288	9.65	13.35	OK	Case 2-6	260	-	-	-	-
2	9	0.288	31.25	13.42	NG	Case 2-6	839			17.83	5.1
3	7.4	0.288	25.69	13.23	NG	Case 2-6	699				
4	2.04	0.288	7.08	4.13	NG	Case 2-6	618			2.96	0.9

Level: **Main Level**
H = 9.56 ft
Typ. Diaphragm Blocked, 8d 15/32 @ 2.5"

GRID	Vu (kips)	ϕvs (k/ft)	Lreq (ft)	Lwall (ft)	Check	Notes	ϕvs req table (lb/ft)	Blocking	ϕvs table (lb/ft)	Drag Strap (min. ft)	Drag Strap Demand (kips)
A	3.05	0.848	3.60	25.40	OK	Case 2-6	150	-	-	-	-
B	15.8	0.848	18.63	26.19	OK	Case 2-6	754	-	-	-	-
C	11.79	0.848	13.90	16.08	OK	Case 2-6	916	-	-	-	-
1	16.68	0.848	19.67	14.56	NG	Case 2-6	1432	-	-	5.11	4.3
2	17.19	0.848	20.27	13.81	NG	Case 2-6	1556	-	-	6.46	5.5
3	7.22	0.848	8.51	20.15	OK	Case 2-6	448	-	-	-	-
4	17.76	0.848	20.94	13.65	NG	Case 2-6	1627	-	-	7.30	6.2
5	1.35	0.848	1.59	20.19	OK	Case 2-6	84	-	-	-	-
6	0.25	0.848	0.29	4.15	OK	Case 2-6	75	-	-	-	-

3.4 | CONNECTOR DESIGN

ASD to LRFD Adjustment Factors

K_F =	3.32
ϕ =	0.65
λ =	1
C_D =	1.6

SST HOLDDOWNS		
MODEL NO.	ALLOWABLE TENSION LOADS (lbs)	
	ASD	LRFD
HDU2-SDS2.5	3075	4147
HDU4-SDS2.5	4565	6157
HDU5-SDS2.5	5645	7614
HDU8-SDS2.5	6765	9124

SST FLOOR TO FLOOR STRAPS		
MODEL NO.	ALLOWABLE TENSION LOADS (lbs)	
	ASD	LRFD
CMSTC16	4690	6326
CMST14	6475	8733
CMST12	9215	12429

SST HANGERS		
MODEL NO.	ALLOWABLE TENSION LOADS (lbs)	
	ASD	LRFD
HHUS5.50/10	2825	3810
MGU5.50-SDS (5 1/4)	7260	9792
HDU5-SDS2.5	5645	7614
HDU8-SDS2.5	6765	9124

4 | FOUNDATION DESIGN

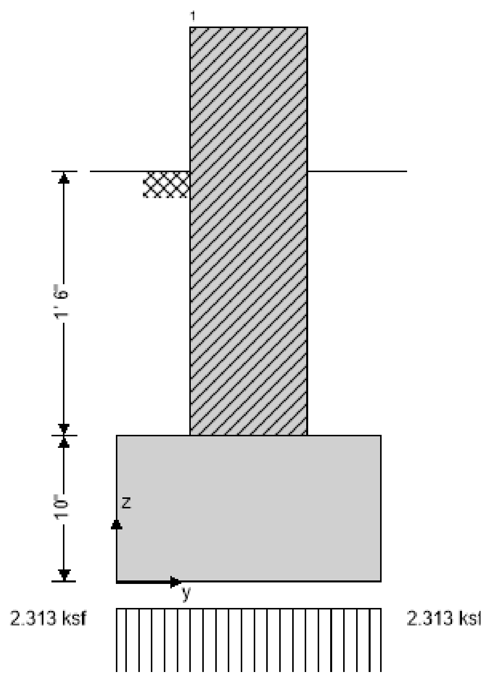
4.1 | FOOTING AND FOUNDATION WALL DESIGN

Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.10

FOOTING ANALYSIS

Length of foundation	L_x = **1** ft
Width of foundation	L_y = **1.5** ft
Foundation area	$A = L_x \times L_y$ = **1.5** ft^2
Depth of foundation	h = **10** in
Depth of soil over foundation	h_{soil} = **18** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Wall no.1 details

Width of wall	l_{y1} = **8** in
position in y-axis	y_1 = **9** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2.5** ksf
Density of soil	γ_{soil} = **125.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **19.3** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.350**

Foundation loads

Self weight	$F_{swt} = h * \gamma_{conc}$ = **125** psf
Soil weight	$F_{soil} = h_{soil} * \gamma_{soil}$ = **187.5** psf

Wall no.1 loads per linear foot

Dead load in z — F_{Dz1} = **1.5** kips

Live load in z — F_{Lz1} = **1.5** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.525)

1.0D + 1.0L (0.925)

Combination 2 results: 1.0D + 1.0L

Forces on foundation per linear foot

Force in z-axis — $F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1}$ = **3.5** kips

Moments on foundation per linear foot

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1)$ = **2.6** kip_ft

Uplift verification

Vertical force — F_{dz} = **3.469** kips

PASS - Foundation is not subject to uplift

Stability against sliding

Resistance due to base friction — $F_{RFriction} = \max(F_{dz}, 0\ kN) * \tan(\delta_{bb})$ = **1.214** kips

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0.000** in

Strip base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dy} / L_y) / (L_y * 1\ ft)$ = **2.312** ksf

$q_2 = F_{dz} * (1 + 6 * e_{dy} / L_y) / (L_y * 1\ ft)$ = **2.312** ksf

Minimum base pressure — $q_{min} = \min(q_1,q_2)$ = **2.312** ksf

Maximum base pressure — $q_{max} = \max(q_1,q_2)$ = **2.312** ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross}$ = **2.5** ksf

q_{max} / q_{allow} = **0.925**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete	f'_c = **4000** psi
Yield strength of reinforcement	f_y = **60000** psi
Compression-controlled strain limit (21.2.2)	ε_{ty} = **0.00200**
Cover to reinforcement	c_{nom} = **3** in
Concrete type	Normal weight
Concrete modification factor	λ = **1.00**
Wall type	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.009)

1.2D + 1.6L + 0.5Lr (0.018)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on foundation per linear foot

Ultimate force in z-axis — $F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1}$ = **4.8** kips

Moments on foundation per linear foot

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1)$ = **3.6** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0.000** in

Strip base pressures

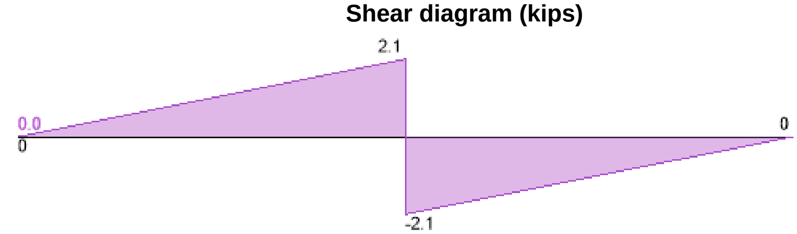
$q_{u1} = F_{uz} * (1 - 6 * e_{uy} / L_y) / (L_y * 1\text{ ft})$ = **3.175** ksf

$q_{u2} = F_{uz} * (1 + 6 * e_{uy} / L_y) / (L_y * 1\text{ ft})$ = **3.175** ksf

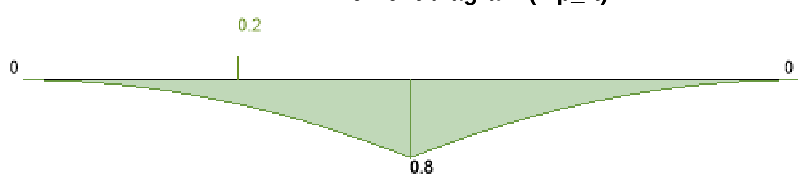
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2})$ = **3.175** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2})$ = **3.175** ksf

Shear diagram (kips)



Moment diagram (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment — $M_{u.y.max}$ = **0.243** kip_ft

Tension reinforcement provided — No.5 bars at 8.0 in c/c bottom

Area of tension reinforcement provided — $A_{sy.bot.prov}$ = **0.465** in^2

Minimum area of reinforcement (7.6.1.1) — $A_{s.min} = 0.0018 * L_x * h$ = **0.216** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (7.7.2.3) — $s_{max} = \min(3 * h, 18\text{ in})$ = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom} - \phi_{y.bot} / 2$ = **6.688** in

Depth of compression block — $a = A_{sy.bot.prov} * f_y / (0.85 * f'_c * L_x)$ = **0.684** in

Neutral axis factor — β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.804** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 * d / c - 0.003$ = **0.02194**

Minimum tensile strain(7.3.3.1) $\varepsilon_{min} = 0.004$ = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sy.bot.prov} * f_y * (d - a / 2)$ = **14.753** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 * (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**

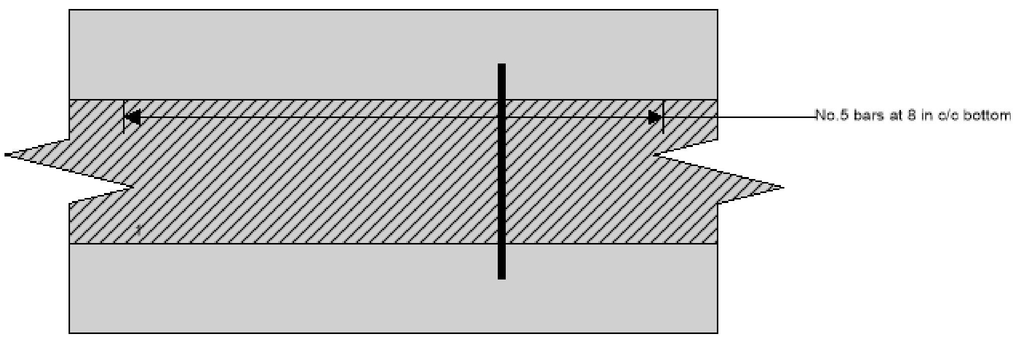
Design moment capacity $\phi M_n = \phi_f * M_n$ = **13.278** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.018**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.

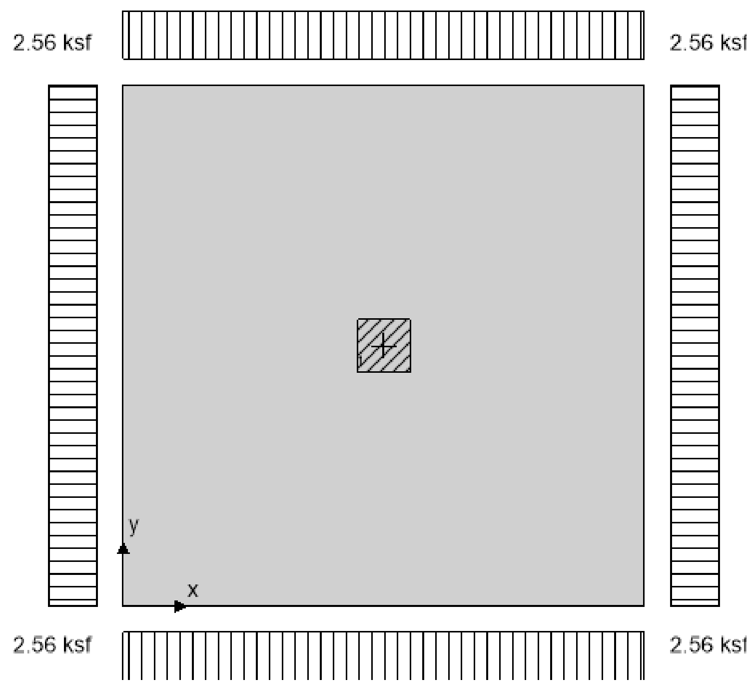


Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.10

FOOTING ANALYSIS

Length of foundation	L_x = **5** ft
Width of foundation	L_y = **5** ft
Foundation area	$A = L_x \times L_y$ = **25** ft^2
Depth of foundation	h = **14** in
Depth of soil over foundation	h_{soil} = **18** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **6.00** in
Width of column	l_{y1} = **6.00** in
position in x-axis	x_1 = **30.00** in
position in y-axis	y_1 = **30.00** in

Soil properties

Net allowable bearing pressure	q_{allow_Net} = **2.5** ksf using a soil factor of safety, FS_{soil}, of 3
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Live surcharge load	F_{Lsur} = **100** psf

Self weight $F_{swt} = h * \gamma_{conc} =$ **175** psf

Soil weight $F_{soil} = h_{soil} * \gamma_{soil} =$ **180** psf

Column no.1 loads

Dead load in z $F_{Dz1} =$ **12.6** kips

Live load in z $F_{Lz1} =$ **26.5** kips

Snow load in z $F_{Sz1} =$ **27.7** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.330)

1.0D + 1.0L (0.775)

1.0D + 0.75L + 0.75S + 0.45W (0.982)

Combination 12 results: 1.0D + 0.75L + 0.75S + 0.45W

Forces on foundation

Force in z-axis $F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_L * A * F_{Lsur} + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_S * F_{Sz1} =$ **64.0** kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_L * A * F_{Lsur} * L_x / 2 + \gamma_D * (F_{Dz1} * x_1) + \gamma_L * (F_{Lz1} * x_1) + \gamma_S * (F_{Sz1} * x_1) =$ **160.0** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_L * A * F_{Lsur} * L_y / 2 + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_S * (F_{Sz1} * y_1) =$ **160.0** kip_ft

Uplift verification

Vertical force $F_{dz} =$ **64** kips

PASS - Foundation is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **0** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y) =$ **2.56** ksf

$q_2 = F_{dz} * (1 - 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y) =$ **2.56** ksf

$q_3 = F_{dz} * (1 + 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y) =$ **2.56** ksf

$q_4 = F_{dz} * (1 + 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y) =$ **2.56** ksf

Minimum base pressure $q_{min} = \min(q_1, q_2, q_3, q_4) =$ **2.56** ksf

Maximum base pressure $q_{max} = \max(q_1, q_2, q_3, q_4) =$ **2.56** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Net} + ((h + h_{soil}) * \gamma_{soil}) / FS_{soil} =$ **2.607** ksf

$q_{max} / q_{allow} =$ **0.982**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete f'_c = **4000** psi

Yield strength of reinforcement f_y = **60000** psi

Compression-controlled strain limit (21.2.2) ε_{ty} = **0.00200**

Cover to reinforcement c_{nom} = **3** in

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.129)

1.2D + 1.6L + 0.5Lr (0.421)

1.2D + 1.6L + 0.5S (0.522)

Combination 3 results: 1.2D + 1.6L + 0.5S

Forces on foundation

Ultimate force in z-axis $F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_L * A * F_{Lsur} + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_S * F_{Sz1}$ = **86.0** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_L * A * F_{Lsur} * L_x / 2 + \gamma_D * (F_{Dz1} * x_1) + \gamma_L * (F_{Lz1} * x_1) + \gamma_S * (F_{Sz1} * x_1)$ = **215.0** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_L * A * F_{Lsur} * L_y / 2 + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_S * (F_{Sz1} * y_1)$ = **215.0** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} * (1 - 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y)$ = **3.441** ksf

$q_{u2} = F_{uz} * (1 - 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y)$ = **3.441** ksf

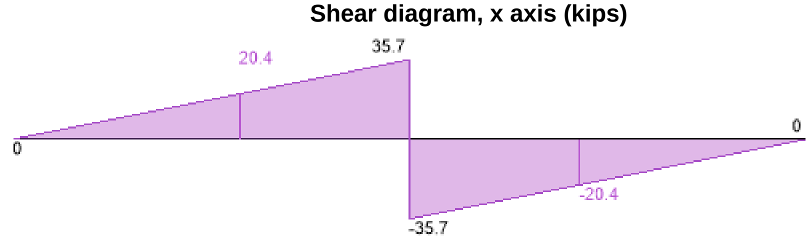
$q_{u3} = F_{uz} * (1 + 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y)$ = **3.441** ksf

$q_{u4} = F_{uz} * (1 + 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y)$ = **3.441** ksf

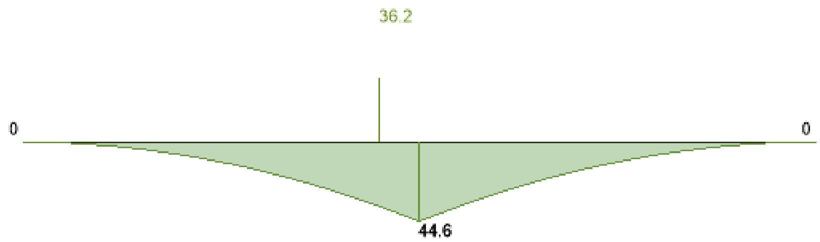
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **3.441** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **3.441** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **36.152** kip_ft

Tension reinforcement provided 5 No.6 bottom bars (13.3 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov}$ = **2.2** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min}$ = 0.0018 * L_y * h = **1.512** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) s_{max} = min(2 * h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement d = h - c_{nom} - $\phi_{x.bot}$ / 2 = **10.625** in

Depth of compression block a = $A_{sx.bot.prov}$ * f_y / (0.85 * f'_c * L_y) = **0.647** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis c = a / β_1 = **0.761** in

Strain in tensile reinforcement ε_t = 0.003 * d / c - 0.003 = **0.03887**

Minimum tensile strain(8.3.3.1) ε_{min} = 0.004 = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity M_n = $A_{sx.bot.prov}$ * f_y * (d - a / 2) = **113.316** kip_ft

Flexural strength reduction factor ϕ_f = min(max(0.65 + 0.25 * (ε_t - ε_{ty}) / (0.005 - ε_{ty}), 0.65), 0.9) = **0.900**

Design moment capacity ϕM_n = ϕ_f * M_n = **101.985** kip_ft

$M_{u.x.max}$ / ϕM_n = **0.354**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x}$ = **20.38** kips

Depth to reinforcement d_v = h - c_{nom} - $\phi_{x.bot}$ / 2 = **10.625** in

Shear strength reduction factor ϕ_v = **0.75**

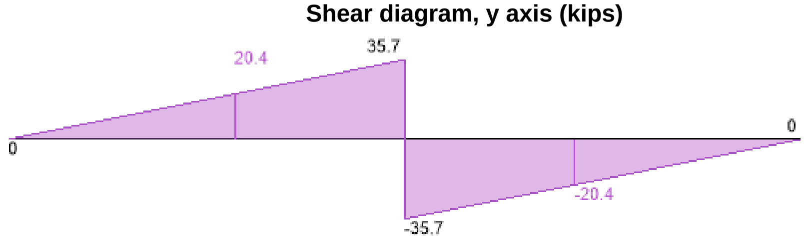
Nominal shear capacity (Eq. 22.5.5.1) V_n = 2 * λ * $\sqrt{(f'_c * 1 psi)}$ * L_y * d_v = **80.638** kips

Design shear capacity ϕV_n = ϕ_v * V_n = **60.479** kips

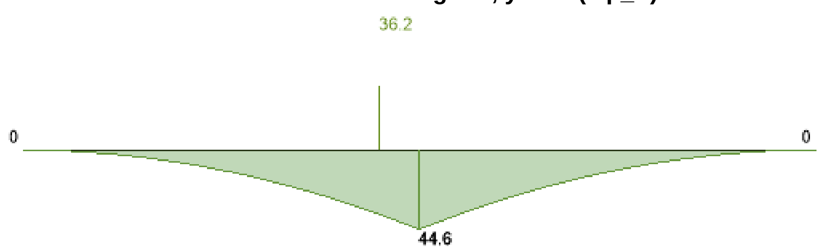
$V_{u.x}$ / ϕV_n = **0.337**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **36.152** kip_ft
Tension reinforcement provided	5 No.6 bottom bars (13.3 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **2.2** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min}$ = 0.0018 * L_x * h = **1.512** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	d = h - c_{nom} - $\phi_{x.bot}$ - $\phi_{y.bot}$ / 2 = **9.875** in
Depth of compression block	a = $A_{sy.bot.prov}$ * f_y / (0.85 * f'$_c$ * L_x) = **0.647** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	c = a / β_1 = **0.761** in
Strain in tensile reinforcement	ε_t = 0.003 * d / c - 0.003 = **0.03592**
Minimum tensile strain(8.3.3.1)	ε_{min} = 0.004 = **0.00400**
	PASS - Tensile strain exceeds minimum required
Nominal moment capacity	M_n = $A_{sy.bot.prov}$ * f_y * (d - a / 2) = **105.066** kip_ft
Flexural strength reduction factor	ϕ_f = min(max(0.65 + 0.25 * (ε_t - ε_{ty}) / (0.005 - ε_{ty}), 0.65), 0.9) = **0.900**
Design moment capacity	ϕM_n = ϕ_f * M_n = **94.56** kip_ft
	$M_{u.y.max}$ / ϕM_n = **0.382**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force	$V_{u.y}$ = **20.38** kips
Depth to reinforcement	d_v = h - c_{nom} - $\phi_{x.bot}$ - $\phi_{y.bot}$ / 2 = **9.875** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	V_n = 2 * λ * $\sqrt{}$(f'$_c$ * 1 psi) * L_x * d_v = **74.946** kips
Design shear capacity	ϕV_n = ϕ_v * V_n = **56.209** kips
	$V_{u.y}$ / ϕV_n = **0.363**
	PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement	d_{v2} = **10.25** in
Shear perimeter length (22.6.4)	l_{xp} = **16.250** in
Shear perimeter width (22.6.4)	l_{yp} = **16.250** in
Shear perimeter (22.6.4)	b_o = 2 * (l_{x1} + d_{v2}) + 2 * (l_{y1} + d_{v2}) = **65.000** in
Shear area	A_p = $l_{x,perim}$ * $l_{y,perim}$ = **264.062** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} * l_{y1} =$ **228.062** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg} =$ **3.441** ksf

Ultimate shear load $F_{up} = \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_S * F_{Sz1} + \gamma_D * A_p * F_{swt} + \gamma_D * A_{sur} * F_{soil} + \gamma_L * A_{sur} * F_{Lsur} - q_{up.avg} * A_p =$ **66.041** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o * d_{v2}), 0\text{ psi}) =$ **99.123** psi

Column geometry factor (Table 22.6.5.2) $\beta = l_{y1} / l_{x1} =$ **1.00**

Column location factor (22.6.5.3) $\alpha_s =$ **40**

Concrete shear strength (22.6.5.2) $v_{cpa} = (2 + 4 / \beta) * \lambda * \sqrt{(f'_c * 1\text{ psi})} =$ **379.473** psi

$v_{cpb} = (\alpha_s * d_{v2} / b_o + 2) * \lambda * \sqrt{(f'_c * 1\text{ psi})} =$ **525.425** psi

$v_{cpc} = 4 * \lambda * \sqrt{(f'_c * 1\text{ psi})} =$ **252.982** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **252.982** psi

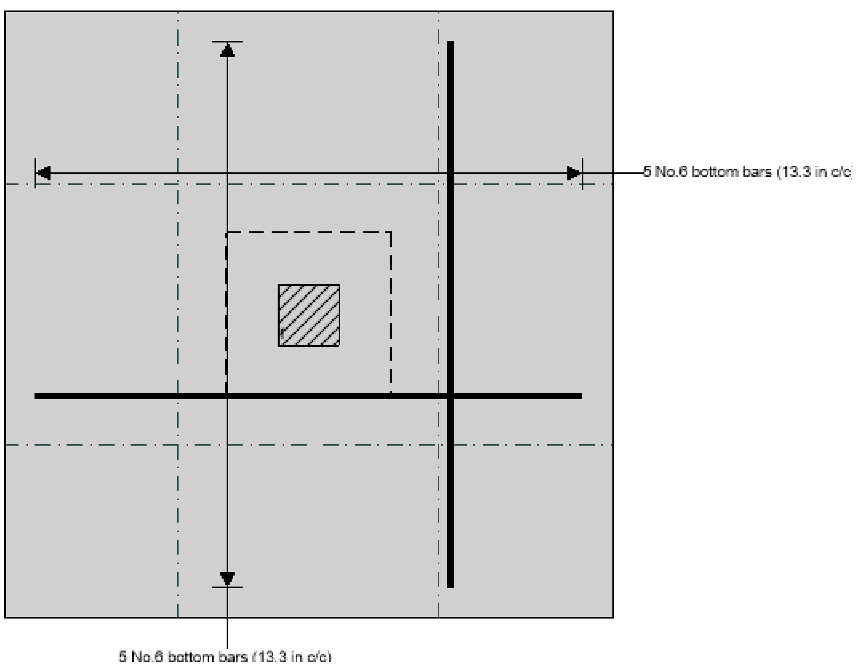
Shear strength reduction factor $\phi_v =$ **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp} =$ **252.982** psi

Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v * v_n =$ **189.737** psi

$v_{ug} / \phi v_n =$ **0.522**

PASS - Design shear stress capacity exceeds ultimate shear stress load

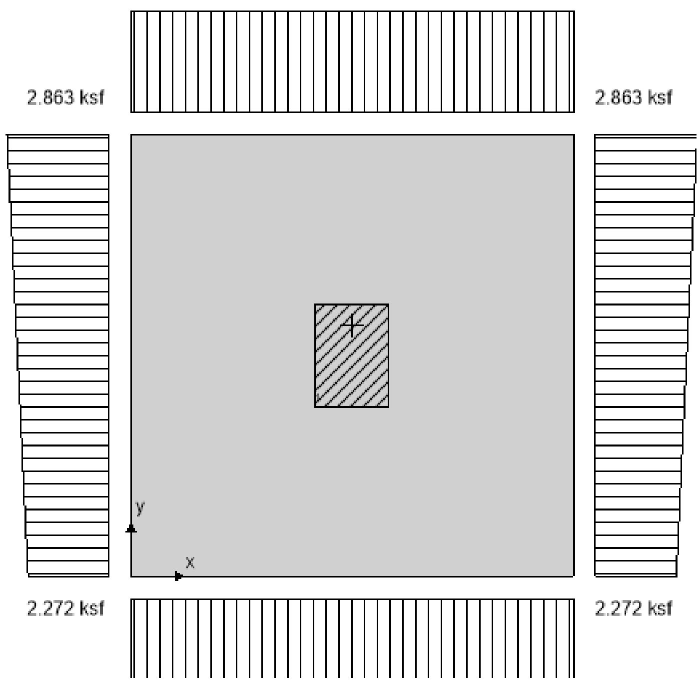


Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.10

FOOTING ANALYSIS

Length of foundation	L_x = **5** ft
Width of foundation	L_y = **5** ft
Foundation area	$A = L_x \times L_y$ = **25** ft^2
Depth of foundation	h = **14** in
Depth of soil over foundation	h_{soil} = **18** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **10.00** in
Width of column	l_{y1} = **14.00** in
position in x-axis	x_1 = **30.00** in
position in y-axis	y_1 = **30.00** in

Soil properties

Net allowable bearing pressure	q_{allow_Net} = **3.325** ksf using a soil factor of safety, FS_{soil}, of 3
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Live surcharge load	F_{Lsur} = **50** psf

Self weight $F_{swt} = h * \gamma_{conc} =$ **175** psf

Soil weight $F_{soil} = h_{soil} * \gamma_{soil} =$ **180** psf

Column no.1 loads

Dead load in z $F_{Dz1} =$ **30.0** kips

Live load in z $F_{Lz1} =$ **20.0** kips

Snow load in z $F_{Sz1} =$ **7.6** kips

Dead load in y $F_{Dy1} =$ **0.2** kips

Live load in y $F_{Ly1} =$ **1.2** kips

Wind load in y $F_{Wy1} =$ **4.3** kips

Seismic load in y $F_{Ey1} =$ **8.0** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.456)

1.0D + 1.0L (0.724)

1.0D + 1.0Lr (0.456)

1.0D + 1.0S (0.545)

1.0D + 1.0R (0.456)

1.0D + 0.75L + 0.75Lr (0.657)

1.0D + 0.75L + 0.75S (0.723)

1.0D + 0.75L + 0.75R (0.657)

1.0D + 0.6W (0.498)

$(1.0 + 0.14 * S_{DS})D + 0.7E$ (0.607)

1.0D + 0.75L + 0.75Lr + 0.45W (0.688)

1.0D + 0.75L + 0.75S + 0.45W (0.754)

1.0D + 0.75L + 0.75R + 0.45W (0.688)

$(1.0 + 0.10 * S_{DS})D + 0.75L + 0.75S + 0.525E$ (0.834)

0.6D + 0.6W (0.315)

$(0.6 - 0.14 * S_{DS})D + 0.7E$ (0.538)

Combination 14 results: $(1.0 + 0.10 * S_{DS})D + 0.75L + 0.75S + 0.525E$

Forces on foundation

Force in y-axis $F_{dy} = \gamma_D * F_{Dy1} + \gamma_L * F_{Ly1} + \gamma_E * F_{Ey1} =$ **5.3** kips

Force in z-axis $F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_L * A * F_{Lsur} + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_S * F_{Sz1} =$ **64.2** kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_L * A * F_{Lsur} * L_x / 2 + \gamma_D * (F_{Dz1} * x_1) + \gamma_L * (F_{Lz1} * x_1) + \gamma_S * (F_{Sz1} * x_1) =$ **160.5** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_L * A * F_{Lsur} * L_y / 2 + \gamma_D * (F_{Dz1} * y_1+F_{Dy1} * h) + \gamma_L * (F_{Lz1} * y_1+F_{Ly1} * h) + \gamma_S * (F_{Sz1} * y_1) + \gamma_E * (F_{Ey1} * h) =$ **166.6** kip_ft

Uplift verification

Vertical force $F_{dz} =$ **64.182** kips

PASS - Foundation is not subject to uplift

Stability against overturning in y direction, moment about y is L_y

Overturning moment $M_{OTyL} = \gamma_D * (F_{Dy1} * h) + \gamma_L * (F_{Ly1} * h) + \gamma_E * (F_{Ey1} * h) =$ **6.16** kip_ft

Resisting moment $M_{RyL} = -1 * (\gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_L * A * F_{Lsur} * L_y / 2) + \gamma_D * (F_{Dz1} * (y_1 - L_y)) + \gamma_L * (F_{Lz1} * (y_1 - L_y)) + \gamma_S * (F_{Sz1} * (y_1 - L_y)) =$ **-160.46** kip_ft

Factor of safety $abs(M_{RyL} / M_{OTyL}) =$ **26.057**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding

Resistance due to base friction $F_{RFriction} = max(F_{dz}, 0 \text{ kN}) * tan(\delta_{bb}) =$ **37.056** kips

Stability against sliding in y direction

Total sliding resistance $F_{Ry} = F_{RFriction} =$ **37.056** kips

Factor of safety $abs(F_{Ry} / F_{dy}) =$ **7.02**

PASS - Sliding factor of safety exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **0** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **1.151** in

Pad base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y) =$ **2.272** ksf

$q_2 = F_{dz} * (1 - 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y) =$ **2.863** ksf

$q_3 = F_{dz} * (1 + 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y) =$ **2.272** ksf

$q_4 = F_{dz} * (1 + 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y) =$ **2.863** ksf

Minimum base pressure $q_{min} = min(q_1,q_2,q_3,q_4) =$ **2.272** ksf

Maximum base pressure $q_{max} = max(q_1,q_2,q_3,q_4) =$ **2.863** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Net} + ((h + h_{soil}) * \gamma_{soil}) / FS_{soil} =$ **3.432** ksf

$q_{max} / q_{allow} =$ **0.834**

PASS - Allowable bearing capacity exceeds design base pressure

<u>FOOTING DESIGN (ACI318)</u>

In accordance with ACI318-14

Material details

Compressive strength of concrete $f'_c =$ **4000** psi

Yield strength of reinforcement $f_y =$ **60000** psi

Compression-controlled strain limit (21.2.2) $\varepsilon_{ty} =$ **0.00200**

Cover to reinforcement $c_{nom} =$ **3** in

Concrete type Normal weight

Concrete modification factor $\lambda =$ **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.208)

1.2D + 1.6L + 0.5Lr (0.336)

Tekla Tedds Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsky Residence				Job Ref. 8119	
	Section C1 Pad Footing (F3)				Sheet no./rev. 4	
	Calc. by BJW	Date 2/25/2021	Chk'd by	Date	App'd by	Date

1.2D + 1.6L + 0.5S (0.355)
1.2D + 1.6L + 0.5R (0.336)
1.2D + 1.0L + 1.6Lr (0.277)
1.2D + 1.0L + 1.6S (0.337)
1.2D + 1.0L + 1.6R (0.277)
1.2D + 1.6Lr + 0.5W (0.176)
1.2D + 1.6S + 0.5W (0.237)
1.2D + 1.6R + 0.5W (0.176)
1.2D + 1.0L + 0.5Lr + 1.0W (0.273)
1.2D + 1.0L + 0.5S + 1.0W (0.292)
1.2D + 1.0L + 0.5R + 1.0W (0.273)
(1.2 + 0.2 * S_{DS})D + 1.0L + 0.2S + 1.0E (0.305)
0.9D + 1.0W (0.130)
(0.9 - 0.2 * S_{DS})D + 1.0E (0.109)

Combination 3 results: 1.2D + 1.6L + 0.5S

Forces on foundation

Ultimate force in y-axis $F_{uy} = \gamma_D * F_{Dy1} + \gamma_L * F_{Ly1} =$ **2.2** kips

Ultimate force in z-axis $F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_L * A * F_{Lsur} + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_S * F_{Sz1} =$ **84.4** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_L * A * F_{Lsur} * L_x / 2 + \gamma_D * (F_{Dz1} * x_1) + \gamma_L * (F_{Lz1} * x_1) + \gamma_S * (F_{Sz1} * x_1) =$ **211.1** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_L * A * F_{Lsur} * L_y / 2 + \gamma_D * (F_{Dz1} * y_1 + F_{Dy1} * h) + \gamma_L * (F_{Lz1} * y_1 + F_{Ly1} * h) + \gamma_S * (F_{Sz1} * y_1) =$ **213.6** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2 =$ **0** in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2 =$ **0.357** in

Pad base pressures

$q_{u1} = F_{uz} * (1 - 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y) =$ **3.257** ksf

$q_{u2} = F_{uz} * (1 - 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y) =$ **3.499** ksf

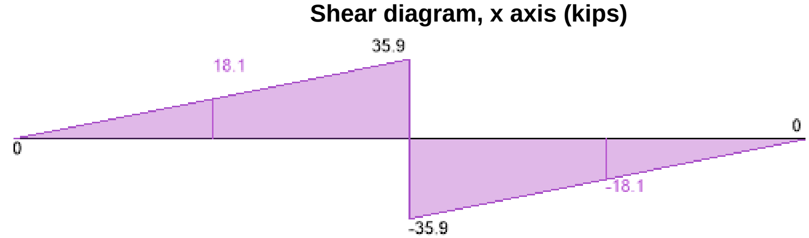
$q_{u3} = F_{uz} * (1 + 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y) =$ **3.257** ksf

$q_{u4} = F_{uz} * (1 + 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y) =$ **3.499** ksf

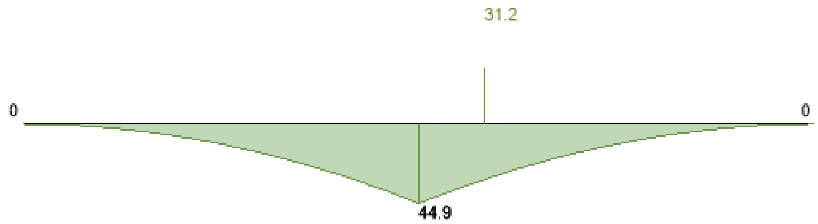
Minimum ultimate base pressure $q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) =$ **3.257** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) =$ **3.499** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **31.163** kip_ft

Tension reinforcement provided 5 No.6 bottom bars (13.3 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov}$ = **2.2** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 * L_y * h$ = **1.512** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) s_{max} = min(2 * h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.bot} / 2$ = **10.625** in

Depth of compression block $a = A_{sx.bot.prov} * f_y / (0.85 * f'_c * L_y)$ = **0.647** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.761** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 * d / c - 0.003$ = **0.03887**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = 0.004$ = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sx.bot.prov} * f_y * (d - a / 2)$ = **113.316** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 * (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f * M_n$ = **101.985** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.306**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x}$ = **18.1** kips

Depth to reinforcement $d_v = h - c_{nom} - \phi_{y.bot} - \phi_{x.bot} / 2$ = **9.875** in

Shear strength reduction factor ϕ_v = **0.75**

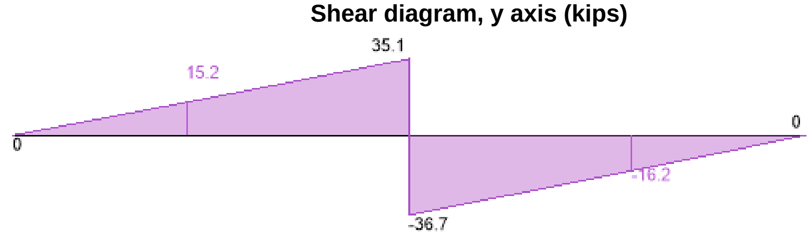
Nominal shear capacity (Eq. 22.5.5.1) $V_n = 2 * \lambda * \sqrt{(f'_c * 1 \text{ psi})} * L_y * d_v$ = **74.946** kips

Design shear capacity $\phi V_n = \phi_v * V_n$ = **56.209** kips

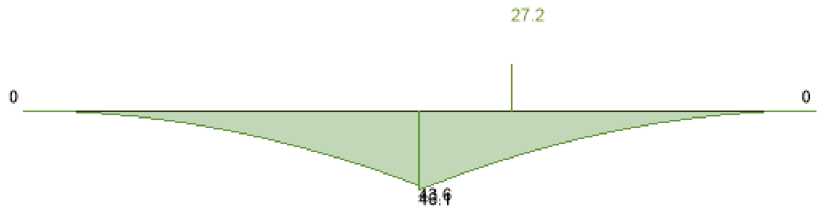
$V_{u.x} / \phi V_n$ = **0.322**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **27.2** kip_ft

Tension reinforcement provided 5 No.6 bottom bars (13.3 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **2.2** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min}$ = 0.0018 * L_x * h = **1.512** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) s_{max} = min(2 * h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement d = h - c_{nom} - $\phi_{x.bot}$ - $\phi_{y.bot}$ / 2 = **9.875** in

Depth of compression block a = $A_{sy.bot.prov}$ * f_y / (0.85 * f'_c * L_x) = **0.647** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis c = a / β_1 = **0.761** in

Strain in tensile reinforcement ε_t = 0.003 * d / c - 0.003 = **0.03592**

Minimum tensile strain(8.3.3.1) ε_{min} = 0.004 = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity M_n = $A_{sy.bot.prov}$ * f_y * (d - a / 2) = **105.066** kip_ft

Flexural strength reduction factor ϕ_f = min(max(0.65 + 0.25 * (ε_t - ε_{ty}) / (0.005 - ε_{ty}), 0.65), 0.9) = **0.900**

Design moment capacity ϕM_n = ϕ_f * M_n = **94.56** kip_ft

$M_{u.y.max}$ / ϕM_n = **0.288**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **16.221** kips

Depth to reinforcement d_v = h - c_{nom} - $\phi_{y.bot}$ / 2 = **10.625** in

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear capacity (Eq. 22.5.5.1) V_n = 2 * λ * $\sqrt{(f'_c * 1\ psi)}$ * L_x * d_v = **80.638** kips

Design shear capacity ϕV_n = ϕ_v * V_n = **60.479** kips

$V_{u.y}$ / ϕV_n = **0.268**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement d_{v2} = **10.25** in

Shear perimeter length (22.6.4) l_{xp} = **20.250** in

Shear perimeter width (22.6.4) l_{yp} = **24.250** in

Shear perimeter (22.6.4) b_o = 2 * (l_{x1} + d_{v2}) + 2 * (l_{y1} + d_{v2}) = **89.000** in

Shear area A_p = $l_{x,perim}$ * $l_{y,perim}$ = **491.062** in^2

Surcharge loaded area A_{sur} = A_p - l_{x1} * l_{y1} = **351.062** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg}$ = **3.475** ksf

Ultimate shear load $F_{up} = \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_S * F_{Sz1} + \gamma_D * A_p * F_{swt} + \gamma_D * A_{sur} * F_{soil} + \gamma_L * A_{sur} * F_{Lsur} - q_{up.avg} * A_p$ = **61.386** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o * d_{v2}),0\ psi)$ = **67.291** psi

Column geometry factor (Table 22.6.5.2) $\beta = l_{y1} / l_{x1}$ = **1.40**

Column location factor (22.6.5.3) α_s =**40**

Concrete shear strength (22.6.5.2) $v_{cpa} = (2 + 4 / \beta) * \lambda * \sqrt{(f'_c * 1\ psi)}$ = **307.193** psi

$v_{cpb} = (\alpha_s * d_{v2} / b_o + 2) * \lambda * \sqrt{(f'_c * 1\ psi)}$ = **417.847** psi

$v_{cpc} = 4 * \lambda * \sqrt{(f'_c * 1\ psi)}$ = **252.982** psi

$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc})$ = **252.982** psi

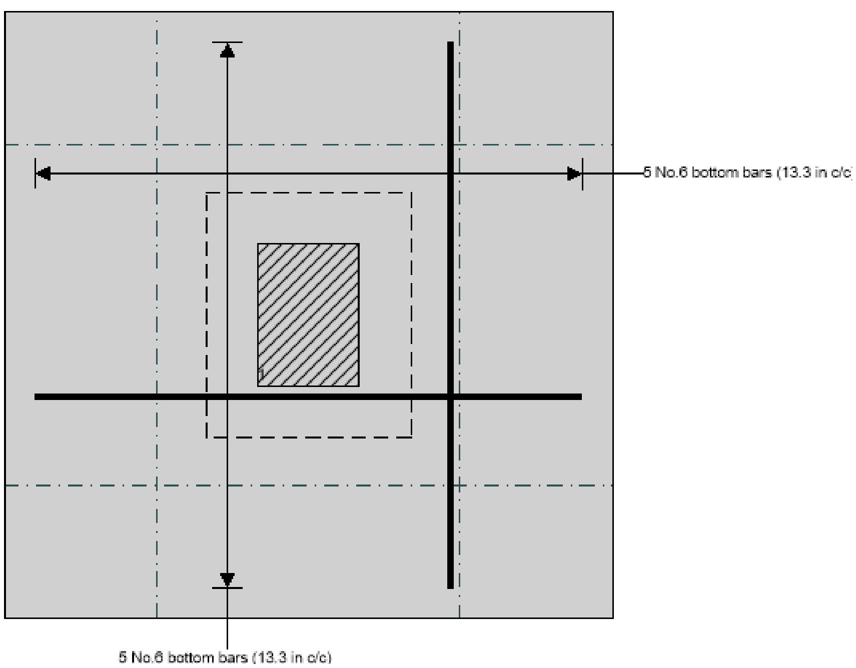
Shear strength reduction factor ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp}$ = **252.982** psi

Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v * v_n$ = **189.737** psi

$v_{ug} / \phi v_n$ = **0.355**

PASS - Design shear stress capacity exceeds ultimate shear stress load



Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Central Wall Load Takedown	**DATE:**	2021-03-02
DESIGN BY:	BJW		

NOTES: Central shear wall supporting high roof and living room - MAIN LEVEL

GEOMETRY:

Tributary area	A_T =	200.000	ft^2
Wall length	L =	5.83	ft

SURFACE LOADS:

Dead load	DL =	0	psf
Superimposed dead load	SDL =	30	psf
Live load	LL =	40	psf
Snow load	SL =	0	psf

LINE LOADS:

Dead load	DL =	**0**	plf	**0**	klf
Superimposed dead load	SDL =	**1028.571**	plf	**1.029**	klf
Live load	LL =	**1371.429**	plf	**1.371**	klf
Snow load	SL =	**0**	plf	**0**	klf

Fast + Epp

INPUT
RESULT

PROJECT:	Yaroslavsky Residence	**PROJECT NUMBER:**	8119
SUBJECT:	Central Wall Load Takedown	**DATE:**	2021-03-02
DESIGN BY:	BJW		

NOTES: Central shear wall supporting high roof and living room - UPPER LEVEL

GEOMETRY:

Wall length	L =	5.83	ft

POINT LOADS (FROM TEDDS OUTPUT):

Dead load	DL =	0	lbs
Superimposed dead load	SDL =	11800	lbs
Live load	LL =	8300	lbs
Snow load	SL =	4700	lbs
Seismic load	EQ =	8000	lbs

LINE LOADS:

Dead load	DL =	**0**	plf	**0**	klf
Superimposed dead load	SDL =	**2022.857**	plf	**2.023**	klf
Live load	LL =	**1422.857**	plf	**1.423**	klf
Snow load	SL =	**805.714**	plf	**0.806**	klf
Seismic load	EQ =	**1371.429**	plf	**1.371**	klf

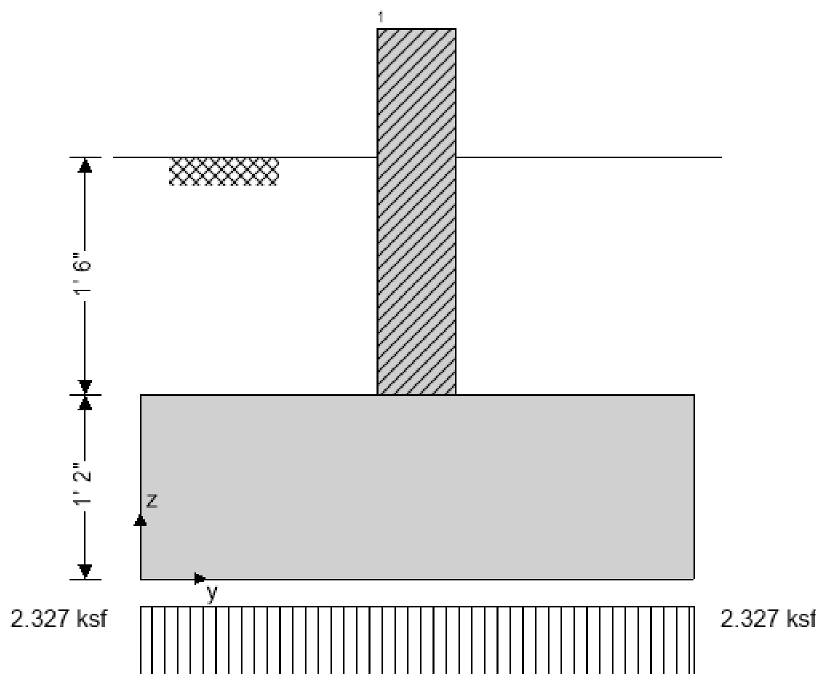
	Line Load Total	
SDL	**3.05**	klf
LL	**2.79**	klf
SL	**0.81**	klf
EQ	**1.37**	klf

Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.10

FOOTING ANALYSIS

Length of foundation	L_x = **1** ft
Width of foundation	L_y = **3.5** ft
Foundation area	$A = L_x \times L_y$ = **3.5** ft^2
Depth of foundation	h = **14** in
Depth of soil over foundation	h_{soil} = **18** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Wall no.1 details

Width of wall	l_{y1} = **6** in
position in y-axis	y_1 = **21** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2.5** ksf
Density of soil	γ_{soil} = **125.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **19.3** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.350**
Self weight	$F_{swt} = h * \gamma_{conc}$ = **175** psf
Soil weight	$F_{soil} = h_{soil} * \gamma_{soil}$ = **187.5** psf

Wall no.1 loads per linear foot

Dead load in z F_{Dz1} = **3.1** kips

Live load in z F_{Lz1} = **2.8** kips

Snow load in z F_{Sz1} = **0.8** kips

Seismic load in z F_{Ez1} = **1.4** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.494)

1.0D + 1.0L (0.812)

1.0D + 1.0Lr (0.494)

1.0D + 1.0S (0.586)

1.0D + 1.0R (0.494)

1.0D + 0.75L + 0.75Lr (0.733)

1.0D + 0.75L + 0.75S (0.802)

1.0D + 0.75L + 0.75R (0.733)

1.0D + 0.6W (0.494)

(1.0 + 0.14 * S_{DS})D + 0.7E (0.668)

1.0D + 0.75L + 0.75Lr + 0.45W (0.733)

1.0D + 0.75L + 0.75S + 0.45W (0.802)

1.0D + 0.75L + 0.75R + 0.45W (0.733)

(1.0 + 0.10 * S_{DS})D + 0.75L + 0.75S + 0.525E (0.931)

0.6D + 0.6W (0.296)

(0.6 - 0.14 * S_{DS})D + 0.7E (0.341)

Combination 14 results: (1.0 + 0.10 * S_{DS})D + 0.75L + 0.75S + 0.525E

Forces on foundation per linear foot

Force in z-axis $F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_S * F_{Sz1} + \gamma_E * F_{Ez1}$ = **8.1** kips

Moments on foundation per linear foot

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_S * (F_{Sz1} * y_1) + \gamma_E * (F_{Ez1} * y_1)$ = **14.3** kip_ft

Uplift verification

Vertical force F_{dz} = **8.146** kips

PASS - Foundation is not subject to uplift

Stability against sliding

Resistance due to base friction $F_{RFriction} = \max(F_{dz}, 0\ kN) * \tan(\delta_{bb})$ = **2.851** kips

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0.000** in

Strip base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dy} / L_y) / (L_y * 1\ ft)$ = **2.327** ksf

$q_2 = F_{dz} * (1 + 6 * e_{dy} / L_y) / (L_y * 1\ ft)$ = **2.327** ksf

Minimum base pressure $q_{min} = \min(q_1, q_2)$ = **2.327** ksf

Maximum base pressure — $q_{max} = max(q_1,q_2) = \mathbf{2.327}$ ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross} = \mathbf{2.5}$ ksf

$q_{max} / q_{allow} = \mathbf{0.931}$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete — $f'_c = \mathbf{4000}$ psi

Yield strength of reinforcement — $f_y = \mathbf{60000}$ psi

Compression-controlled strain limit (21.2.2) — $\varepsilon_{ty} = \mathbf{0.00200}$

Cover to reinforcement — $c_{nom} = \mathbf{3}$ in

Concrete type — Normal weight

Concrete modification factor — $\lambda = \mathbf{1.00}$

Wall type — Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.094)

1.2D + 1.6L + 0.5Lr (0.179)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on foundation per linear foot

Ultimate force in z-axis — $F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} = \mathbf{9.6}$ kips

Moments on foundation per linear foot

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) = \mathbf{16.9}$ kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0.000}$ in

Strip base pressures

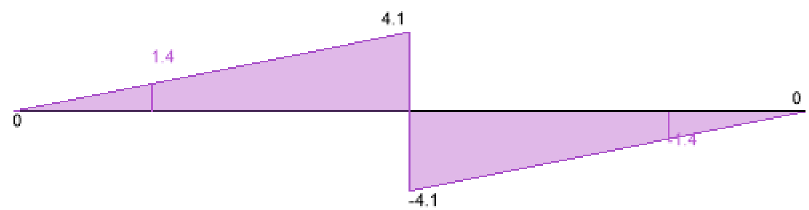
$q_{u1} = F_{uz} * (1 - 6 * e_{uy} / L_y) / (L_y * 1 \text{ ft}) = \mathbf{2.756}$ ksf

$q_{u2} = F_{uz} * (1 + 6 * e_{uy} / L_y) / (L_y * 1 \text{ ft}) = \mathbf{2.756}$ ksf

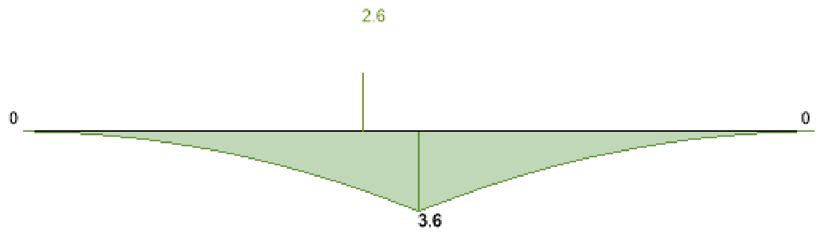
Minimum ultimate base pressure — $q_{umin} = min(q_{u1},q_{u2}) = \mathbf{2.756}$ ksf

Maximum ultimate base pressure — $q_{umax} = max(q_{u1},q_{u2}) = \mathbf{2.756}$ ksf

Shear diagram (kips)



Moment diagram (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **2.614** kip_ft

Tension reinforcement provided No.5 bars at 12.0 in c/c bottom

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **0.31** in^2

Minimum area of reinforcement (7.6.1.1) $A_{s.min}$ = 0.0018 * L_x * h = **0.302** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (7.7.2.3) s_{max} = min(3 * h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement d = h - c_{nom} - $\phi_{y.bot}$ / 2 = **10.688** in

Depth of compression block a = $A_{sy.bot.prov}$ * f_y / (0.85 * f'_c * L_x) = **0.456** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis c = a / β_1 = **0.536** in

Strain in tensile reinforcement ε_t = 0.003 * d / c - 0.003 = **0.05678**

Minimum tensile strain(7.3.3.1) ε_{min} = 0.004 = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity M_n = $A_{sy.bot.prov}$ * f_y * (d - a / 2) = **16.212** kip_ft

Flexural strength reduction factor ϕ_f = min(max(0.65 + 0.25 * (ε_t - ε_{ty}) / (0.005 - ε_{ty}), 0.65), 0.9) = **0.900**

Design moment capacity ϕM_n = ϕ_f * M_n = **14.591** kip_ft

$M_{u.y.max}$ / ϕM_n = **0.179**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **1.416** kips

Depth to reinforcement d_v = h - c_{nom} - $\phi_{y.bot}$ / 2 = **10.688** in

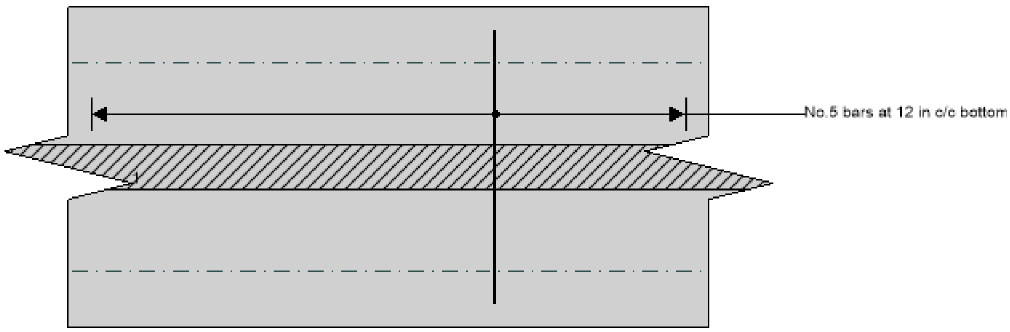
Shear strength reduction factor ϕ_v = **0.75**

Nominal shear capacity (Eq. 22.5.5.1) V_n = 2 * λ * $\sqrt{(f'_c * 1 \text{ psi})}$ * L_x * d_v = **16.222** kips

Design shear capacity ϕV_n = ϕ_v * V_n = **12.167** kips

$V_{u.y}$ / ϕV_n = **0.116**

PASS - Design shear capacity exceeds ultimate shear load



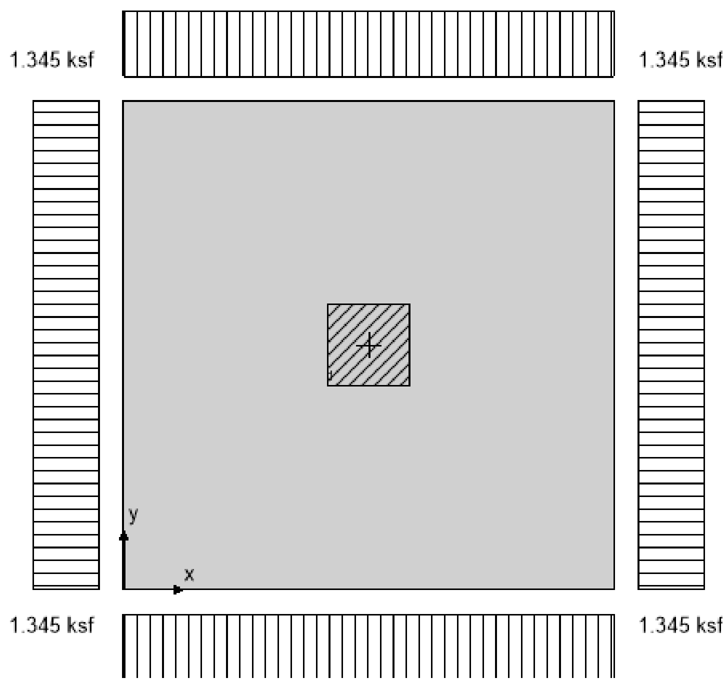
No.5 bars at 12 in c/c bottom

Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.10

FOOTING ANALYSIS

Length of foundation	L_x = **3** ft
Width of foundation	L_y = **3** ft
Foundation area	$A = L_x \times L_y$ = **9** ft^2
Depth of foundation	h = **14** in
Depth of soil over foundation	h_{soil} = **18** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **6.00** in
Width of column	l_{y1} = **6.00** in
position in x-axis	x_1 = **18.00** in
position in y-axis	y_1 = **18.00** in

Soil properties

Net allowable bearing pressure	q_{allow_Net} = **2.5** ksf using a soil factor of safety, FS_{soil}, of 3
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Live surcharge load	F_{Lsur} = **100** psf

Self weight $F_{swt} = h * \gamma_{conc} = \mathbf{175}$ psf

Soil weight $F_{soil} = h_{soil} * \gamma_{soil} = \mathbf{180}$ psf

Column no.1 loads

Dead load in z $F_{Dz1} = \mathbf{1.8}$ kips

Live load in z $F_{Lz1} = \mathbf{2.5}$ kips

Seismic load in z $F_{Ez1} = \mathbf{7.9}$ kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.211)

1.0D + 1.0L (0.354)

1.0D + 1.0Lr (0.211)

1.0D + 1.0S (0.211)

1.0D + 1.0R (0.211)

1.0D + 0.75L + 0.75Lr (0.319)

1.0D + 0.75L + 0.75S (0.319)

1.0D + 0.75L + 0.75R (0.319)

1.0D + 0.6W (0.211)

$(1.0 + 0.14 * S_{DS})$D + 0.7E (0.476)

1.0D + 0.75L + 0.75Lr + 0.45W (0.319)

1.0D + 0.75L + 0.75S + 0.45W (0.319)

1.0D + 0.75L + 0.75R + 0.45W (0.319)

$(1.0 + 0.10 * S_{DS})$D + 0.75L + 0.75S + 0.525E (0.516)

0.6D + 0.6W (0.127)

$(0.6 - 0.14 * S_{DS})$D + 0.7E (0.335)

Combination 14 results: $(1.0 + 0.10 * S_{DS})$D + 0.75L + 0.75S + 0.525E

Forces on foundation

Force in z-axis $F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_L * A * F_{Lsur} + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_E * F_{Ez1} = \mathbf{12.1}$ kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_L * A * F_{Lsur} * L_x / 2 + \gamma_D * (F_{Dz1} * x_1) + \gamma_L * (F_{Lz1} * x_1) + \gamma_E * (F_{Ez1} * x_1) = \mathbf{18.2}$ kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_L * A * F_{Lsur} * L_y / 2 + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) + \gamma_E * (F_{Ez1} * y_1) = \mathbf{18.2}$ kip_ft

Uplift verification

Vertical force $F_{dz} = \mathbf{12.106}$ kips

PASS - Foundation is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0}$ in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0}$ in

Pad base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y) = \mathbf{1.345}$ ksf

$q_2 = F_{dz} * (1 - 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y) = \mathbf{1.345}$ ksf

$q_3 = F_{dz} * (1 + 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y) = \mathbf{1.345}$ ksf

$q_4 = F_{dz} * (1 + 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y) = \mathbf{1.345}$ ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4) = \mathbf{1.345}$ ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4) = \mathbf{1.345}$ ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Net} + ((h + h_{soil}) * \gamma_{soil}) / FS_{soil} = \mathbf{2.607}$ ksf

$q_{max} / q_{allow} = \mathbf{0.516}$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete $f'_c = \mathbf{4000}$ psi

Yield strength of reinforcement $f_y = \mathbf{60000}$ psi

Compression-controlled strain limit (21.2.2) $\varepsilon_{ty} = \mathbf{0.00200}$

Cover to reinforcement $c_{nom} = \mathbf{3}$ in

Concrete type Normal weight

Concrete modification factor $\lambda = \mathbf{1.00}$

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.015)

1.2D + 1.6L + 0.5Lr (0.036)

1.2D + 1.6L + 0.5S (0.036)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on foundation

Ultimate force in z-axis $F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_L * A * F_{Lsur} + \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} = \mathbf{11.3}$ kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_L * A * F_{Lsur} * L_x / 2 + \gamma_D * (F_{Dz1} * x_1) + \gamma_L * (F_{Lz1} * x_1) = \mathbf{17.0}$ kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_L * A * F_{Lsur} * L_y / 2 + \gamma_D * (F_{Dz1} * y_1) + \gamma_L * (F_{Lz1} * y_1) = \mathbf{17.0}$ kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2 = \mathbf{0}$ in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0}$ in

Pad base pressures

$q_{u1} = F_{uz} * (1 - 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y) = \mathbf{1.258}$ ksf

$q_{u2} = F_{uz} * (1 - 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y) = \mathbf{1.258}$ ksf

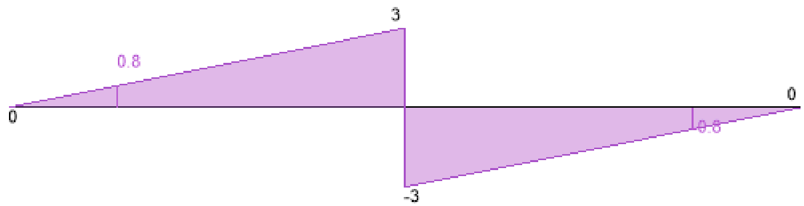
$q_{u3} = F_{uz} * (1 + 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y) = \mathbf{1.258}$ ksf

$q_{u4} = F_{uz} * (1 + 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y) = \mathbf{1.258}$ ksf

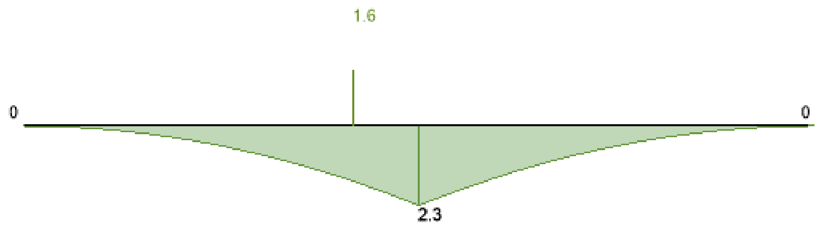
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{1.258}$ ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{1.258}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max} = \mathbf{1.577}$ kip_ft

Tension reinforcement provided 4 No.5 bottom bars (9.7 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov} = \mathbf{1.24}$ in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 * L_y * h = \mathbf{0.907}$ in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) $s_{max} = \min(2 * h, 18 \text{ in}) = \mathbf{18}$ in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.bot} / 2 = \mathbf{10.688}$ in

Depth of compression block $a = A_{sx.bot.prov} * f_y / (0.85 * f'_c * L_y) = \mathbf{0.608}$ in

Neutral axis factor $\beta_1 = \mathbf{0.85}$

Depth to neutral axis $c = a / \beta_1 = \mathbf{0.715}$ in

Strain in tensile reinforcement $\varepsilon_t = 0.003 * d / c - 0.003 = \mathbf{0.04184}$

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = 0.004 = \mathbf{0.00400}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sx.bot.prov} * f_y * (d - a / 2) = \mathbf{64.378}$ kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 * (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity $\phi M_n = \phi_f * M_n = \mathbf{57.94}$ kip_ft

$M_{u.x.max} / \phi M_n = \mathbf{0.027}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x} = \mathbf{0.831}$ kips

Depth to reinforcement $d_v = h - c_{nom} - \phi_{x.bot} / 2 = \mathbf{10.688}$ in

Shear strength reduction factor $\phi_v = \mathbf{0.75}$

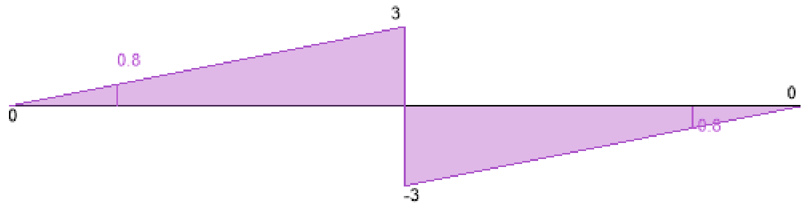
Nominal shear capacity (Eq. 22.5.5.1) $V_n = 2 * \lambda * \sqrt{(f'_c * 1 \text{ psi})} * L_y * d_v = \mathbf{48.667}$ kips

Design shear capacity $\phi V_n = \phi_v * V_n = \mathbf{36.501}$ kips

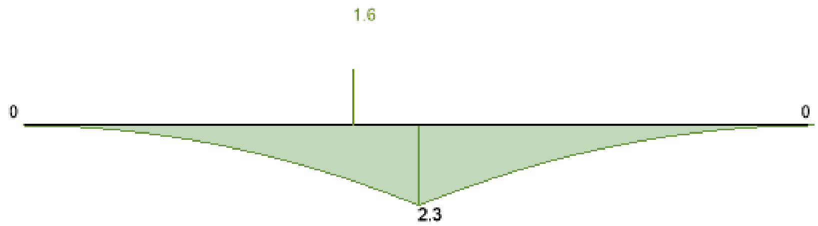
$V_{u.x} / \phi V_n$ = **0.023**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment — $M_{u.y.max}$ = **1.577** kip_ft

Tension reinforcement provided — 4 No.5 bottom bars (9.7 in c/c)

Area of tension reinforcement provided — $A_{sy.bot.prov}$ = **1.24** in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 * L_x * h$ = **0.907** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) — s_{max} = min(2 * h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **10.063** in

Depth of compression block — $a = A_{sy.bot.prov} * f_y / (0.85 * f'_c * L_x)$ = **0.608** in

Neutral axis factor — β_1 = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **0.715** in

Strain in tensile reinforcement — $\varepsilon_t = 0.003 * d / c - 0.003$ = **0.03921**

Minimum tensile strain(8.3.3.1) — $\varepsilon_{min} = 0.004$ = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sy.bot.prov} * f_y * (d - a / 2)$ = **60.503** kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 * (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**

Design moment capacity — $\phi M_n = \phi_f * M_n$ = **54.453** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.029**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force — $V_{u.y}$ = **0.831** kips

Depth to reinforcement — $d_v = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **10.063** in

Shear strength reduction factor — ϕ_v = **0.75**

Nominal shear capacity (Eq. 22.5.5.1) — $V_n = 2 * \lambda * \sqrt{(f'_c * 1 \text{ psi})} * L_x * d_v$ = **45.821** kips

Design shear capacity $\phi V_n = \phi_v * V_n =$ **34.366** kips

$V_{u.y} / \phi V_n =$ **0.024**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement $d_{v2} =$ **10.375** in

Shear perimeter length (22.6.4) $l_{xp} =$ **16.375** in

Shear perimeter width (22.6.4) $l_{yp} =$ **16.375** in

Shear perimeter (22.6.4) $b_o = 2 * (l_{x1} + d_{v2}) + 2 * (l_{y1} + d_{v2}) =$ **65.500** in

Shear area $A_p = l_{x,perim} * l_{y,perim} =$ **268.141** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} * l_{y1} =$ **232.141** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg} =$ **1.258** ksf

Ultimate shear load $F_{up} = \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_D * A_p * F_{swt} + \gamma_D * A_{sur} * F_{soil} + \gamma_L * A_{sur} * F_{Lsur} - q_{up.avg} * A_p =$ **4.703** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o * d_{v2}),0 \text{ psi}) =$ **6.921** psi

Column geometry factor (Table 22.6.5.2) $\beta = l_{y1} / l_{x1} =$ **1.00**

Column location factor (22.6.5.3) $\alpha_s =$ **40**

Concrete shear strength (22.6.5.2) $v_{cpa} = (2 + 4 / \beta) * \lambda * \sqrt{(f'_c * 1 \text{ psi})} =$ **379.473** psi

$v_{cpb} = (\alpha_s * d_{v2} / b_o + 2) * \lambda * \sqrt{(f'_c * 1 \text{ psi})} =$ **527.207** psi

$v_{cpc} = 4 * \lambda * \sqrt{(f'_c * 1 \text{ psi})} =$ **252.982** psi

$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc}) =$ **252.982** psi

Shear strength reduction factor $\phi_v =$ **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp} =$ **252.982** psi

Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v * v_n =$ **189.737** psi

$v_{ug} / \phi v_n =$ **0.036**

PASS - Design shear stress capacity exceeds ultimate shear stress load

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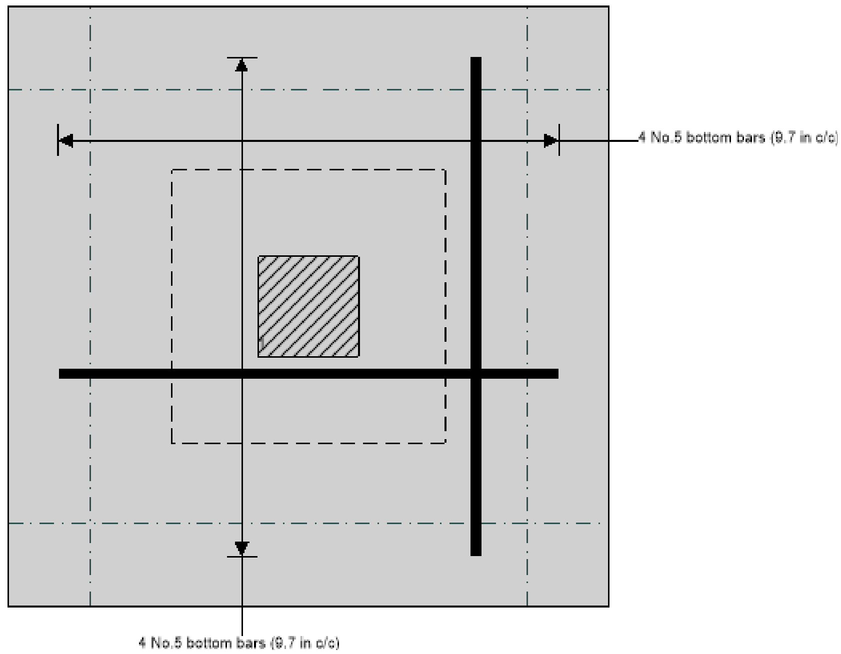

RETAINING WALL ANALYSIS

In accordance with International Building Code 2018

Tedds calculation version 2.9.08

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **9.6** ft
Stem thickness	t_{stem} = **10** in
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **150** pcf
Toe length	l_{toe} = **1** ft
Heel length	l_{heel} = **4.667** ft
Base thickness	t_{base} = **12** in
Base density	γ_{base} = **150** pcf
Height of retained soil	h_{ret} = **8.89** ft
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0.5** ft

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **135** pcf
Saturated density	γ_{sr} = **145** pcf
Prescribed active lateral soil pressure	p_{Ar} = **35** psf/ft

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **125** pcf
Prescribed passive lateral soil pressure	p_{0b} = **225** psf/ft
Allowable bearing pressure	$P_{bearing}$ = **2500** psf

Loading details

Live surcharge load	$Surcharge_L$ = **50** psf
Vertical line load at 1.417 ft	P_{D1} = **485** plf
	P_{L1} = **646** plf

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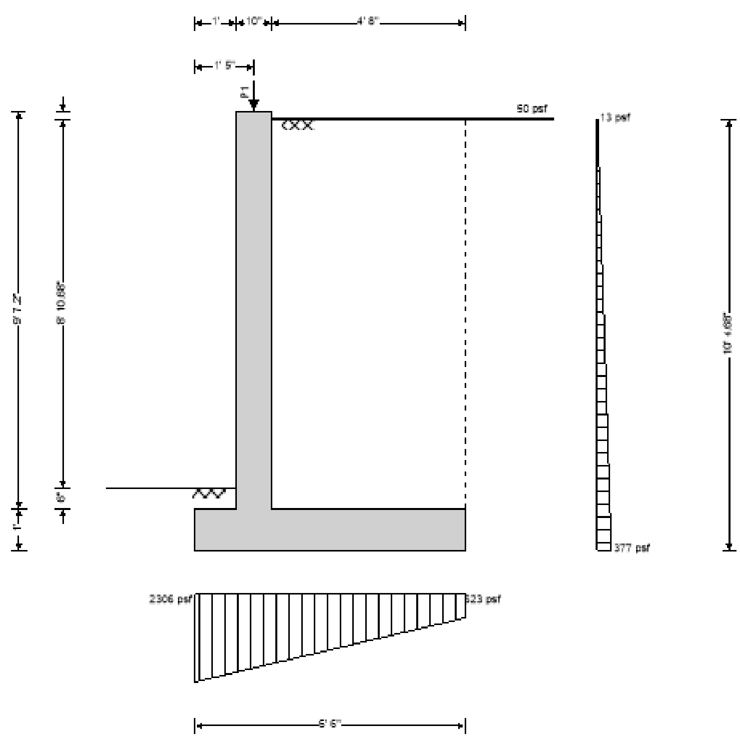

General arrangement

Calculate retaining wall geometry

Base length $l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{6.5}$ ft

Moist soil height $h_{moist} = h_{soil} = \mathbf{9.39}$ ft

Length of surcharge load $l_{sur} = l_{heel} = \mathbf{4.667}$ ft

- Distance to vertical component $x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{4.167}$ ft

Effective height of wall $h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{10.39}$ ft

- Distance to horizontal component $x_{sur\_h} = h_{eff} / 2 = \mathbf{5.195}$ ft

Area of wall stem $A_{stem} = h_{stem} * t_{stem} = \mathbf{8}$ ft^2

- Distance to vertical component $x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1.417}$ ft

Area of wall base $A_{base} = l_{base} * t_{base} = \mathbf{6.5}$ ft^2

- Distance to vertical component $x_{base} = l_{base} / 2 = \mathbf{3.25}$ ft

Area of moist soil $A_{moist} = h_{moist} * l_{heel} = \mathbf{43.82}$ ft^2

- Distance to vertical component $x_{moist\_v} = l_{base} - (h_{moist} * l_{heel}^2 / 2) / A_{moist} = \mathbf{4.167}$ ft

- Distance to horizontal component $x_{moist\_h} = h_{eff} / 3 = \mathbf{3.463}$ ft

Area of base soil $A_{pass} = d_{cover} * l_{toe} = \mathbf{0.5}$ ft^2

- Distance to vertical component $x_{pass\_v} = l_{base} - (d_{cover} * l_{toe} * (l_{base} - l_{toe} / 2)) / A_{pass} = \mathbf{0.5}$ ft

- Distance to horizontal component $x_{pass\_h} = (d_{cover} + h_{base}) / 3 = \mathbf{0.5}$ ft

Area of excavated base soil $A_{exc} = h_{pass} * l_{toe} = \mathbf{0.5}$ ft^2

- Distance to vertical component $x_{exc\_v} = l_{base} - (h_{pass} * l_{toe} * (l_{base} - l_{toe} / 2)) / A_{exc} = \mathbf{0.5}$ ft

- Distance to horizontal component $x_{exc\_h} = (h_{pass} + h_{base}) / 3 = \mathbf{0.5}$ ft

Soil coefficients

Coefficient of friction to back of wall — K_{fr} = **0.300**

Coefficient of friction to front of wall — K_{fb} = **0.300**

Coefficient of friction beneath base — K_{fbb} = **0.350**

From IBC 2018 cl.1807.2.3 Safety factor — ALSO CHECK 0.7EQ W/ F.O.S. = 1.1 FOR OVERTURNING & SLIDING

Load combination 1 — 1.0 * Dead + 1.0 * Live + 1.0 * Lateral earth

Sliding check

Vertical forces on wall

Wall stem — $F_{stem} = A_{stem} * \gamma_{stem}$ = **1200** plf

Wall base — $F_{base} = A_{base} * \gamma_{base}$ = **975** plf

Line loads — $F_{P\_v} = P_{D1} + 0 * P_{L1}$ = **485** plf

Moist retained soil — $F_{moist\_v} = A_{moist} * \gamma_{mr}$ = **5916** plf

Base soil — $F_{exc\_v} = A_{exc} * \gamma_b$ = **63** plf

Total — $F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{moist\_v} + F_{exc\_v}$ = **8638** plf

Horizontal forces on wall — Feq_h = 0.7 * 7H * heff = 488.7456 plf

Surcharge load — $F_{sur\_h} = p_{Ar} / \gamma_{mr} * Surcharge_L * h_{eff}$ = **135** plf

Moist retained soil — $F_{moist\_h} = p_{Ar} * h_{eff}^2 / 2$ = **1889** plf

Total — $F_{total\_h} = F_{sur\_h} + F_{moist\_h}$ = **2024** plf + Feq_h = 2513 plf

Check stability against sliding

Base soil resistance — $F_{exc\_h} = p_{0b} * (h_{pass} + h_{base})^2 / 2$ = **253** plf

Base friction — $F_{friction} = F_{total\_v} * K_{fbb}$ = **3023** plf

Resistance to sliding — $F_{rest} = F_{exc\_h} + F_{friction}$ = **3277** plf

Factor of safety — $FoS_{sl} = F_{rest} / F_{total\_h}$ = **1.619** > 1.5

CHECK WITH EQ:
3277 plf / 2513 plf = 1.3 > 1.1 OK

PASS - Factor of safety against sliding is adequate

Overturning check

Vertical forces on wall

Wall stem — $F_{stem} = A_{stem} * \gamma_{stem}$ = **1200** plf

Wall base — $F_{base} = A_{base} * \gamma_{base}$ = **975** plf

Line loads — $F_{P\_v} = P_{D1} + 0 * P_{L1}$ = **485** plf

Moist retained soil — $F_{moist\_v} = A_{moist} * \gamma_{mr}$ = **5916** plf

Base soil — $F_{exc\_v} = A_{exc} * \gamma_b$ = **63** plf

Total — $F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{moist\_v} + F_{exc\_v}$ = **8638** plf

Horizontal forces on wall

Surcharge load — $F_{sur\_h} = p_{Ar} / \gamma_{mr} * Surcharge_L * h_{eff}$ = **135** plf

Moist retained soil — $F_{moist\_h} = p_{Ar} * h_{eff}^2 / 2$ = **1889** plf

Base soil — $F_{exc\_h} = -p_{0b} * (h_{pass} + h_{base})^2 / 2$ = **-253** plf

Total — $F_{total\_h} = F_{sur\_h} + F_{moist\_h} + F_{exc\_h}$ = **1771** plf

Overturning moments on wall — Meq_OT = Feq_h * heff/2 = 2539 lb_ft/ft

Surcharge load — $M_{sur\_OT} = F_{sur\_h} * x_{sur\_h}$ = **700** lb_ft/ft

Moist retained soil — $M_{moist\_OT} = F_{moist\_h} * x_{moist\_h}$ = **6543** lb_ft/ft

Total — $M_{total\_OT} = M_{sur\_OT} + M_{moist\_OT}$ = **7242** lb_ft/ft + Meq_OT = 9781 lb_ft/ft

Restoring moments on wall

Wall stem $M_{stem\_R} = F_{stem} * x_{stem} = \mathbf{1700}$ lb_ft/ft

Wall base $M_{base\_R} = F_{base} * x_{base} = \mathbf{3169}$ lb_ft/ft

Line loads $M_{P\_R} = (abs(P_{D1} + 0 * P_{L1})) * p_1 = \mathbf{687}$ lb_ft/ft

Moist retained soil $M_{moist\_R} = F_{moist\_v} * x_{moist\_v} = \mathbf{24649}$ lb_ft/ft

Base soil $M_{exc\_R} = F_{exc\_v} * x_{exc\_v} - F_{exc\_h} * x_{exc\_h} = \mathbf{158}$ lb_ft/ft

Total $M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{P\_R} + M_{moist\_R} + M_{exc\_R} = \mathbf{30363}$ lb_ft/ft

Check stability against overturning

CHECK WITH EQ:

Factor of safety $FoS_{ot} = M_{total\_R} / M_{total\_OT} = \mathbf{4.192} > 1.5$ 30363 lb_ft/ft / 9781 lb_ft/ft = 3.1 > 1.1 OK

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} * \gamma_{stem} = \mathbf{1200}$ plf

Wall base $F_{base} = A_{base} * \gamma_{base} = \mathbf{975}$ plf

Surcharge load $F_{sur\_v} = Surcharge_L * l_{heel} = \mathbf{233}$ plf

Line loads $F_{P\_v} = P_{D1} + P_{L1} = \mathbf{1131}$ plf

Moist retained soil $F_{moist\_v} = A_{moist} * \gamma_{mr} = \mathbf{5916}$ plf

Base soil $F_{pass\_v} = A_{pass} * \gamma_b = \mathbf{63}$ plf

Total $F_{total\_v} = F_{stem} + F_{base} + F_{sur\_v} + F_{P\_v} + F_{moist\_v} + F_{pass\_v} = \mathbf{9518}$ plf

Horizontal forces on wall

Surcharge load $F_{sur\_h} = p_{Ar} / \gamma_{mr} * Surcharge_L * h_{eff} = \mathbf{135}$ plf

Moist retained soil $F_{moist\_h} = p_{Ar} * h_{eff}^2 / 2 = \mathbf{1889}$ plf

Base soil $F_{pass\_h} = -p_{0b} * (d_{cover} + h_{base})^2 / 2 = \mathbf{-253}$ plf

Total $F_{total\_h} = max(F_{sur\_h} + F_{moist\_h} + F_{pass\_h} - F_{total\_v} * K_{fbb}, 0 \text{ plf}) = \mathbf{0}$ plf

Moments on wall

Wall stem $M_{stem} = F_{stem} * x_{stem} = \mathbf{1700}$ lb_ft/ft

Wall base $M_{base} = F_{base} * x_{base} = \mathbf{3169}$ lb_ft/ft

Surcharge load $M_{sur} = F_{sur\_v} * x_{sur\_v} - F_{sur\_h} * x_{sur\_h} = \mathbf{273}$ lb_ft/ft

Line loads $M_P = ((P_{D1} + P_{L1})) * p_1 = \mathbf{1602}$ lb_ft/ft

Moist retained soil $M_{moist} = F_{moist\_v} * x_{moist\_v} - F_{moist\_h} * x_{moist\_h} = \mathbf{18106}$ lb_ft/ft

Base soil $M_{pass} = F_{pass\_v} * x_{pass\_v} - F_{pass\_h} * x_{pass\_h} = \mathbf{158}$ lb_ft/ft

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{moist} + M_{pass} = \mathbf{25008}$ lb_ft/ft

Check bearing pressure

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} = \mathbf{2.628}$ ft

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 = \mathbf{-0.622}$ ft

Loaded length of base $l_{load} = l_{base} = \mathbf{6.5}$ ft

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} * (1 - 6 * e / l_{base}) = \mathbf{2306}$ psf

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} * (1 + 6 * e / l_{base}) = \mathbf{623}$ psf

Factor of safety $FoS_{bp} = P_{bearing} / max(q_{toe}, q_{heel}) = \mathbf{1.084}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

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RETAINING WALL DESIGN

In accordance with ACI 318-14

Tedds calculation version 2.9.08

Concrete details

Compressive strength of concrete f'_c = **4000** psi

Concrete type Normal weight

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Modulus of elasticity or reinforcement E_s = **29000000** psi

Compression-controlled strain limit ε_{ty} = **0.002**

Cover to reinforcement

Front face of stem c_{sf} = **1.5** in

Rear face of stem c_{sr} = **2** in

Top face of base c_{bt} = **2** in

Bottom face of base c_{bb} = **3** in

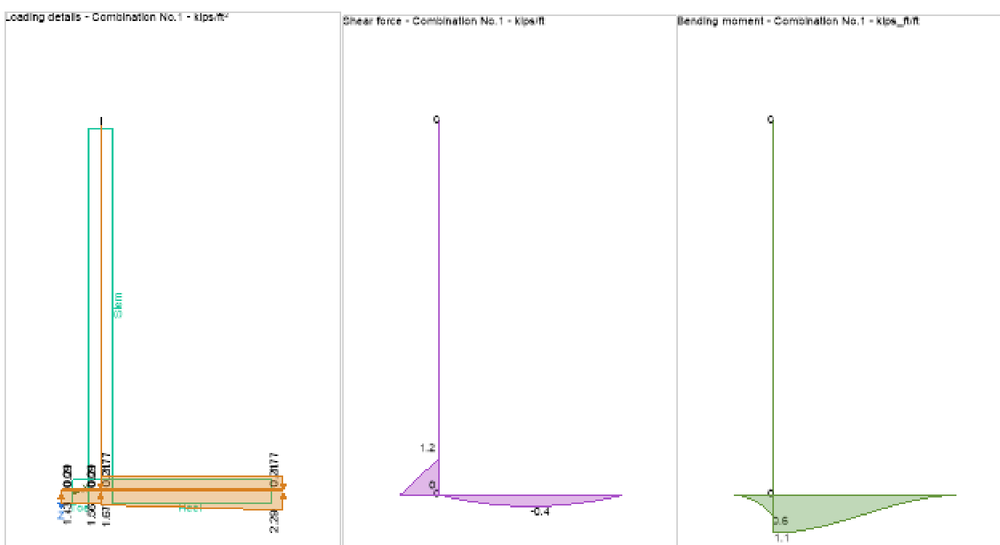
From IBC 2018 cl.1605.2 Basic load combinations

Load combination no.1 1.4 * Dead

Load combination no.2 1.2 * Dead + 1.6 * Live + 1.6 * Lateral earth

Load combination no.3 1.2 * Dead + 1.0 * Earthquake + 1.0 * Live + 1.6 * Lateral earth

Load combination no.4 0.9 * Dead + 1.0 * Earthquake + 1.6 * Lateral earth



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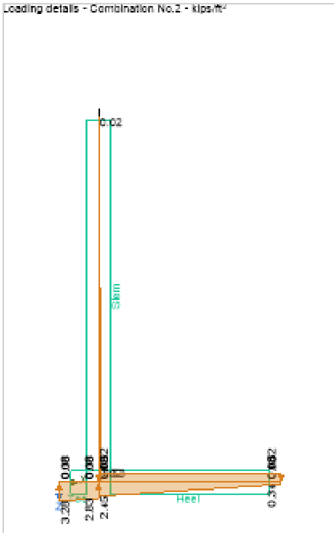
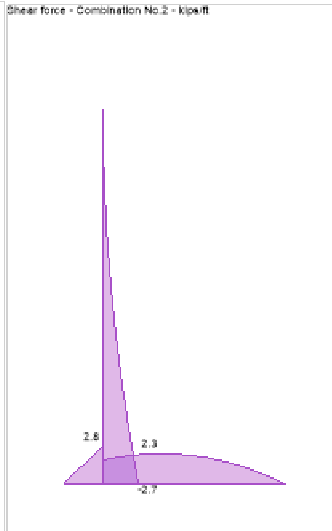
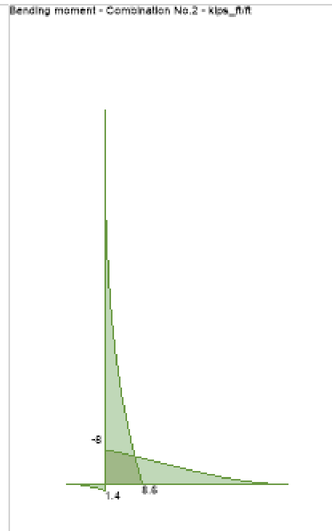
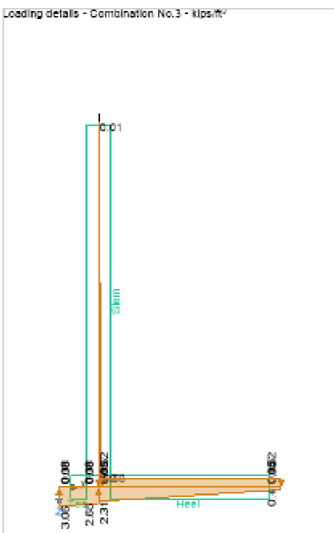
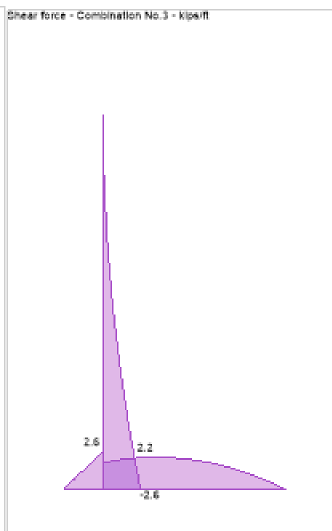
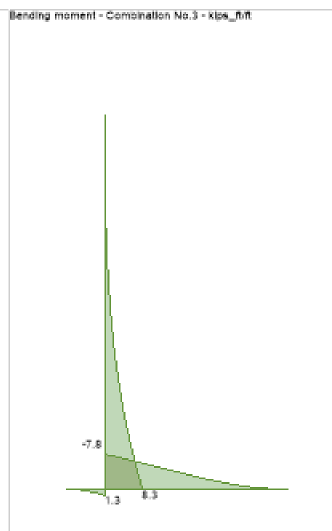


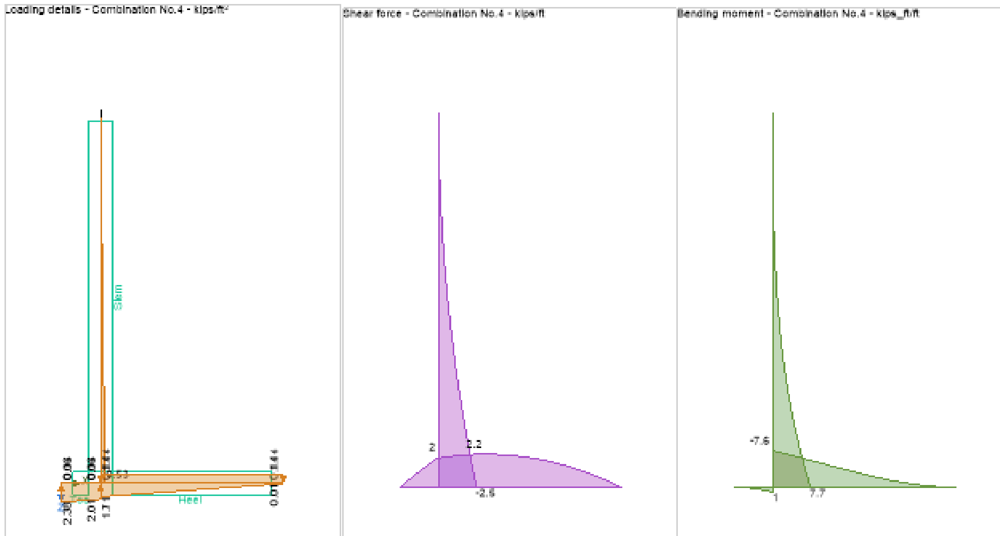












Check stem design at base of stem

Depth of section $h = \mathbf{10}$ in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 $M = \mathbf{8642}$ lb_ft/ft

Depth of tension reinforcement $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{7.625}$ in

Compression reinforcement provided No.5 bars @ 18" c/c

Area of compression reinforcement provided $A_{sf.prov} = \pi * \phi_{sf}^2 / (4 * s_{sf}) = \mathbf{0.205}$ in²/ft

Tension reinforcement provided No.6 bars @ 12" c/c

Area of tension reinforcement provided $A_{sr.prov} = \pi * \phi_{sr}^2 / (4 * s_{sr}) = \mathbf{0.442}$ in²/ft

Maximum reinforcement spacing - cl.11.7.2 $s_{max} = \min(18 \text{ in}, 3 * h) = \mathbf{18}$ in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{sr.prov} * f_y / (0.85 * f'_c) = \mathbf{0.65}$ in

Neutral axis factor - cl.22.2.2.4.3 $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) = \mathbf{0.85}$

Depth to neutral axis $c = a / \beta_1 = \mathbf{0.764}$ in

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c = \mathbf{0.026928}$

Section is in the tension controlled zone

Strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 *(\varepsilon_t - \varepsilon_{ty}) / 0.003, 0.65), 0.9) = \mathbf{0.9}$

Nominal flexural strength $M_n = A_{sr.prov} * f_y * (d - a / 2) = \mathbf{16126}$ lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n = \mathbf{14513}$ lb_ft/ft

$M / \phi M_n = \mathbf{0.595}$

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{sr.des} = \mathbf{0.258}$ in²/ft

Minimum area of reinforcement - cl.9.6.1.2 $A_{sr.min} = \max(3 * \sqrt{f'_c * 1 \text{ psi}}, 200 \text{ psi}) * d / f_y = \mathbf{0.305}$ in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force $V = \mathbf{2664}$ lb/ft

Concrete modification factor - cl.19.2.4 $\lambda = \mathbf{1}$

Nominal concrete shear strength - eqn.22.5.5.1 $V_c = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times d = \mathbf{11574}$ lb/ft

Strength reduction factor ϕ_s = **0.75**

Design concrete shear strength - cl.11.5.1.1 $\phi V_c = \phi_s \times V_c$ = **8680** lb/ft

$V / \phi V_c$ = **0.307**

PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.11.6.1 $A_{sx.req} = 0.002 * t_{stem}$ = **0.24** in^2/ft

Transverse reinforcement provided No.5 bars @ 18" c/c each face

Area of transverse reinforcement provided $A_{sx.prov} = 2 * \pi * \phi_{sx}^2 / (4 * s_{sx})$ = **0.409** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section h = **12** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 M = **1437** lb_ft/ft

Depth of tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2$ = **8.688** in

Compression reinforcement provided No.5 bars @ 12" c/c

Area of compression reinforcement provided $A_{bt.prov} = \pi * \phi_{bt}^2 / (4 * s_{bt})$ = **0.307** in^2/ft

Tension reinforcement provided No.5 bars @ 12" c/c

Area of tension reinforcement provided $A_{bb.prov} = \pi * \phi_{bb}^2 / (4 * s_{bb})$ = **0.307** in^2/ft

Maximum reinforcement spacing - cl.7.7.2.3 s_{max} = min(18 in, 3 * h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{bb.prov} * f_y / (0.85 * f'_c)$ = **0.451** in

Neutral axis factor - cl.22.2.2.4.3 β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.531** in

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c$ = **0.046101**

Section is in the tension controlled zone

Strength reduction factor ϕ_f = min(max(0.65 + 0.25 *(ε_t - ε_{ty}) / 0.003, 0.65), 0.9) = **0.9**

Nominal flexural strength $M_n = A_{bb.prov} * f_y * (d - a / 2)$ = **12980** lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n$ = **11682** lb_ft/ft

$M / \phi M_n$ = **0.123**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{bb.des}$ = **0.037** in^2/ft

Minimum area of reinforcement - cl.7.6.1.1 $A_{bb.min} = 0.0018 * h$ = **0.259** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force V = **2799** lb/ft

Concrete modification factor - cl.19.2.4 λ = **1**

Nominal concrete shear strength - eqn.22.5.5.1 $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d$ = **13187** lb/ft

Strength reduction factor ϕ_s = **0.75**

Design concrete shear strength - cl.7.6.3.1 $\phi V_c = \phi_s \times V_c$ = **9890** lb/ft

$V / \phi V_c$ = **0.283**

PASS - No shear reinforcement is required

Check base design at heel

Depth of section h = **12** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 — M = **8013** lb_ft/ft

Depth of tension reinforcement — $d = h - c_{bt} - \phi_{bt} / 2$ = **9.687** in

Compression reinforcement provided — No.5 bars @ 12" c/c

Area of compression reinforcement provided — $A_{bb.prov} = \pi * \phi_{bb}^2 / (4 * s_{bb})$ = **0.307** in²/ft

Tension reinforcement provided — No.5 bars @ 12" c/c

Area of tension reinforcement provided — $A_{bt.prov} = \pi * \phi_{bt}^2 / (4 * s_{bt})$ = **0.307** in²/ft

Maximum reinforcement spacing - cl.7.7.2.3 — s_{max} = min(18 in, 3 * h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block — $a = A_{bt.prov} * f_y / (0.85 * f'_c)$ = **0.451** in

Neutral axis factor - cl.22.2.2.4.3 — β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **0.531** in

Strain in reinforcement — $\varepsilon_t = 0.003 \times (d - c) / c$ = **0.051753**

Section is in the tension controlled zone

Strength reduction factor — ϕ_f = min(max(0.65 + 0.25 *($\varepsilon_t - \varepsilon_{ty}$) / 0.003, 0.65), 0.9) = **0.9**

Nominal flexural strength — $M_n = A_{bt.prov} * f_y * (d - a / 2)$ = **14514** lb_ft/ft

Design flexural strength — $\phi M_n = \phi_f \times M_n$ = **13063** lb_ft/ft

$M / \phi M_n$ = **0.613**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis — $A_{bt.des}$ = **0.186** in²/ft

Minimum area of reinforcement - cl.7.6.1.1 — $A_{bt.min}$ = 0.0018 * h = **0.259** in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force — V = **2291** lb/ft

Concrete modification factor - cl.19.2.4 — λ = **1**

Nominal concrete shear strength - eqn.22.5.5.1 — $V_c = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times d$ = **14705** lb/ft

Strength reduction factor — ϕ_s = **0.75**

Design concrete shear strength - cl.7.6.3.1 — $\phi V_c = \phi_s \times V_c$ = **11028** lb/ft

$V / \phi V_c$ = **0.208**

PASS - No shear reinforcement is required

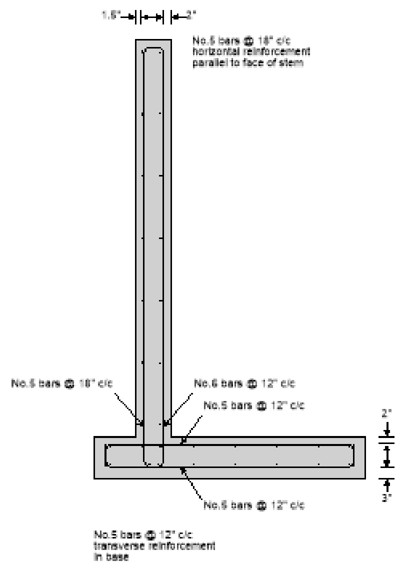
Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.7.6.1.1 — $A_{bx.req} = 0.0018 * t_{base}$ = **0.259** in²/ft

Transverse reinforcement provided — No.5 bars @ 12" c/c each face

Area of transverse reinforcement provided — $A_{bx.prov} = 2 * \pi * \phi_{bx}^2 / (4 * s_{bx})$ = **0.614** in²/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required



Reinforcement details

RETAINING WALL ANALYSIS

In accordance with International Building Code 2018

Tedds calculation version 2.9.08

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **9.39** ft
Stem thickness	t_{stem} = **8** in
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **150** pcf
Toe length	l_{toe} = **1** ft
Heel length	l_{heel} = **3.833** ft
Base thickness	t_{base} = **12** in
Base density	γ_{base} = **150** pcf
Height of retained soil	h_{ret} = **6** ft
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0.5** ft

Retained soil properties

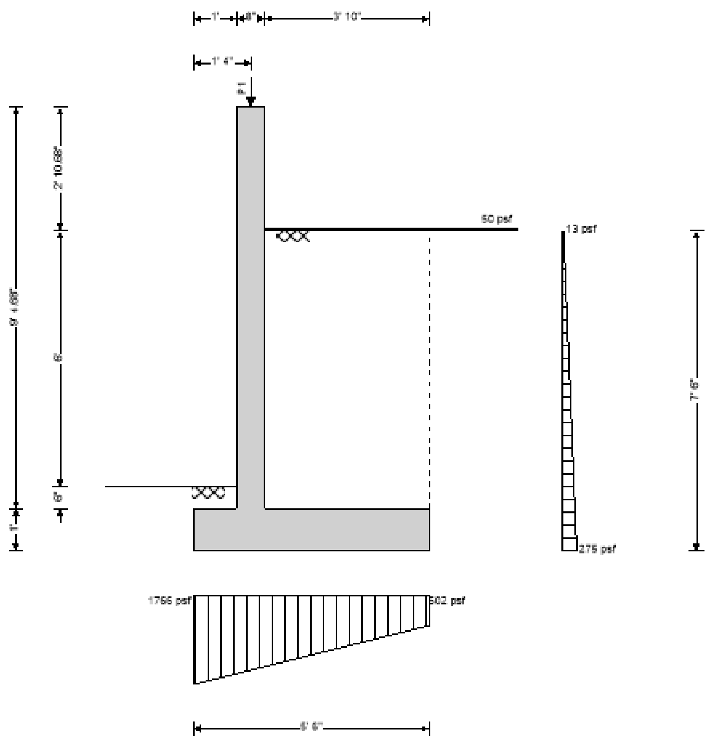
Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **135** pcf
Saturated density	γ_{sr} = **145** pcf
Prescribed active lateral soil pressure	p_{Ar} = **35** psf/ft

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **125** pcf
Prescribed passive lateral soil pressure	p_{0b} = **1** psf/ft
Allowable bearing pressure	$P_{bearing}$ = **2500** psf

Loading details

Live surcharge load	$Surcharge_L$ = **50** psf
Vertical line load at 1.333 ft	P_{D1} = **485** plf
	P_{L1} = **646** plf



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{5.5}$ ft
Moist soil height	$h_{moist} = h_{soil} = \mathbf{6.5}$ ft
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{3.833}$ ft
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{3.583}$ ft
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{7.5}$ ft
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{3.75}$ ft
Area of wall stem	$A_{stem} = h_{stem} * t_{stem} = \mathbf{6.26}$ ft^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1.333}$ ft
Area of wall base	$A_{base} = l_{base} * t_{base} = \mathbf{5.5}$ ft^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{2.75}$ ft
Area of moist soil	$A_{moist} = h_{moist} * l_{heel} = \mathbf{24.916}$ ft^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} * l_{heel}^2 / 2) / A_{moist} = \mathbf{3.583}$ ft
- Distance to horizontal component	$x_{moist\_h} = h_{eff} / 3 = \mathbf{2.5}$ ft
Area of base soil	$A_{pass} = d_{cover} * l_{toe} = \mathbf{0.5}$ ft^2
- Distance to vertical component	$x_{pass\_v} = l_{base} - (d_{cover} * l_{toe} * (l_{base} - l_{toe} / 2)) / A_{pass} = \mathbf{0.5}$ ft
- Distance to horizontal component	$x_{pass\_h} = (d_{cover} + h_{base}) / 3 = \mathbf{0.5}$ ft
Area of excavated base soil	$A_{exc} = h_{pass} * l_{toe} = \mathbf{0.5}$ ft^2
- Distance to vertical component	$x_{exc\_v} = l_{base} - (h_{pass} * l_{toe} * (l_{base} - l_{toe} / 2)) / A_{exc} = \mathbf{0.5}$ ft
- Distance to horizontal component	$x_{exc\_h} = (h_{pass} + h_{base}) / 3 = \mathbf{0.5}$ ft

Soil coefficients

Coefficient of friction to back of wall	K_{fr} = **0.300**
Coefficient of friction to front of wall	K_{fb} = **0.300**
Coefficient of friction beneath base	K_{fbb} = **0.350**

From IBC 2018 cl.1807.2.3 Safety factor ALSO CHECK 0.7EQ W/ F.O.S. = 1.1 FOR OVERTURNING & SLIDING

Load combination 1 1.0 * Dead + 1.0 * Live + 1.0 * Lateral earth

Sliding check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} * \gamma_{stem}$ = **939** plf
Wall base	$F_{base} = A_{base} * \gamma_{base}$ = **825** plf
Line loads	$F_{P\_v} = P_{D1} + 0 * P_{L1}$ = **485** plf
Moist retained soil	$F_{moist\_v} = A_{moist} * \gamma_{mr}$ = **3364** plf
Base soil	$F_{exc\_v} = A_{exc} * \gamma_b$ = **63** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{moist\_v} + F_{exc\_v}$ = **5675** plf

Horizontal forces on wall Feq_h = 0.7 * 7H * heff = 345.0825 plf

Surcharge load	$F_{sur\_h} = p_{Ar} / \gamma_{mr} * Surcharge_L * h_{eff}$ = **97** plf
Moist retained soil	$F_{moist\_h} = p_{Ar} * h_{eff}^2 / 2$ = **984** plf
Total	$F_{total\_h} = F_{sur\_h} + F_{moist\_h}$ = **1082** plf + Feq_h = 1428 plf

Check stability against sliding

Base soil resistance	$F_{exc\_h} = p_{0b} * (h_{pass} + h_{base})^2 / 2$ = **1** plf
Base friction	$F_{friction} = F_{total\_v} * K_{fbb}$ = **1986** plf
Resistance to sliding	$F_{rest} = F_{exc\_h} + F_{friction}$ = **1987** plf
Factor of safety	$FoS_{sl} = F_{rest} / F_{total\_h}$ = **1.838** > 1.5

CHECK WITH EQ:
1987 plf / 1428 plf = 1.39 > 1.1 OK

PASS - Factor of safety against sliding is adequate

Overturning check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} * \gamma_{stem}$ = **939** plf
Wall base	$F_{base} = A_{base} * \gamma_{base}$ = **825** plf
Line loads	$F_{P\_v} = P_{D1} + 0 * P_{L1}$ = **485** plf
Moist retained soil	$F_{moist\_v} = A_{moist} * \gamma_{mr}$ = **3364** plf
Base soil	$F_{exc\_v} = A_{exc} * \gamma_b$ = **63** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{moist\_v} + F_{exc\_v}$ = **5675** plf

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = p_{Ar} / \gamma_{mr} * Surcharge_L * h_{eff}$ = **97** plf
Moist retained soil	$F_{moist\_h} = p_{Ar} * h_{eff}^2 / 2$ = **984** plf
Base soil	$F_{exc\_h} = -p_{0b} * (h_{pass} + h_{base})^2 / 2$ = **-1** plf
Total	$F_{total\_h} = F_{sur\_h} + F_{moist\_h} + F_{exc\_h}$ = **1080** plf

Overturning moments on wall Meq_OT = Feq_h * heff/2 = 1295 lb_ft/ft

Surcharge load	$M_{sur\_OT} = F_{sur\_h} * x_{sur\_h}$ = **365** lb_ft/ft
Moist retained soil	$M_{moist\_OT} = F_{moist\_h} * x_{moist\_h}$ = **2461** lb_ft/ft
Total	$M_{total\_OT} = M_{sur\_OT} + M_{moist\_OT}$ = **2826** lb_ft/ft + Meq_OT = 4121 lb_ft/ft

Restoring moments on wall

Wall stem: $M_{stem\_R} = F_{stem} * x_{stem} = \mathbf{1252}$ lb_ft/ft

Wall base: $M_{base\_R} = F_{base} * x_{base} = \mathbf{2269}$ lb_ft/ft

Line loads: $M_{P\_R} = (abs(P_{D1} + 0 * P_{L1})) * p_1 = \mathbf{647}$ lb_ft/ft

Moist retained soil: $M_{moist\_R} = F_{moist\_v} * x_{moist\_v} = \mathbf{12053}$ lb_ft/ft

Base soil: $M_{exc\_R} = F_{exc\_v} * x_{exc\_v} - F_{exc\_h} * x_{exc\_h} = \mathbf{32}$ lb_ft/ft

Total: $M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{P\_R} + M_{moist\_R} + M_{exc\_R} = \mathbf{16252}$ lb_ft/ft

Check stability against overturning

CHECK WITH EQ:

Factor of safety: $FoS_{ot} = M_{total\_R} / M_{total\_OT} = \mathbf{5.752}$ > 1.5 16252 lb_ft/ft / 4121 lb_ft/ft = 3.9 > 1.1 OK

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem: $F_{stem} = A_{stem} * \gamma_{stem} = \mathbf{939}$ plf

Wall base: $F_{base} = A_{base} * \gamma_{base} = \mathbf{825}$ plf

Surcharge load: $F_{sur\_v} = Surcharge_L * l_{heel} = \mathbf{192}$ plf

Line loads: $F_{P\_v} = P_{D1} + P_{L1} = \mathbf{1131}$ plf

Moist retained soil: $F_{moist\_v} = A_{moist} * \gamma_{mr} = \mathbf{3364}$ plf

Base soil: $F_{pass\_v} = A_{pass} * \gamma_b = \mathbf{63}$ plf

Total: $F_{total\_v} = F_{stem} + F_{base} + F_{sur\_v} + F_{P\_v} + F_{moist\_v} + F_{pass\_v} = \mathbf{6513}$ plf

Horizontal forces on wall

Surcharge load: $F_{sur\_h} = p_{Ar} / \gamma_{mr} * Surcharge_L * h_{eff} = \mathbf{97}$ plf

Moist retained soil: $F_{moist\_h} = p_{Ar} * h_{eff}^2 / 2 = \mathbf{984}$ plf

Base soil: $F_{pass\_h} = -p_{0b} * (d_{cover} + h_{base})^2 / 2 = \mathbf{-1}$ plf

Total: $F_{total\_h} = max(F_{sur\_h} + F_{moist\_h} + F_{pass\_h} - F_{total\_v} * K_{fbb}, 0 \text{ plf}) = \mathbf{0}$ plf

Moments on wall

Wall stem: $M_{stem} = F_{stem} * x_{stem} = \mathbf{1252}$ lb_ft/ft

Wall base: $M_{base} = F_{base} * x_{base} = \mathbf{2269}$ lb_ft/ft

Surcharge load: $M_{sur} = F_{sur\_v} * x_{sur\_v} - F_{sur\_h} * x_{sur\_h} = \mathbf{322}$ lb_ft/ft

Line loads: $M_P = ((P_{D1} + P_{L1})) * p_1 = \mathbf{1508}$ lb_ft/ft

Moist retained soil: $M_{moist} = F_{moist\_v} * x_{moist\_v} - F_{moist\_h} * x_{moist\_h} = \mathbf{9592}$ lb_ft/ft

Base soil: $M_{pass} = F_{pass\_v} * x_{pass\_v} - F_{pass\_h} * x_{pass\_h} = \mathbf{32}$ lb_ft/ft

Total: $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{moist} + M_{pass} = \mathbf{14975}$ lb_ft/ft

Check bearing pressure

Distance to reaction: $\bar{x} = M_{total} / F_{total\_v} = \mathbf{2.299}$ ft

Eccentricity of reaction: $e = \bar{x} - l_{base} / 2 = \mathbf{-0.451}$ ft

Loaded length of base: $l_{load} = l_{base} = \mathbf{5.5}$ ft

Bearing pressure at toe: $q_{toe} = F_{total\_v} / l_{base} * (1 - 6 * e / l_{base}) = \mathbf{1766}$ psf

Bearing pressure at heel: $q_{heel} = F_{total\_v} / l_{base} * (1 + 6 * e / l_{base}) = \mathbf{602}$ psf

Factor of safety: $FoS_{bp} = P_{bearing} / max(q_{toe}, q_{heel}) = \mathbf{1.415}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with ACI 318-14

Tedds calculation version 2.9.08

Concrete details

Compressive strength of concrete f'_c = **4000** psi

Concrete type Normal weight

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Modulus of elasticity or reinforcement E_s = **29000000** psi

Compression-controlled strain limit ε_{ty} = **0.002**

Cover to reinforcement

Front face of stem c_{sf} = **1.5** in

Rear face of stem c_{sr} = **2** in

Top face of base c_{bt} = **2** in

Bottom face of base c_{bb} = **3** in

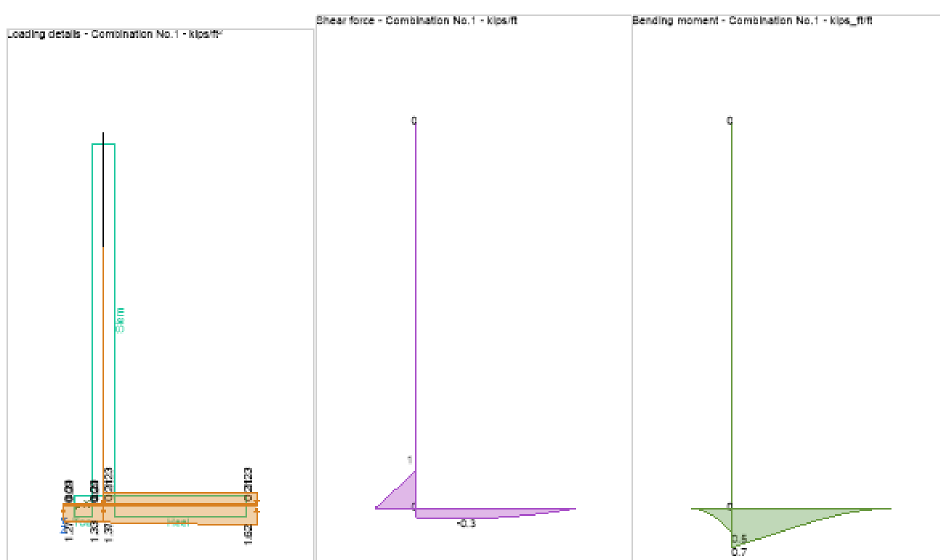
From IBC 2018 cl.1605.2 Basic load combinations

Load combination no.1 1.4 * Dead

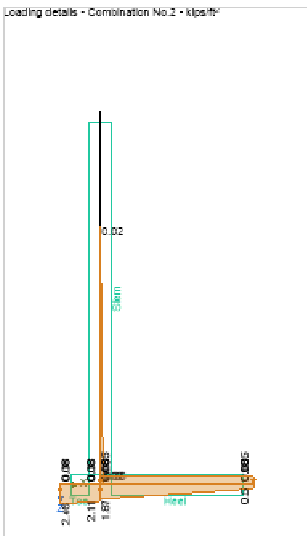
Load combination no.2 1.2 * Dead + 1.6 * Live + 1.6 * Lateral earth

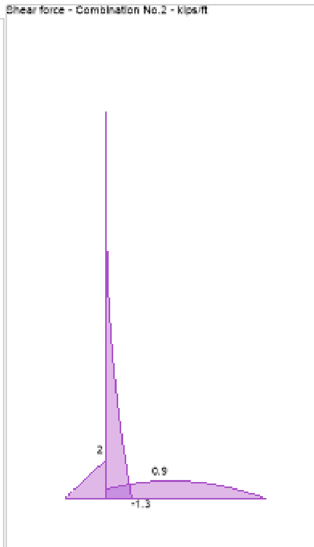
Load combination no.3 1.2 * Dead + 1.0 * Earthquake + 1.0 * Live + 1.6 * Lateral earth

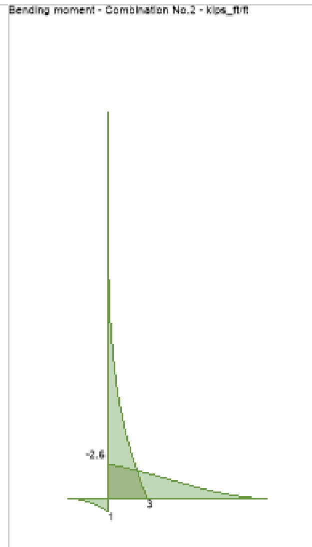
Load combination no.4 0.9 * Dead + 1.0 * Earthquake + 1.6 * Lateral earth

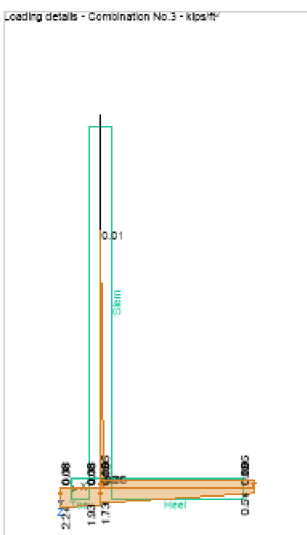


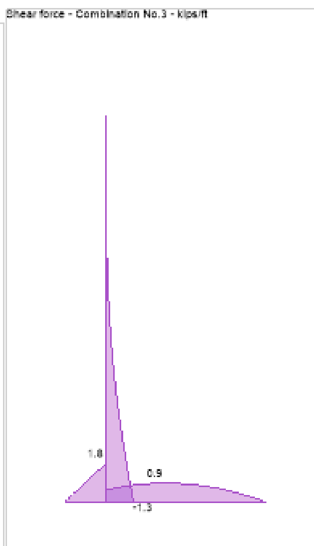
Tekla Tedds Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsky Residence			Job Ref. 8119	
	Section 8" Cantilever Retaining Wall - 6 ft Soil			Sheet no./rev. 6	
	Calc. by BJW	Date 3/3/2021	Chk'd by	Date	App'd by	Date

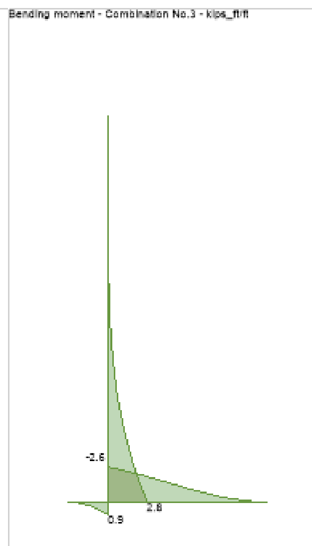




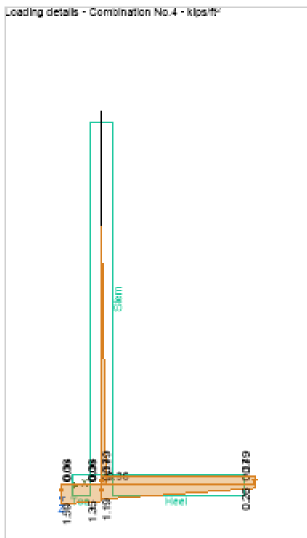


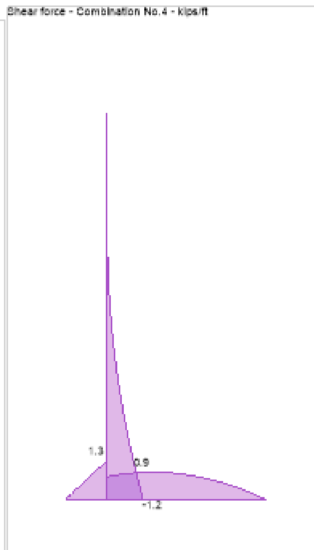


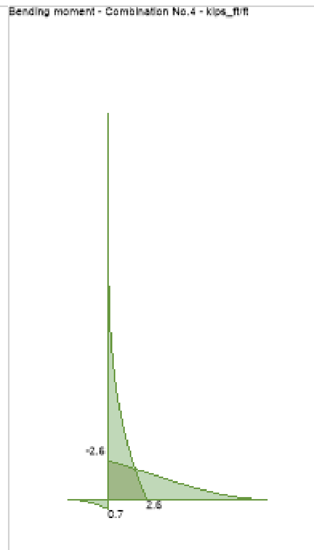




Tekla Tedds Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsky Residence			Job Ref. 8119	
	Section 8" Cantilever Retaining Wall - 6 ft Soil			Sheet no./rev. 7	
	Calc. by BJW	Date 3/3/2021	Chk'd by	Date	App'd by	Date







Check stem design at base of stem

Depth of section — $h = \mathbf{8}$ in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 — $M = \mathbf{3001}$ lb_ft/ft

Depth of tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{5.688}$ in

Compression reinforcement provided — No.4 bars @ 18" c/c

Area of compression reinforcement provided — $A_{sf.prov} = \pi * \phi_{sf}^2 / (4 * s_{sf}) = \mathbf{0.131}$ in²/ft

Tension reinforcement provided — No.5 bars @ 8" c/c

Area of tension reinforcement provided — $A_{sr.prov} = \pi * \phi_{sr}^2 / (4 * s_{sr}) = \mathbf{0.46}$ in²/ft

Maximum reinforcement spacing - cl.11.7.2 — $s_{max} = \min(18\text{ in}, 3 * h) = \mathbf{18}$ in

PASS - Reinforcement is adequately spaced

Depth of compression block — $a = A_{sr.prov} * f_y / (0.85 * f'_c) = \mathbf{0.677}$ in

Neutral axis factor - cl.22.2.2.4.3 — $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\text{ ksi}) / 1\text{ ksi}, 0.65), 0.85) = \mathbf{0.85}$

Depth to neutral axis — $c = a / \beta_1 = \mathbf{0.796}$ in

Strain in reinforcement — $\varepsilon_t = 0.003 \times (d - c) / c = \mathbf{0.01843}$

Section is in the tension controlled zone

Strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 * (\varepsilon_t - \varepsilon_{ty}) / 0.003, 0.65), 0.9) = \mathbf{0.9}$

Nominal flexural strength — $M_n = A_{sr.prov} * f_y * (d - a / 2) = \mathbf{12308}$ lb_ft/ft

Design flexural strength — $\phi M_n = \phi_f \times M_n = \mathbf{11077}$ lb_ft/ft

$M / \phi M_n = \mathbf{0.271}$

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis — $A_{sr.des} = \mathbf{0.119}$ in²/ft

Minimum area of reinforcement - cl.9.6.1.3 — $A_{sr.mod} = 4 * A_{sr.des} / 3 = \mathbf{0.159}$ in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force — $V = \mathbf{1318}$ lb/ft

Concrete modification factor - cl.19.2.4 — $\lambda = \mathbf{1}$

Nominal concrete shear strength - eqn.22.5.5.1 — $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times d = \mathbf{8633}$ lb/ft

Strength reduction factor ϕ_s = **0.75**

Design concrete shear strength - cl.11.5.1.1 $\phi V_c = \phi_s \times V_c$ = **6475** lb/ft

$V / \phi V_c$ = **0.204**

PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.11.6.1 $A_{sx.req} = 0.002 * t_{stem}$ = **0.192** in^2/ft

Transverse reinforcement provided No.5 bars @ 18" c/c each face

Area of transverse reinforcement provided $A_{sx.prov} = 2 * \pi * \phi_{sx}^2 / (4 * s_{sx})$ = **0.409** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section h = **12** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 M = **1045** lb_ft/ft

Depth of tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2$ = **8.688** in

Compression reinforcement provided No.5 bars @ 8" c/c

Area of compression reinforcement provided $A_{bt.prov} = \pi * \phi_{bt}^2 / (4 * s_{bt})$ = **0.46** in^2/ft

Tension reinforcement provided No.5 bars @ 8" c/c

Area of tension reinforcement provided $A_{bb.prov} = \pi * \phi_{bb}^2 / (4 * s_{bb})$ = **0.46** in^2/ft

Maximum reinforcement spacing - cl.7.7.2.3 s_{max} = min(18 in, 3 * h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{bb.prov} * f_y / (0.85 * f'_c)$ = **0.677** in

Neutral axis factor - cl.22.2.2.4.3 β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.796** in

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c$ = **0.029734**

Section is in the tension controlled zone

Strength reduction factor ϕ_f = min(max(0.65 + 0.25 *($\varepsilon_t - \varepsilon_{ty}$) / 0.003, 0.65), 0.9) = **0.9**

Nominal flexural strength $M_n = A_{bb.prov} * f_y * (d - a / 2)$ = **19211** lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n$ = **17290** lb_ft/ft

$M / \phi M_n$ = **0.060**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{bb.des}$ = **0.027** in^2/ft

Minimum area of reinforcement - cl.7.6.1.1 $A_{bb.min} = 0.0018 * h$ = **0.259** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force V = **2031** lb/ft

Concrete modification factor - cl.19.2.4 λ = **1**

Nominal concrete shear strength - eqn.22.5.5.1 $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times d$ = **13187** lb/ft

Strength reduction factor ϕ_s = **0.75**

Design concrete shear strength - cl.7.6.3.1 $\phi V_c = \phi_s \times V_c$ = **9890** lb/ft

$V / \phi V_c$ = **0.205**

PASS - No shear reinforcement is required

Check base design at heel

Depth of section h = **12** in

Rectangular section in flexure - Section 22.3

Design bending moment combination 2 — M = **2624** lb_ft/ft

Depth of tension reinforcement — $d = h - c_{bt} - \phi_{bt} / 2 =$ **9.687** in

Compression reinforcement provided — No.5 bars @ 8" c/c

Area of compression reinforcement provided — $A_{bb.prov} = \pi * \phi_{bb}^2 / (4 * s_{bb}) =$ **0.46** in²/ft

Tension reinforcement provided — No.5 bars @ 8" c/c

Area of tension reinforcement provided — $A_{bt.prov} = \pi * \phi_{bt}^2 / (4 * s_{bt}) =$ **0.46** in²/ft

Maximum reinforcement spacing - cl.7.7.2.3 — $s_{max} = \min(18 \text{ in}, 3 * h) =$ **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block — $a = A_{bt.prov} * f_y / (0.85 * f'_c) =$ **0.677** in

Neutral axis factor - cl.22.2.2.4.3 — $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) =$ **0.85**

Depth to neutral axis — $c = a / \beta_1 =$ **0.796** in

Strain in reinforcement — $\varepsilon_t = 0.003 \times (d - c) / c =$ **0.033502**

Section is in the tension controlled zone

Strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 *(\varepsilon_t - \varepsilon_{ty}) / 0.003, 0.65), 0.9) =$ **0.9**

Nominal flexural strength — $M_n = A_{bt.prov} * f_y * (d - a / 2) =$ **21512** lb_ft/ft

Design flexural strength — $\phi M_n = \phi_f \times M_n =$ **19361** lb_ft/ft

$M / \phi M_n =$ **0.136**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis — $A_{bt.des} =$ **0.06** in²/ft

Minimum area of reinforcement - cl.7.6.1.1 — $A_{bt.min} = 0.0018 * h =$ **0.259** in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Section 22.5

Design shear force — V = **927** lb/ft

Concrete modification factor - cl.19.2.4 — $\lambda =$ **1**

Nominal concrete shear strength - eqn.22.5.5.1 — $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d =$ **14705** lb/ft

Strength reduction factor — $\phi_s =$ **0.75**

Design concrete shear strength - cl.7.6.3.1 — $\phi V_c = \phi_s \times V_c =$ **11028** lb/ft

$V / \phi V_c =$ **0.084**

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

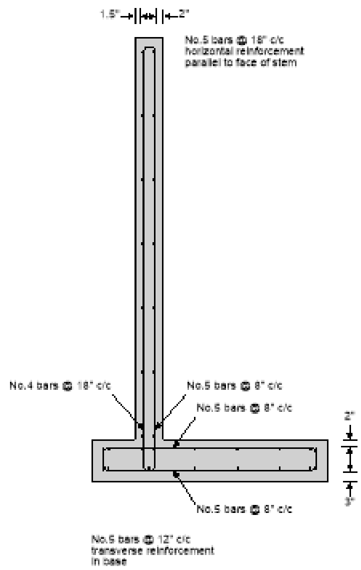
Minimum area of reinforcement - cl.7.6.1.1 — $A_{bx.req} = 0.0018 * t_{base} =$ **0.259** in²/ft

Transverse reinforcement provided — No.5 bars @ 12" c/c each face

Area of transverse reinforcement provided — $A_{bx.prov} = 2 * \pi * \phi_{bx}^2 / (4 * s_{bx}) =$ **0.614** in²/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

Tekla Tedds Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsky Residence			Job Ref. 8119	
	Section 8" Cantilever Retaining Wall - 6 ft Soil			Sheet no./rev. 10	
	Calc. by BJW	Date 3/3/2021	Chk'd by	Date	App'd by	Date



Reinforcement details