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

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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):
 - o SBC 2018
 - o IBC 2018
 - o ASCE 7-16

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PROJECT: Yaroslavsky Residence SUBJECT: Gravity Loading DESIGN BY: BJW		PROJECT NUMBER: DATE:	8119 20210210
NOTES:			
			INPUT
1.2 - DEAD LOAD (PSF)			RESULT
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

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:	2020-10-30T18:33:30.626Z
Hazard Type:	Wind



ASCE 7-16

ASCE 7-10

ASCE 7-05

MRI 10-Year 67 mph	MRI 10-Year 72 mph	ASCE 7-05 Wind Speed 85 mph
MRI 25-Year	MRI 25-Year	
MRI 50-Year 78 mph	MRI 50-Year 85 mph	
MRI 100-Year 83 mph	MRI 100-Year 91 mph	
Risk Category I 92 mph	Risk Category I 100 mph	
Risk Category II 98 mph	Risk Category II 110 mph	
Risk Category III 105 mph	Risk Category III-IV 115 mph	
Risk Category IV 108 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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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)

Evenesian Catagory		D		
Exposure Category		В	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	
Gust Effect Factor	G _{used} =	0.85	Sec. 26.11	CONSERVATIVE, PER —— RECOMMENDATION DURING
Enclosure Type		Enclosed	Sec. 26.12	PRE-APPLICATION MEETING
Importance Factor	lw =	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		
	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	н	Kz	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	

	Surface	1.2	Ср	_
Fig 27.3-1	Windwa	0.8	qz	
	Leeward	-0.46	qh	
	L/B 0-1	1	-0.5	
	L/B 2	2	-0.3	
	L/B >= 4	4	-0.2	
	Side Wa	-0.7	qh	

Sec.26.7.3	2	Exp Cat
	1	A
	2	В
	3	С
	4	D

<u>L/B</u>			
Surface	1.17	Ср	_
Windwa	rd 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 Wal	-0.7	qh	

Table	Calculate	e Kz								
26.9-1	а	z _g (ft)	а	b	a-bar	b-bar	С		oislon bi	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
Н	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

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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)

Evenesium Catagoriu		D		
Exposure Category		В	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	
Gust Effect Factor	G _{used} =	0.85	Sec. 26.11	CONSERVATIVE, PER —— RECOMMENDATION DURING
Enclosure Type		Enclosed	Sec. 26.12	PRE-APPLICATION MEETING
Importance Factor	lw =	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	7	
Cp - WW	0.8		
Cp - LW	-0.5	Worst Case	-0.5
Cp - SW	-0.7		

Description	Floor	Story H	н	Kz	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	

	Surface	0.9	Ср	_
Fig 27.3-1	Windwa	0.8	qz	
	Leeward	-0.5	qh	
	L/B 0-1	1	-0.5	
	L/B 2	2	-0.3	
	L/B >= 4	4	-0.2	
	Side Wal	-0.7	qh	

Sec.26.7.3	2	Exp Cat
	1	A
	2	В
	3	С
	4	D

<u>L/B</u>			
Surface	0.86	Ср	_
Windwa	rd 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 Wal	I	-0.7	qh

Table	Calculat	e Kz								
26.9-1	а	z _g (ft)	а	b	a-bar	b-bar	С	—	oislon bi	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
Н	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 980 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
Fa	1	Site amplification factor at 0.2s
Fv	* 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

ΤL	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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Tekla Tedds	Project Yaroslavsky	y Residence	Job Ref. 8119			
Fast + Epp 323 Dean Street, Suite #3	Section				Sheet no./rev.	
Brooklyn, NY 11217	1.6 Seismic	Loads - LFW (X	-Direction)		1	
	Calc. by BW	Date 2/23/2021	Chk'd by	Date	App'd by	Date
SEISMIC FORCES (ASCE 7-16)						
Site nerometero					Tedds calo	culation version
Site parameters Site class		D utilizina e	exception per 11	4 8(2)		
Mapped acceleration parameters	(Section 11.4	-	sception per 1	1.4.0(2)		
at short period		Ss = 1.415				
at 1 sec period		S ₁ = 0.492				
Site coefficientat short period (Ta	ble 11 4-1)	$F_a = 1.000$				
at 1 sec period (Table 11.4-2)		$F_v = 1.808$				
Spectral response acceleration	n parameters					
at short period (Eq. 11.4-1)	•	Sмs = Fa * S	s = 1.415			
at 1 sec period (Eq. 11.4-2)		Sм1 = Fv * S				
Design spectral acceleration pa	arameters (Se	ct 11.4.4)				
at short period (Eq. 11.4-3)	•	-	* Sмs = 0.943			
at 1 sec period (Eq. 11.4-4)		S _{D1} = 2 / 3 *				
Seismic design category						
Occupancy category (Table 1-1)		Ш				
Seismic design category based of	n short period	response acceler	ation (Table 11	6-1)		
Cersmic design calegory based of	an anon penou	D		.0-1)		
Seismic design category based c	on 1 sec period	response accele	ration (Table 11	1.6-2)		
		D	,			
Seismic design category		D				
Approximate fundamental peri	od					
Height above base to highest lev		hn = 30.69 ft				
From Table 12.8-2:						
Structure type		All other sys	tems			
Building period parameter Ct		$C_t = 0.02$				
Building period parameter x		x = 0.75				
Approximate fundamental period	(Eq 12.8-7)	$T_a = C_t * (h_n)$) [×] * 1sec / (1ft)×:	= 0.261 sec		
Building fundamental period (Sec	· · /	$T = T_a = 0.2$. ,			
Long-period transition period		T∟ = 6 sec				
Limiting period		Ts = S _{D1} / S	os * 1 sec = 0.6	29 sec		
Seismic response coefficient						
Seismic force-resisting system (T	able 12.2-1)	A. Bearing_	Wall_Systems			
		15. Light-fra	me (wood) wall	s sheathed wit	h wood structura	l panels
Response modification factor (Ta	ble 12.2-1)	R = 6.5				
Seismic importance factor (Table		le = 1.000				
Seismic response coefficient (Se	ct 11.4.8)					
Calculated (Eq 12.8-3)			/ (R / Ie) = 0.14			
		$C \rightarrow max$	0 044 * 500 * 1	e,0.01) = 0.04 1	5	
Minimum (Eq 9.5.5.2.1-3)		$Cs_min = IIIax$.(0.044 305 1	e,0.01) - 0.0 4		

Tekla Tedds Fast + Epp	Project Yaroslavsky Re	esidence			Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section 1.6 Seismic Loa	ads - LFW (X-Di	rection)		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	Cs = 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
Vertical distribution of seismic forces (Sect 12	2.8.3)	PROVISIONS
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)	
	$F_{\text{pxmin}} = 0.2 * S_{\text{DS}} * I_{\text{e}} * w_{\text{px}}$	
	$F_{pxmax} = 0.4 * S_{DS} * I_{e} * W_{px}$	
Vertical force distribution table		

Portion of Weight effective Distribution Lateral Minimum Height Vertical tributary to seismic exponent force diaphragm from base distributio the induced at Level weight related to force at to Level i n factor, diaphragm assigned building Level i Level i (ft), hx C_{vx} at Level i to Level i period, k (kips), Fx (kips), F_{px} (kips), w_{px} (kips), wx 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; 74.0 12.6 26.1 **14.0** 26.1 3 30.7; 15.0; 1.00; 0.179; 3.8 8 15.0 3.8 8

Tekla Tedds	Project Yaroslavsky	Residence	Job Ref. 8119				
Fast + Epp 323 Dean Street, Suite #3	Section				Sheet no./rev	<i>I</i> .	
Brooklyn, NY 11217	1.6 Seismic	Loads - OMF (Y-	Direction)		1		
	Calc. by BW	Date 2/23/2021	Chk'd by	Date	App'd by	Date	
SEISMIC FORCES (ASCE 7-16)					.		
Site parameters					redds ca	Iculation version	
Site class		D. utilizina e	xception per 11	1.4.8(2)			
Mapped acceleration parameters	(Section 11.4.)	-					
at short period		, Ss = 1.415					
at 1 sec period		S ₁ = 0.492					
Site coefficientat short period (Ta	able 11.4-1)	Fa = 1.000					
at 1 sec period (Table 11.4-2)		F _v = 1.808					
Spectral response acceleration	n parameters						
at short period (Eq. 11.4-1)		Sмs = Fa * Ss	s = 1.415				
at 1 sec period (Eq. 11.4-2)		Sm1 = Fv * S1	= 0.890				
Design spectral acceleration p	arameters (Se	ct 11.4.4)					
at short period (Eq. 11.4-3)		$S_{DS} = 2/3$ *	Sms = 0.943				
at 1 sec period (Eq. 11.4-4)		S _{D1} = 2 / 3 *	Sm1 = 0.593				
Seismic design category							
Occupancy category (Table 1-1)		II					
Seismic design category based of	on short period (esponse acceler	ation (Table 11	6-1)			
Seisinic design category based t	n short period i	D		.0-1)			
Seismic design category based of	on 1 sec period	response accelei	ation (Table 1	1.6-2)			
	-	D	-				
		D					
Seismic design category							
Seismic design category Approximate fundamental peri	od						
		hn = 30.69 ft					
Approximate fundamental peri		hn = 30.69 ft					
Approximate fundamental peri Height above base to highest lev		h₅ = 30.69 ft All other sys					
Approximate fundamental peri Height above base to highest lev From Table 12.8-2:							
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type		All other sys					
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type Building period parameter Ct Building period parameter x	el of building	All other sys Ct = 0.02 x = 0.75	tems	= 0.261 sec			
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type Building period parameter Ct Building period parameter x Approximate fundamental period	el of building (Eq 12.8-7)	All other sys $C_t = 0.02$ x = 0.75 $T_a = C_t * (h_n)$	tems × * 1sec / (1ft) ^{x,}	= 0.261 sec			
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type Building period parameter Ct Building period parameter x Approximate fundamental period Building fundamental period (Sec	el of building (Eq 12.8-7)	All other sys $C_t = 0.02$ x = 0.75 $T_a = C_t * (h_n)$ $T = T_a = 0.26$	tems × * 1sec / (1ft) ^{x,}	= 0.261 sec			
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type Building period parameter Ct Building period parameter x Approximate fundamental period Building fundamental period (Sec Long-period transition period	el of building (Eq 12.8-7)	All other sys $C_t = 0.02$ x = 0.75 $T_a = C_t * (h_n)$ $T = T_a = 0.26$ $T_L = 6 \sec c$	tems × * 1sec / (1ft) ^{x,}				
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type Building period parameter Ct Building period parameter x Approximate fundamental period Building fundamental period (Sec Long-period transition period Limiting period	el of building (Eq 12.8-7)	All other sys $C_t = 0.02$ x = 0.75 $T_a = C_t * (h_n)$ $T = T_a = 0.26$ $T_L = 6 \sec c$	tems ^{x *} 1sec / (1ft) ^{x,} 51 sec				
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type Building period parameter Ct Building period parameter x Approximate fundamental period Building fundamental period (Sec Long-period transition period Limiting period Seismic response coefficient	el of building (Eq 12.8-7) ct 12.8.2)	All other sys $C_t = 0.02$ x = 0.75 $T_a = C_t * (h_n)$ $T = T_a = 0.26$ $T_L = 6 \sec$ $T_S = S_{D1} / S_{C}$	tems * * 1sec / (1ft)*: 51 sec s * 1 sec = 0.6	29 sec	EMS		
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type Building period parameter Ct Building period parameter x Approximate fundamental period Building fundamental period (Sec Long-period transition period Limiting period	el of building (Eq 12.8-7) ct 12.8.2)	All other sys $C_t = 0.02$ x = 0.75 $T_a = C_t * (h_n)$ $T = T_a = 0.26$ $T_L = 6 \sec$ $T_S = S_{D1} / S_{D1}$ C_MOMENT	tems * * 1sec / (1ft) ^{x,} 5 1 sec s * 1 sec = 0.6 RESISTING_	29 sec _FRAME_SYST	EMS		
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type Building period parameter Ct Building period parameter x Approximate fundamental period Building fundamental period Building fundamental period Limiting period Seismic response coefficient Seismic force-resisting system (T	el of building (Eq 12.8-7) t 12.8.2)	All other sys $C_t = 0.02$ x = 0.75 $T_a = C_t * (h_n)$ $T = T_a = 0.26$ $T_L = 6 \sec$ $T_S = S_{D1} / S_{D1}$ C_MOMENT	tems * * 1sec / (1ft)*: 51 sec s * 1 sec = 0.6	29 sec _FRAME_SYST	EMS		
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type Building period parameter Ct Building period parameter x Approximate fundamental period Building fundamental period (Sec Long-period transition period Limiting period Seismic response coefficient	el of building (Eq 12.8-7) ct 12.8.2) - able 12.2-1)	All other sys $C_t = 0.02$ x = 0.75 $T_a = C_t * (h_n)$ $T = T_a = 0.26$ $T_L = 6 \sec$ $T_S = S_{D1} / S_E$ C_MOMENT 4. Steel ordin	tems * * 1sec / (1ft) ^{x,} 5 1 sec s * 1 sec = 0.6 RESISTING_	29 sec _FRAME_SYST	EMS		
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type Building period parameter Ct Building period parameter x Approximate fundamental period Building fundamental period Long-period transition period Limiting period Seismic response coefficient Seismic force-resisting system (Ta	el of building (Eq 12.8-7) ct 12.8.2) Fable 12.2-1) able 12.2-1) e 1.5-2)	All other sys $C_t = 0.02$ x = 0.75 $T_a = C_t * (h_n)$ $T = T_a = 0.26$ $T_L = 6 \sec$ $T_S = S_{D1} / S_{C}$ C_MOMENT 4. Steel ordin R = 3.5	tems * * 1sec / (1ft) ^{x,} 5 1 sec s * 1 sec = 0.6 RESISTING_	29 sec _FRAME_SYST	EMS		
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type Building period parameter Ct Building period parameter x Approximate fundamental period Building fundamental period Building fundamental period Long-period transition period Limiting period Seismic response coefficient Seismic force-resisting system (Ta Response modification factor (Ta	el of building (Eq 12.8-7) ct 12.8.2) Fable 12.2-1) able 12.2-1) e 1.5-2)	All other sys $C_t = 0.02$ x = 0.75 $T_a = C_t * (h_n)$ $T = T_a = 0.26$ $T_L = 6 \sec C$ $T_S = S_{D1} / S_E$ C_MOMENT 4. Steel ordin R = 3.5 $I_e = 1.000$	tems * * 1sec / (1ft) ^{x,} 5 1 sec s * 1 sec = 0.6 RESISTING_	29 sec _FRAME_SYST rames	EMS		
Approximate fundamental peri Height above base to highest lev From Table 12.8-2: Structure type Building period parameter Ct Building period parameter x Approximate fundamental period Building fundamental period Building fundamental period Long-period transition period Limiting period Seismic response coefficient Seismic force-resisting system (T Response modification factor (Ta Seismic importance factor (Table Seismic response coefficient (Se	el of building (Eq 12.8-7) ct 12.8.2) Fable 12.2-1) able 12.2-1) e 1.5-2)	All other sys: $C_t = 0.02$ x = 0.75 $T_a = C_t * (h_n)$ $T = T_a = 0.26$ $T_L = 6 \sec$ $T_S = S_{D1} / S_{C}$ C_MOMENT 4. Steel ordin R = 3.5 $I_e = 1.000$ $C_s_calc = S_{DS}$	tems * * 1sec / (1ft)*: 51 sec s * 1 sec = 0.6 RESISTING_ hary moment fr	29 sec _FRAME_SYST rames			

	Project Yaroslavsky Re	Job Ref. 8119				
323 Dean Street, Suite #3	Section 1.6 Seismic Loa	ads - OMF (Y-Di	rection)		Sheet no./rev. 2	
		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 = 143.0 kips	
Seismic response coefficient	Cs = 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
Vertical distribution of seismic forces (Sect 1	2.8.3)	PROVISIONS
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)	
	$F_{\text{pxmin}} = 0.2 * S_{\text{DS}} * I_{\text{e}} * w_{\text{px}}$	
	Fpxmax = 0.4 * Sps * Ie * Wpx	

Vertical force distribution table

Level	Height from base to Level i (ft), hx	Portion of effective seismic weight assigned to Level i (kips), wx	Distribution exponent related to building period, k	Vertical distributio n 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



20210225 Dormit Sot

		20210225 Permit Set		
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		
31 Kitchen: Flush Beam 2	Passed	1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL		
31 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		

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

C7 Post Transfer

Passed



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

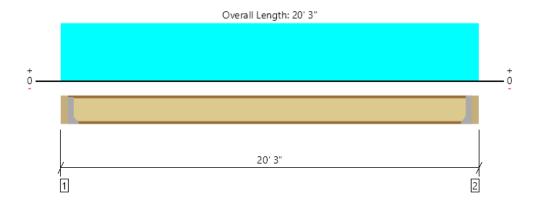
ForteWEB Software Operator
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Job Notes

Weyerhaeuser



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)

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

• Deflection criteria: LL (L/360) and TL (L/240).

• Allowed moment does not reflect the adjustment for the beam stability factor.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Roof Live	Snow	Total	Accessories
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-T	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.

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

Weyerhaeuser Notes

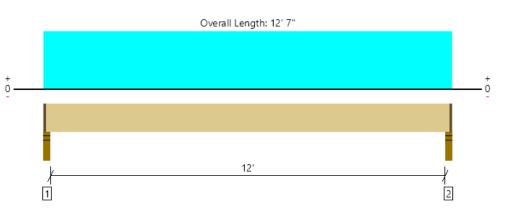
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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
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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)

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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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.

			Dead	Floor Live	Snow	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	(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

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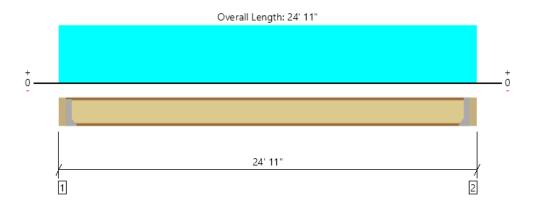
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
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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)

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

• Deflection criteria: LL (L/360) and TL (L/240).

• Allowed moment does not reflect the adjustment for the beam stability factor.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Roof Live	Snow	Total	Accessories
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					
TTI jejete ava anly analyzed using Maximum Alleurable byzeing celutions						

•TJI joists are only analyzed using Maximum Allowable bracing solutions.

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-1	Гie					
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.

			Dead	Roof Live	Snow	
Vertical Load	Location (Side)	Spacing	(0.90)	(non-snow: 1.25)	(1.15)	Comments
1 - Uniform (PSF)	0 to 24' 11"	16"	15.0	20.0	30.0	Default Load

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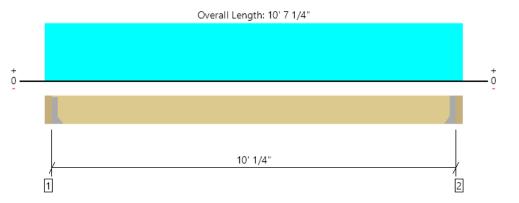
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Garage Roof, B13 Garage Roof: Edge Beam (LVL) 2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL



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)

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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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 ellevente le maine internet en enelle el control de la control d							

•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.

			Dead	Floor Live	Snow	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	(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

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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..

	Bearing Length			Loads t	o Supports		
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
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						
TIL joists are only analyzed using	TTI jojsts zro oply zpolyzod using Maximum Allourable brasing solutions						

TJI joists are only analyzed using Maximum Allowable bracing solutions.

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

1 5						
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	
Pofor to manufacturor notos and instructi	one for proper installation and use	of all connectors				

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

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

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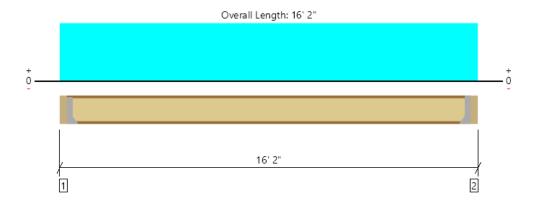
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All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

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..

	Bearing Length			Loads t	o Supports (
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
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.						

• 151 Joists are only analyzed using Maximum Allowable bracing solution

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

1 5									
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				
Pofor to manufacturor notos and instructi	Defor to manufacturer notes and instructions for proper installation and use of all connectors								

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

			Dead	Floor Live	
Vertical Load	Location (Side)	Spacing	(0.90)	(1.00)	Comments
1 - Uniform (PSF)	0 to 16' 2"	16"	30.0	40.0	Default Load

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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 TM Panel (24" Span Rating) that is nailed down.

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

	B	Bearing Length			o Supports		
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					
TTI joints are only applying using Maximum Allowable bracing colutions						

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

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			
Defer to manufacturer notes and instructions for proper installation and use of all connectors								

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

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

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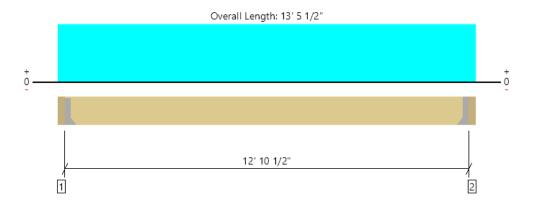
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Main Level, J3 Exterior Deck: Joist (9.5" LVL) 1 piece(s) 1 3/4" x 9 1/2" 2.0E Microllam® LVL @ 16" OC



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-Ibs)	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..

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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					

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.

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

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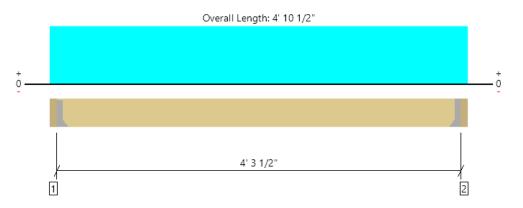
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All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

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

PASSED

• 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.

	В	Bearing Length			oads to Sup			
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
1 - Hanger on 9 1/4" PSL beam	3.50"	Hanger ¹	1.50"	97	195	97	389	See note 1
2 - Hanger on 9 1/4" PSL beam	3.50"	Hanger ¹	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.

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

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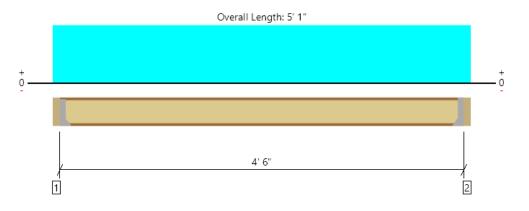
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Main Level, J8 Main Level Shower: Joist (11-7/8" TJI) 1 piece(s) 11 7/8" TJI ® 110 @ 16" OC



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..

	Bearing Length			Loads t	o Supports		
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
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				
TIL joists are only analyzed using Maximum Allowable brasing solutions					

TJI joists are only analyzed using Maximum Allowable bracing solutions.

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	odel Seat Length Top Fasteners Face Fasteners Merr		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		
Defer to manufacturer notes and instructions for proper installation and use of all connectors.							

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

			Dead	Floor Live	
Vertical Load	Location (Side)	Spacing	(0.90)	(1.00)	Comments
1 - Uniform (PSF)	0 to 5' 1"	16"	30.0	40.0	Default Load

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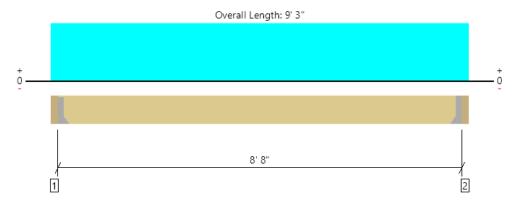
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Main Level, B1 Kitchen: Flush Beam 1 1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL



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)
Design Results	Actual e Eocation	Anowed	Result	LDI	Edda. combination (Fattern)
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.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
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 brasing intervale based on applied load						

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.

			Dead	Floor Live	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(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

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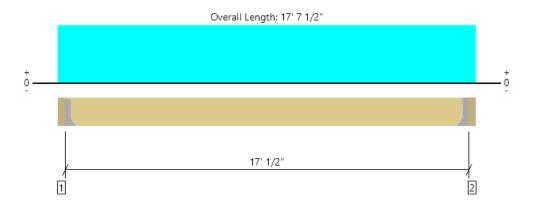
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Main Level, B1 Kitchen: Flush Beam 2 1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL



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)

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.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
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 brasing intervals based on applied load						

Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-I	Ie		
Support	Model	Seat Length	Тој

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.

			Dead	Floor Live	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(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

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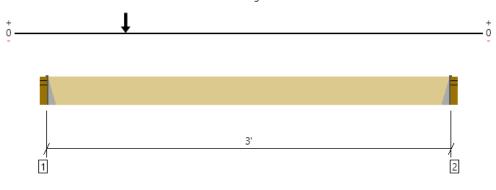


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Main Level, B1 Kitchen: Flush Beam 3 1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL

Overall Length: 3' 7"



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.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
1 - Hanger on SPF studWall	3.50"	Hanger ¹	1.50"	2152	2453	4605	See note 1
2 - Hanger on SPF studWall	3.50"	Hanger ¹	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 elleviselle longetine internele legend en genelied leged					

Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

1 5						
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	
2 - Top Mount Hanger		3.50"	6-16d	16-16d	10-16d	

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

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

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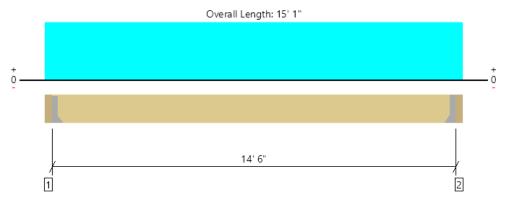


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PASSED



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)

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.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
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 burster internal based on annitable d					

Maximum allowable bracing intervals based on applied load.

Connector: S	Simpson	Strong-Tie
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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.

			Dead	Floor Live	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(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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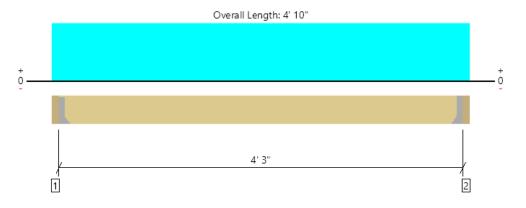


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PASSED



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)

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.

	Bearing Length			Loads to Supports (Ibs)			
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
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 ellevente la serie internet la serie de serie d						

Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
MGU5.50-SDS H=15.938	4.50"	N/A	24-SDS25212	16-SDS25212	
MGU5.50-SDS H=15.938	4.50"	N/A	24-SDS25212	16-SDS25212	
	MGU5.50-SDS H=15.938	MGU5.50-SDS H=15.938 4.50"	MGU5.50-SDS H=15.938 4.50" N/A	MGU5.50-SDS H=15.938 4.50" N/A 24-SDS25212	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.

			Dead	Floor Live	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(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

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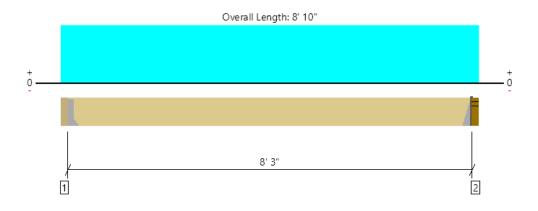


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PASSED



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.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
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						
-Mavimum allowable brasing inter	Maximum allowable brasing intervale based on applied load						

Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-	-Tie
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Support	Model	Seat Length Top Fasteners Face Fasten		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.

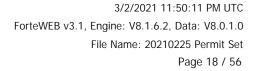
			Dead	Floor Live	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	Comments
0 - Self Weight (PLF)	3 1/2" to 8' 6 1/2"	N/A	26.3		
1 - Uniform (PSF)	0 to 8' 10" (Front)	5' 6"	30.0	60.0	
2 - Uniform (PSF)	0 to 8' 10" (Front)	5' 8"	30.0	40.0	

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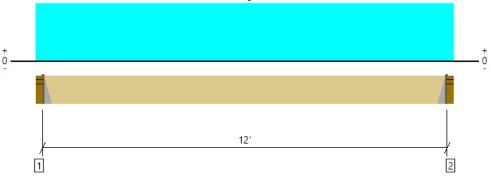
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Main Level, B1 Main Level: Transfer Beam 2 (16" PSL) 1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL





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.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
1 - Hanger on SPF studWall	3.50"	Hanger ¹	1.50"	1557	2087	3644	See note 1
2 - Hanger on SPF studWall	3.50"	Hanger ¹	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 busics inter-		Maximum allowable burgins intervals based on analised load						

•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.

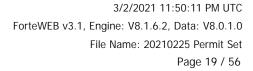
			Dead	Floor Live	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	Comments
0 - Self Weight (PLF)	3 1/2" to 12' 3 1/2"	N/A	26.3		
1 - Uniform (PSF)	0 to 12' 7" (Front)	1' 9"	30.0	60.0	
2 - Uniform (PSF)	0 to 12' 7" (Front)	5' 8"	30.0	40.0	

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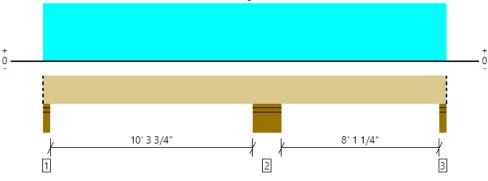
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All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

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)

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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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					

num allowable bracing intervals based on applied load

			Dead	Floor Live	Snow	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	(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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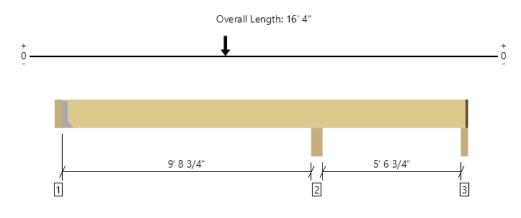




Main Level, B5 Main Level: Wall Transfer Beam 1 1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL

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.

	Bearing Length			Loads to Supports (Ibs)			
Supports	Total	Available	Required	Dead	Seismic	Total	Accessories
1 - Hanger on 16" SPF beam	3.50"	Hanger ¹	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 alloughle bracing intervals based on applied lead					

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.

			Dead	Seismic	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 16' 2 3/4"	N/A	26.3		
1 - Point (Ib)	6' 7 1/2" (Front)	N/A	-	25512	

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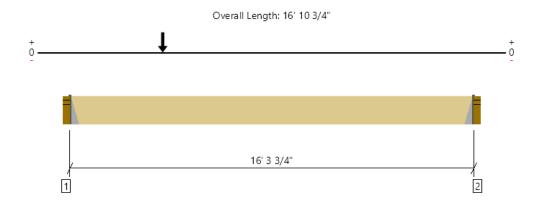
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Main Level, B5 Main Level: Wall Transfer Beam 2 1 piece(s) 5 1/4" x 16" 2.2E Parallam® PSL

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

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



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)

• Deflection criteria: LL (L/480) and TL (L/240).

• Allowed moment does not reflect the adjustment for the beam stability factor.

	Bearing Length			Loads t	o Supports (
Supports	Total	Available	Required	Dead	Seismic	Total	Accessories
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					

laximum 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

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

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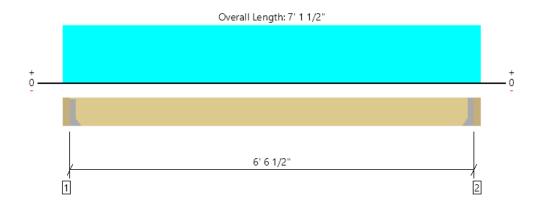
System : Floor Member Type : Flush Beam Building Use : Residential Building Code : IBC 2018 Design Methodology : ASD

SEISMIC CASE W/ **OVERSTRENGTH IS NOT** APPLICABLE FOR SERVICEABILITY DEFLECTION



Main Level, B6 Exterior Deck: Flush Beam 1 piece(s) 3 1/2" x 9 1/2" 2.2E Parallam® PSL

PASSED



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)

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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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 brasing intervals based on applied land					

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.

			Dead	Floor Live	Snow	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	(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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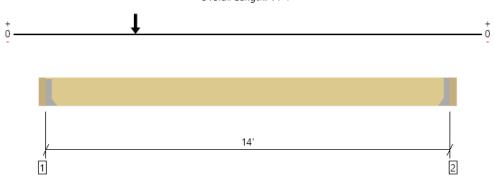
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Main Level, B6 Exterior Deck: Flush Beam (East) 1 piece(s) 3 1/2" x 9 1/2" 2.2E Parallam® PSL

PASSED





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)

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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
1 - Hanger on 9 1/2" SPF beam	3.50"	Hanger ¹	1.50"	589	1031	516	2136	See note 1
2 - Hanger on 9 1/2" SPF beam	3.50"	Hanger ¹	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 elleviselle binerine internele based as explicit land					

•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.

			Dead	Floor Live	Snow	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	(1.15)	Comments
0 - Self Weight (PLF)	3 1/2" to 14' 3 1/2"	N/A	10.4			
1 - Point (Ib)	3' 4 3/4" (Front)	N/A	663	1325	663	Default Load

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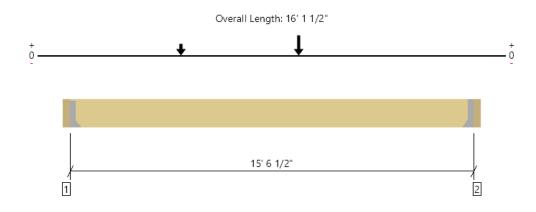
FAILED

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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Seismic	Total	Accessories
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

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

• 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					

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 (Ib)	9' 1 1/4" (Front)	N/A	-	-	5822	Default Load
2 - Point (Ib)	4' 6 3/4" (Front)	N/A	900	1048	-	

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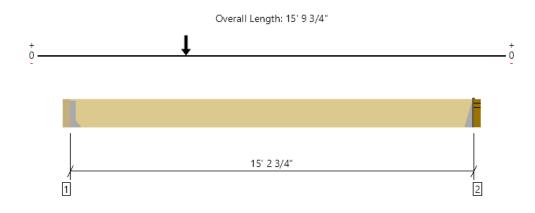
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Main Level, B12 Family Room: Wall Transfer Beam 2 1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL

An excessive uplift of -5443 lbs at support located at 3 1/2" failed this product.

SUPPORTS HAVE SUFFICIENT UPLIFT CAPACITY An excessive uplift of -2131 lbs at support located at 15' 6 1/4" failed this product.



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)

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.

	Bearing Length			Loads to Supports (Ibs)			
Supports	Total	Available	Required	Dead	Seismic	Total	Accessories
1 - Hanger on 14" SPF beam	3.50"	Hanger ¹	1.74"	175	7926/-7926	8101/- 7926	See note 1
2 - Hanger on SPF studWall	3.50"	Hanger ¹	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 interv	als based on applied load	·

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

			Dead	Seismic	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(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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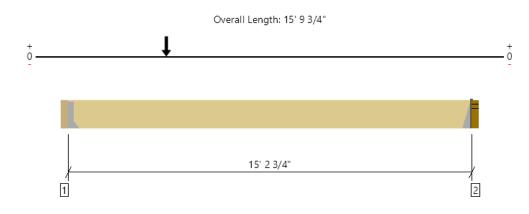
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Main Level, B18 Family Room: Wall Transfer Beam 1 1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL

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

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



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)

• Deflection criteria: LL (L/480) and TL (L/240).

• Allowed moment does not reflect the adjustment for the beam stability factor.

	Bearing Length			Loads t	o Supports (
Supports	Total	Available	Required	Dead	Seismic	Total	Accessories
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 inten	als based on applied load	

kimum 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.

			Dead	Seismic	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(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	

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

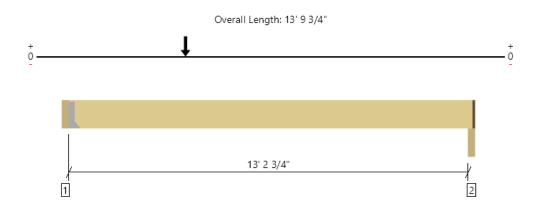
SEISMIC CASE W/ **OVERSTRENGTH IS NOT** APPLICABLE FOR SERVICEABILITY DEFLECTION



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)

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.

	Bearing Length			Loads t	o Supports		
Supports	Total	Available	Required	Dead	Seismic	Total	Accessories
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 ellevende har include hered as environment						

Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie									
Support Model Sea			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				
	6 1 1 11 11 1	C 11 1							

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

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

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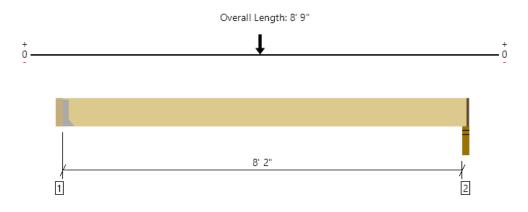
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Main Level, B19 Family Room: Transfer Beam 4 1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL

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.

	Bearing Length			Loads t	to Supports		
Supports	Total	Available	Required	Dead	Seismic	Total	Accessories
1 - Hanger on 14" SPF beam	3.50"	Hanger1	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					
Manimum allowable burging intervals beard on any lind land						

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.

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

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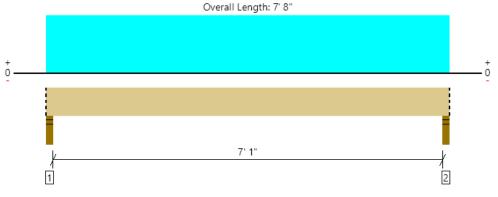
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Main Level, B20 Living Room: Drop Beam 1 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL



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 (Ibs)	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)

System : Floor Member Type : Drop 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.

	Bearing Length			Loads t	o Supports (
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories			
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 load	Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.									

Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 8" o/c	
Bottom Edge (Lu)	7' 8" o/c	

•Maximum allowable bracing intervals based on applied load.

			Dead	Floor Live	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(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

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Main Level, C7 Post Transfer 1 piece(s) 5 1/4" x 7" 1.8E Parallam® PSL

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	Туре		Material	N	
Base	Plate		Parallam® PSL] ^B	
Max Unbraced Length		Comments			
Full Member Length			No bracing assumed.		

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

Drawing is Conceptual

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

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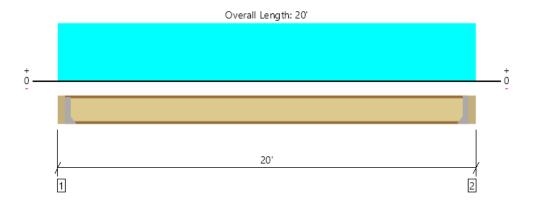
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Upper Level, J5 Upper Level: Joist (14" TJI) 1 piece(s) 14" TJI ® 360 @ 12" OC



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..

	B	Bearing Length			o Supports		
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
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			
aTTI jojete are only analyzed using Maximum Allowable bracing colutions				

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			
Defer to manufacturer potes and instructi	 Defer to manufacturer notes and instructions for proper installation and use of all connectors. 							

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

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

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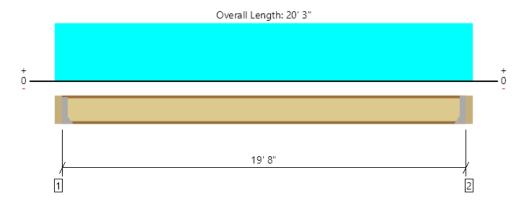
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Upper Level, J6 Upper Deck: Joist - Long (11-7/8" TJI) 1 piece(s) 11 7/8" TJI ® 560 @ 12" OC



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

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		

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..

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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						
TI Jointe are only analyzed using Maximum Allowable bracing colutions							

•TJI joists are only analyzed using Maximum Allowable bracing solutions.

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

nber Fasteners Accessories	Face Fasteners	Top Fasteners	Seat Length	Model	Support
P-Strong-Grip	12-10dx1.5	N/A	2.00"	IUS3.56/11.88	1 - Face Mount Hanger
2-Strong-Grip	12-10dx1.5	N/A	2.00"	IUS3.56/11.88	2 - Face Mount Hanger
5			2.00"	IUS3.56/11.88	Jan Start Start

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

			Dead	Floor Live	Snow	
Vertical Load	Location (Side)	Spacing	(0.90)	(1.00)	(1.15)	Comments
1 - Uniform (PSF)	0 to 20' 3"	12"	30.0	60.0	30.0	Default Load

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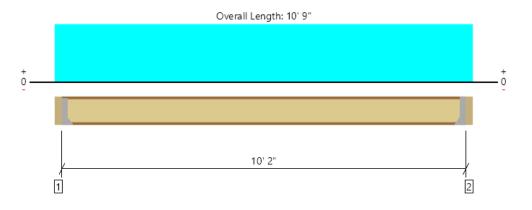
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Upper Level, J8 Upper Deck: Joist - Short (11-7/8" TJI) 1 piece(s) 11 7/8" TJI ® 110 @ 16" OC



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

Accessories

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 EdgeTM 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..

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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						
TTI jojete are anty analyzed using Maximum Alloughle brasing colutions							

•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 1 - Face Mount Hanger IUS1.81/11.88 2.00" N/A 10-10dx1.5 2-Strong-Grip IUS1.81/11.88 2.00" 10-10dx1.5 2 - Face Mount Hanger N/A 2-Strong-Grip

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

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

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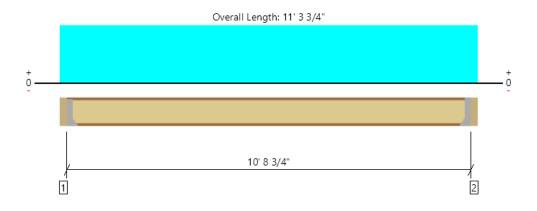
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Upper Level, J8 Stair Roof: Joist (11-7/8" TJI) 1 piece(s) 11 7/8" TJI ® 110 @ 16" OC



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

Accessories

2-Strong-Grip

• 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..

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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 1 - Face Mount Hanger IUS1.81/11.88 2.00" N/A 10-10dx1.5 2-Strong-Grip IUS1.81/11.88 10-10dx1.5 2 - Face Mount Hanger 2.00" N/A

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

			Dead	Floor Live	Snow	
Vertical Load	Location (Side)	Spacing	(0.90)	(1.00)	(1.15)	Comments
1 - Uniform (PSF)	0 to 11' 3 3/4"	16"	15.0	20.0	30.0	Default Load

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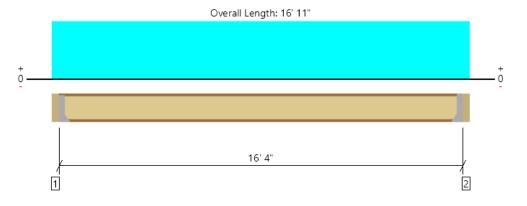
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Upper Level, J9 Upper Deck: Joist - Med (11-7/8" TJI) 1 piece(s) 11 7/8" TJI ® 230 @ 12" OC



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..

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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				
aTTI jointe are only applying during Maximum Allowable bracing colutions					

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				
Defer to menufacturer notes and instructions for proper installation and use of all connectors									

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

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

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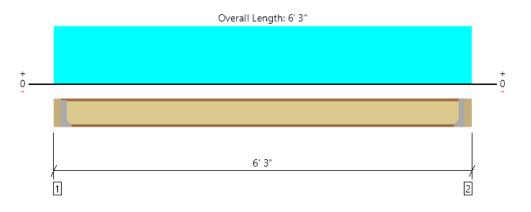
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Upper Level, J10 Upper Level Shower: Joist (9-1/2" TJI) 1 piece(s) 9 1/2" TJI ® 110 @ 16" OC



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..

	Bearing Length			Loads t	o Supports		
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
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				
aTTI joints are only applying using Maximum Allowable bracing colutions					

TJI joists are only analyzed using Maximum Allowable bracing solutions.

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

1 5		_							
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				
Defer to manufacturer notes and instructions for proper installation and use of all connectors									

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

			Dead	Floor Live	
Vertical Load	Location (Side)	Spacing	(0.90)	(1.00)	Comments
1 - Uniform (PSF)	0 to 6' 3"	16"	30.0	40.0	Default Load

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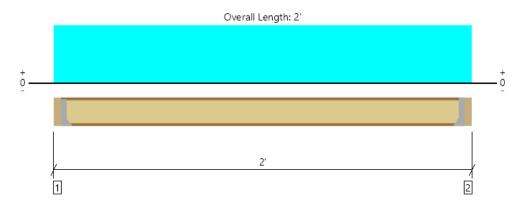
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Upper Level, J11 Upper Deck: Joist (11-7/8" TJI) 1 piece(s) 11 7/8" TJI ® 110 @ 24" OC



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..

	В	Bearing Length		Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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			
TIL joists are only analyzed using Maximum Allowable bracing solutions				

•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.

			Dead	Floor Live	Snow	
Vertical Load	Location (Side)	Spacing	(0.90)	(1.00)	(1.15)	Comments
1 - Uniform (PSF)	0 to 2'	24"	30.0	20.0	30.0	Default Load

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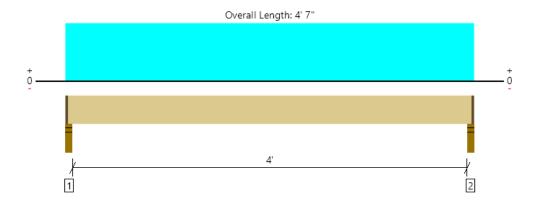
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Upper Level, B4 Upper Level Shower: Short Flush Beam (14" PSL) 1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL



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)

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.

Total Available Required Dead Floor Liv		
Total Available Required Dead Thorein	e Total	Accessories
3.50" 2.25" 1.50" 715 909	1624	1 1/4" Rim Board
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.

			Dead	Floor Live	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(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

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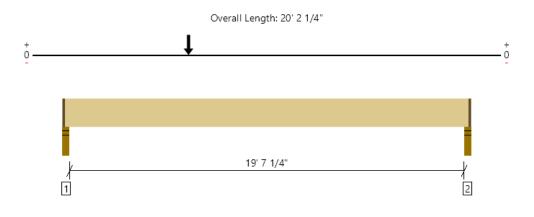
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PASSED

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)

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.

	Bearing Length			Loads t	o Supports (
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
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.

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

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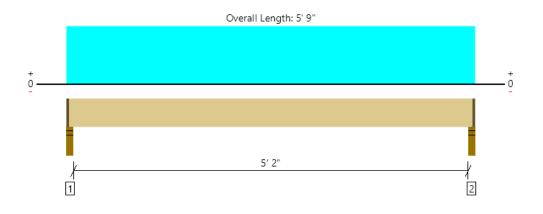
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Upper Level, B4 Upper Level: Flush Beam (14" PSL) 1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL



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)

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.

	Bearing Length			Loads t	o Supports (
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories					
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					
· Pim Poard is assumed to carry all loads applie	d directly abo	wo it hypacci	Pim Roard is assumed to carry all loads applied directly above it, hypassing the member being designed									

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.

			Dead	Floor Live	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(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

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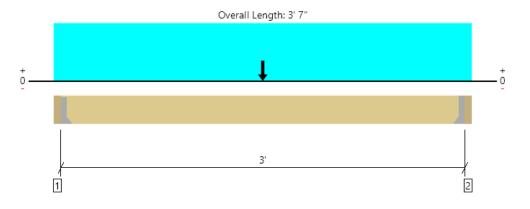
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Upper Level, B4 Upper Level: Transfer Beam 4 (14" PSL) 1 piece(s) 3 1/2" x 14" 2.2E Parallam® PSL



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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Seismic	Total	Accessories
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 Faste		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				
Defor to manufacturar notae and instructi	one for proper installation and use	of all connectors							

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

			Dead	Floor Live	Seismic	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	(1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 3' 3 1/2"	N/A	15.3			
1 - Point (Ib)	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	-	

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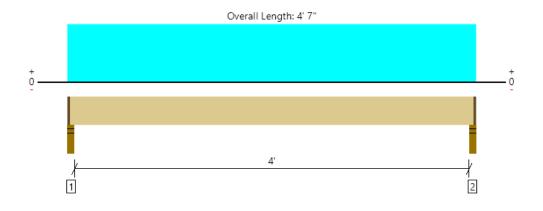
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Upper Level, B7 Upper Level: Typical Header Beam (2-2x10) 2 piece(s) 2 x 10 Spruce-Pine-Fir No. 1 / No. 2



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.

	Bearing Length			Loads to S (Ibs		
Supports	Total	Available	Required	Dead	Total	Accessories
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.

		Tuile at a mark of the	Dead	
Vertical Loads	Location (Side)	Tributary Width	(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

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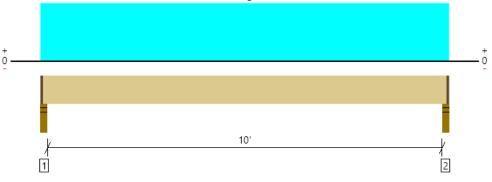
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Upper Level, B12 Upper Level: Flush Beam (14" PSL) 1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL





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)

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.

	Bearing Length			Loads t	to Supports (
Supports	Total	Available	Required	Dead	Floor Live	Total	Accessories
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	-	

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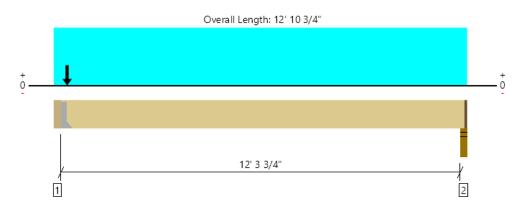
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Upper Level, B12 Upper Level: Transfer Beam (14" PSL) 1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL

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.

	Bearing Length			Loads to Supports (lbs)					
Supports	Total	Available	Required	Dead	Floor Live	Snow	Seismic	Total	Accessories
1 - Hanger on 14" SPF beam	3.50"	Hanger ¹	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.					

racing 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.

			Dead	Floor Live	Snow	Seismic	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	(1.15)	(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 (Ib)	5 3/4" (Front)	N/A	-	-	-	5630	

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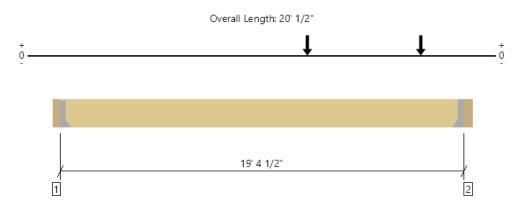
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Upper Level, B12 Upper Level: Transfer Beam 2 (14" PSL) 1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL

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)

Deflection criteria: LL (L/360) and TL (L/240).

• Allowed moment does not reflect the adjustment for the beam stability factor.

	Bearing Length			Loads t	o Supports (
Supports	Total	Available	Required	Dead	Seismic	Total	Accessories
1 - Hanger on 14" SPF beam	3.50"	Hanger ¹	1.50"	223	2835/-2835	3058/- 2835	See note 1
2 - Hanger on 14" SPF beam	4.50"	Hanger ¹	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.					

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.

			Dead	Seismic	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.60)	Comments
0 - Self Weight (PLF)	3 1/2" to 19' 8"	N/A	23.0		
1 - Point (Ib)	12' 2 1/2" (Front)	N/A	-	5781	Default Load
2 - Point (Ib)	17' 7 1/2" (Front)	N/A	-	5781	Default Load

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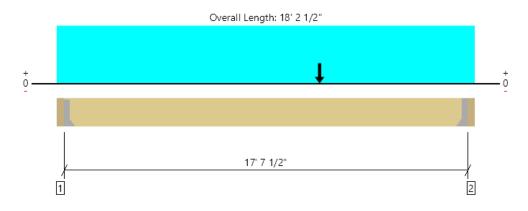
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Upper Level, B12 Upper Level: Transfer Beam 3 (14" PSL) 1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL

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)

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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Seismic	Total	Accessories
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					

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 (Ib)	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	-	

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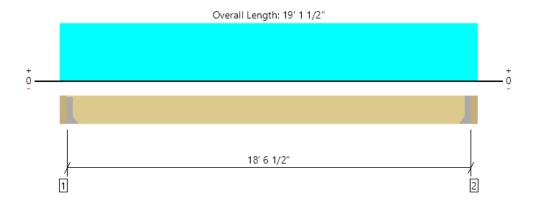
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Upper Level, B13 Upper Deck: Edge Beam (11-7/8" LVL) 2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL



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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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 alloughle brasing intervals based on applied load					

Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Ti

Model	el Seat Length Top Fasteners		Face Fasteners	Member Fasteners	Accessories				
LUS410	2.00"	N/A	8-10dx1.5	6-10d					
LUS410	2.00"	N/A	8-10dx1.5	6-10d					
	Model LUS410	Model Seat Length LUS410 2.00"	Model Seat Length Top Fasteners LUS410 2.00" N/A	Model Seat Length Top Fasteners Face Fasteners LUS410 2.00" N/A 8-10dx1.5	Model Seat Length Top Fasteners Face Fasteners Member Fasteners LUS410 2.00" N/A 8-10dx1.5 6-10d				

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

			Dead	Floor Live	Snow	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	(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

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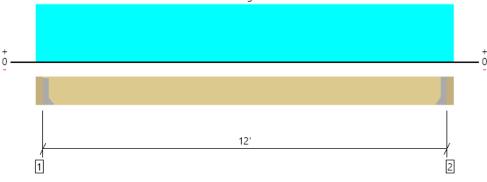


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Upper Level, B13 Upper Deck: Flush Beam (11-7/8" LVL) 2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL





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-Ibs)	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)

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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
1 - Hanger on 11 7/8" SPF beam	3.50"	Hanger ¹	1.50"	1579	1261	1506	4346	See note 1
2 - Hanger on 11 7/8" SPF beam	3.50"	Hanger ¹	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 elleviselle binesine internale based on excited land							

•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.

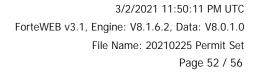
		Dead	Floor Live	Snow	
Location (Side)	Tributary Width	(0.90)	(1.00)	(1.15)	Comments
3 1/2" to 12' 3 1/2"	N/A	12.1			
0 to 12' 7" (Front)	6' 11 1/2"	30.0	20.0	30.0	
0 to 12' 7" (Front)	1' 1/4"	30.0	60.0	30.0	
	3 1/2" to 12' 3 1/2" 0 to 12' 7" (Front)	3 1/2" to 12' 3 1/2" N/A 0 to 12' 7" (Front) 6' 11 1/2"	Location (Side) Tributary Width (0.90) 3 1/2" to 12' 3 1/2" N/A 12.1 0 to 12' 7" (Front) 6' 11 1/2" 30.0	Location (Side) Tributary Width (0.90) (1.00) 3 1/2" to 12' 3 1/2" N/A 12.1 0 to 12' 7" (Front) 6' 11 1/2" 30.0 20.0	Location (Side) Tributary Width (0.90) (1.00) (1.15) 3 1/2" to 12' 3 1/2" N/A 12.1 0 to 12' 7" (Front) 6' 11 1/2" 30.0 20.0 30.0

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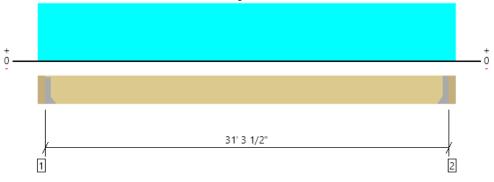
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Upper Level, B13 Upper Deck: Edge Beam 2 (11-7/8" LVL) 2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL

Overall Length: 31' 10 1/2"



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)	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)

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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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 brazing intervale based on applied load				

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.

			Dead	Floor Live	Snow	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	(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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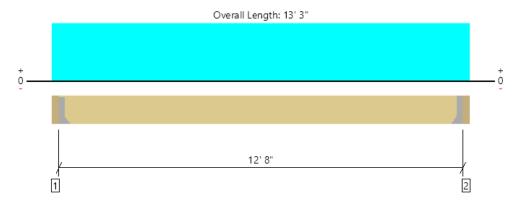
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MEMBER REPORT

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.

	Bearing Length			L	oads to Sup			
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
1 - Hanger on 11 7/8" SPF beam	3.50"	Hanger ¹	1.50"	971	596	894	2461	See note 1
2 - Hanger on 11 7/8" SPF beam	3.50"	Hanger ¹	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 alloughle brasing intended based on applied lead								

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.

			Dead	Floor Live	Snow	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	(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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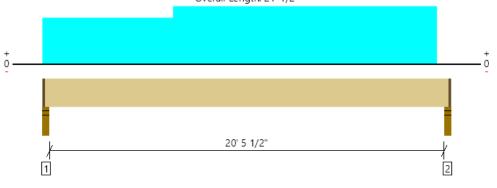
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

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



MEMBER REPORT

Overall Length: 21' 1/2"



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)	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.

	Bearing Length			L	oads to Sup					
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories		
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 applie 	Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.									

• 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.

		Tuile stars (Alistate	Dead	Floor Live	Snow	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	(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	

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

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MEMBER REPORT

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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
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.

			Dead	Floor Live	Snow	
Vertical Loads	Location (Side)	Tributary Width	(0.90)	(1.00)	(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

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2.2 | STEEL FRAMING DESIGN

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

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

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

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:

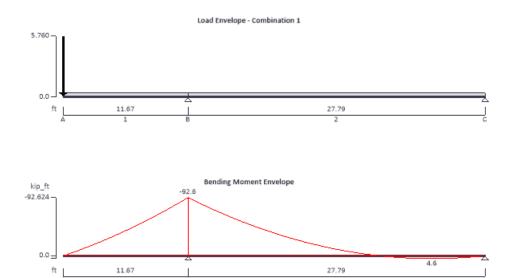
Dead load	DL =	0	plf	0.00	klf
Superimposed dead load	SDL =	57	plf	0.06	klf
Live load	LL =	76.000	plf	0.08	klf
Snow load	SL =	57.000	plf	0.06	klf
REACTIONS:	RDL =	0.00	kips		

Overhang reaction	RSDL =	0.59	kips
	RLL =	0.78	kips
	RSL =	0.59	kips

Tekla Tedds Fast + Epp	Project Yaroslavsky Re	esidence			Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Main Level West Perimeter Beam (B8)			Sheet no./rev. 1		
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date

In accordance with AISC360-16 using the ASD method

Tedds calculation version 3.0.15





Support conditions

Support A

Support B

Support C

Applied loading

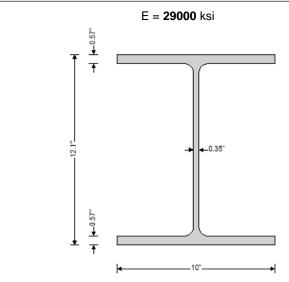
Beam loads

Vertically free Rotationally free Vertically restrained Rotationally free Vertically restrained Rotationally free

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

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Load combinations							
Load combination 1		Support A		Dead *	1.00		
				Live * 1	.00		
				Roof liv	re * 1.00		
				Snow *	1.00		
				Dead *			
				Live * 1	.00		
					'e * 1.00		
				Snow *			
		Support B		Dead *			
				Live * 1			
					re * 1.00		
				Snow * 1.00			
				Dead * 1.00			
				Live * 1.00 Roof live * 1.00			
				Snow * 1.00			
		Support C			Dead * 1.00		
		oupport o		Live * 1.00			
					.00 /e * 1.00		
				Snow *			
Analysis results			· ,	• -	00.011		
Maximum moment		M _{max} = 4.6 k	-		•92.6 kips_ft		
Maximum moment span 1		$M_{s1_max} = 0$	-		= -92.6 kips_ft		
Maximum moment span 2 Maximum shear		Ms2_max = 4.6 V _{max} = 8.5 ki	-		= -92.6 kips_ft 10.1 kips		
Maximum shear span 1		$V_{max} = 0.5 \text{ K}$ $V_{s1_max} = -5.3$			= -10.1 kips		
Maximum shear span 2		Vs1_max = - 3. Vs2_max = 8.5	-		= -10.1 kips = -1.9 kips		
Deflection		νs2_max = 0.3 δmax = 0.6 in	-	vs2_min = δmin = 0	-		
Deflection span 1		$\delta_{s1_max} = 0.0$ III		δ_{s1} min = 0			
Deflection span 2		$\delta s_{max} = 0.0$ $\delta s_{max} = 0$ ir		$\delta_{s2} min =$			
Maximum reaction at support A		$R_{A_{max}} = 0$ II RA_max = 0 ki		Os2_min = RA_min =	-		
Maximum reaction at support P		$RA_{max} = 0 R$ $RB_{max} = 18.0$			= 0 kips = 18.6 kips		
Unfactored dead load reaction		$RB_{Dead} = 7k$	•	T CD_(())) -			
Unfactored live load reaction at	••	RB_Live = 7.5	-				
Unfactored snow load reaction		RB_Snow = 4.1	-				
Maximum reaction at support C		Rc_max = 1.9	•	Rc_min =	= 1.9 kips		
Unfactored dead load reaction		Rc_Dead = 1. 1	-				
Unfactored live load reaction at		Rc_Live = 0.8	-				
Unfactored snow load reaction	at support C	Rc_Snow = 0	kips				
Section details							
Section type		W 12x53 (A	ISC 15th Edn	(v15.0))			
ASTM steel designation		A992					
Steel yield stress		F _y = 50 ksi					
Steel tensile stress		Fu = 65 ksi					

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Safety factors

Modulus of elasticity

Safety factor for tensile yielding	$\Omega_{ty} = 1.67$
Safety factor for tensile rupture	$\Omega_{tr} = 2.00$
Safety factor for compression	$\Omega_c = 1.67$
Safety factor for flexure	$\Omega_{\text{b}} = 1.67$

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	bf / (2 * tf) = 8.70			
Limiting ratio for compact section	λ_{pff} = 0.38 * $\sqrt[4]{\text{E} / \text{F}_{y}}$ = 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

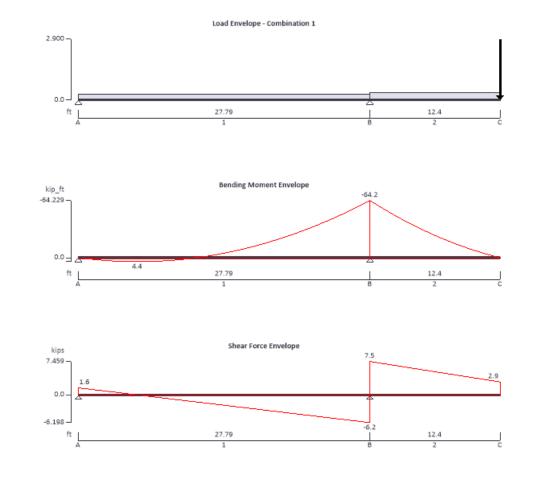
•	
Required shear strength	$V_r = max(abs(V_{max}), abs(V_{min})) = 10.114 \text{ kips}$
Web area	$A_w = d * t_w = 4.174 in^2$
Web plate buckling coefficient	k _v = 5.34
Web shear coefficient - eq G2-3	C _{v1} = 1
Nominal shear strength – eq G6-1	$V_n = 0.6 * F_y * A_w * C_{v1} = 125.235$ kips
Safety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

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Allowable shear strength		$V_c = V_n / \Omega_v$	= 83.490 kips					
		PASS -	Allowable she	ear strength e	exceeds required	shear strength		
Design of members for flexu	re in the maior a	xis at span 1 - (Chapter F					
200.9								
Required flexural strength	· · · · · · · · · · · · · · · · · · ·	-	os(Ms1_max), abs((Ms1_min)) = 92.	. 624 kips_ft			
-		-	-	(Ms1_min)) = 92 .	624 kips_ft			
Required flexural strength	-	Mr = max(at	-		624 kips_ft			
Required flexural strength Yielding - Section F2.1	-	$M_r = max(ab)$ $M_{nyld} = M_p =$	os(M _{s1_max}), abs(33 kips_ft	624 kips_ft			
Required flexural strength Yielding - Section F2.1 Nominal flexural strength for y	-	$M_r = max(ab)$ $M_{nyld} = M_p =$ $M_n = M_{nyld} =$	os(M _{s1_max}), abs(F _y * Z _x = 324.5 8	33 kips_ft	624 kips_ft			
Required flexural strength Yielding - Section F2.1 Nominal flexural strength for yield Nominal flexural strength	-	$M_{r} = max(ab)$ $M_{nyld} = M_{p} =$ $M_{n} = M_{nyld} =$ $M_{c} = M_{n} / \Omega_{t}$	os(M _{s1_max}), abs(Fy * Z _x = 324.58 324.583 kips_ft = 194.361 kips	33 kips_ft t s_ft	624 kips_ft ceeds required fl	exural strength		
Required flexural strength Yielding - Section F2.1 Nominal flexural strength for yield Nominal flexural strength	elding - eq F2-1	$M_{r} = max(ab)$ $M_{nyld} = M_{p} =$ $M_{n} = M_{nyld} =$ $M_{c} = M_{n} / \Omega_{t}$	os(M _{s1_max}), abs(Fy * Z _x = 324.58 324.583 kips_ft = 194.361 kips	33 kips_ft t s_ft		exural strengtl		
Required flexural strength Yielding - Section F2.1 Nominal flexural strength for yield Nominal flexural strength Allowable flexural strength	elding - eq F2-1 cal deflection	$M_{r} = max(ab)$ $M_{nyld} = M_{p} =$ $M_{n} = M_{nyld} =$ $M_{c} = M_{n} / \Omega_{t}$	os(M _{s1_max}), abs(Fy * Z _x = 324.58 324.583 kips_ft = 194.361 kips	33 kips_ft t s_ft		exural strength		
Required flexural strength Yielding - Section F2.1 Nominal flexural strength for yi Nominal flexural strength Allowable flexural strength Design of members for verti	elding - eq F2-1 cal deflection	$M_{r} = max(at)$ $M_{nyld} = M_{p} =$ $M_{n} = M_{nyld} =$ $M_{c} = M_{n} / \Omega_{t}$ $PASS - Allo$	os(M _{s1_max}), abs(Fy * Z _x = 324.58 324.583 kips_ft = 194.361 kips	83 kips_ft t s_ft I strength exc		exural strengt		
Required flexural strength Yielding - Section F2.1 Nominal flexural strength for yield Nominal flexural strength Allowable flexural strength Design of members for verting Consider deflection due to live	elding - eq F2-1 cal deflection	$M_r = max(ab)$ $M_{ryld} = M_p =$ $M_n = M_{nyld} =$ $M_c = M_n / \Omega_t$ $PASS - Allo$ $\delta_{lim} = 2 * L_{s1}$	os(M _{s1_max}), abs(Fy * Zx = 324.5 324.583 kips_ft = 194.361 kips owable flexura	33 kips_ft t s_ft I strength exc		exural strengtl		

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In accordance with AISC360-16 using the LRFD method

Tedds calculation version 3.0.15



Support conditions

Support A

Support B

Support C

Applied loading

Beam loads

Vertically restrained Rotationally free Vertically restrained Rotationally free Vertically free Rotationally free

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

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	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date	
Load combinations							
Load combination 1		Support A		Dead *	1.20		
				Live * 1	.60		
				Snow *	0.50		
				Dead *	1.20		
				Live * 1			
				Snow *			
		Support B		Dead *			
				Live * 1			
				Snow *			
				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		
				Roof liv			
				Snow *	1.60		
Analysis results							
Maximum moment		M _{max} = 4.4 k	-		-64.2 kips_ft		
Maximum moment span 1 Maximum moment span 2		Ms1_max = 4.4 Ms2_max = 0 k	-		= -64.2 kips_ft = -64.2 kips_ft		
Maximum shear		Vmax = 7.5 ki	•		6.2 kips		
Maximum shear span 1		$V_{s1_max} = 1.6$	-		= -6.2 kips		
Maximum shear span 2		Vs2_max = 7.5	-		= 2.9 kips		
Deflection		δ _{max} = 0.6 in	-	$\delta_{min} = 0$			
Deflection span 1		δs1_max = 0 ir	1	δ s1_min =	= 0.1 in		
Deflection span 2		$\delta_{s2} \max = 0.6$	in	δ s2_min =	= 0 in		
Maximum reaction at support A	L .	RA_max = 1.6	kips	RA_min =	= 1.6 kips		
Unfactored dead load reaction		RA_Dead = 1.2	2 kips		·		
Unfactored live load reaction at	support A	RA_Live = 0.2	kips				
Unfactored snow load reaction	at support A	RA_Snow = 0.6	3 kips				
Maximum reaction at support B	5	R _{B_max} = 13.	7 kips	RB_min =	= 13.7 kips		
Unfactored dead load reaction		RB_Dead = 4.7	-				
Unfactored live load reaction at		RB_Live = 2.6	-				
Unfactored snow load reaction		RB_Snow = 3.2	-	_			
Maximum reaction at support C		Rc_max = 0 k	ips	Rc_min =	= 0 kips		
Section details							
Section type		-	ISC 15th Edn ((v15.0))			
ASTM steel designation Steel yield stress		A992					
		F _y = 50 ksi					
Steel tensile stress		Fu = 65 ksi					

Fast + Epp	Project Yaroslavsk	y Residence		Job Ref. 8119	
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Brooklyn, NY 11217		Level Deck West Perimeter Beam (B8) 3			
	Calc. by BJW	Date Chk'd by I 3/2/2021	Date	App'd by	Date
	+ + 				
	↓ ▼ Ŧ	↓ 10" ↓			
Resistance factors					
Resistance factor for tensile yie	lding	$\phi_{ty} = 0.90$			
Resistance factor for tensile rup		$\phi_{tr} = 0.75$			
Resistance factor for compress		$\phi_{\rm c} = 0.90$			
Resistance factor for flexure	1011	$\phi_{\rm c} = 0.90$			
		$\psi_{\text{D}} = 0.90$			
Lateral bracing		Span 1 has continuous lateral br Span 2 has continuous lateral br Cantilever tip is unbraced Cantilever support is continuous	acing	and torsional r	estraint
	local buckling	- Section B4.1			
Classification of sections for					
Classification of sections for	-	1 1b (case 10)			
Classification of flanges in fle	-	· ·			
Classification of flanges in fle Width to thickness ratio	exure - Table B4	bf / (2 * tf) = 8.70			
Classification of flanges in fle Width to thickness ratio Limiting ratio for compact section	exure - Table B4	bf / (2 * tf) = 8.70 $\lambda_{\text{pff}} = 0.38 * \sqrt{[E / F_y]} = 9.15$	Compact		
Classification of flanges in fla Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section	exure - Table B4	bf / (2 * tf) = 8.70 $\lambda_{pff} = 0.38 * \sqrt{[E / F_y]} = 9.15$ $\lambda_{rff} = 1.0 * \sqrt{[E / F_y]} = 24.08$	Compact		
Classification of flanges in fla Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact section Classification of web in flexue	exure - Table B4	bf / (2 * tf) = 8.70 $\lambda_{pff} = 0.38 * \sqrt{[E / F_y]} = 9.15$ $\lambda_{rff} = 1.0 * \sqrt{[E / F_y]} = 24.08$ b (case 15)	Compact		
Classification of flanges in fla Width to thickness ratio Limiting ratio for compact sector Limiting ratio for non-compact sector Classification of web in flexue Width to thickness ratio	exure - Table B4 on section re - Table B4.1k	bf / (2 * tf) = 8.70 $\lambda_{pff} = 0.38 * \sqrt{[E / F_y]} = 9.15$ $\lambda_{rff} = 1.0 * \sqrt{[E / F_y]} = 24.08$ b (case 15) (d - 2 * k) / tw = 28.23	Compact		
Classification of flanges in fla Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Classification of web in flexue Width to thickness ratio Limiting ratio for compact section	exure - Table B4 on section re - Table B4.1k	bf / (2 * tf) = 8.70 $\lambda_{pff} = 0.38 * \sqrt{[E / F_y]} = 9.15$ $\lambda_{rff} = 1.0 * \sqrt{[E / F_y]} = 24.08$ b (case 15) (d - 2 * k) / tw = 28.23 $\lambda_{pwf} = 3.76 * \sqrt{[E / F_y]} = 90.55$			
Classification of flanges in fla Width to thickness ratio Limiting ratio for compact sector Limiting ratio for non-compact sector Classification of web in flexue Width to thickness ratio	exure - Table B4 on section re - Table B4.1k	bf / (2 * tf) = 8.70 $\lambda_{pff} = 0.38 * \sqrt{[E / F_y]} = 9.15$ $\lambda_{rff} = 1.0 * \sqrt{[E / F_y]} = 24.08$ b (case 15) (d - 2 * k) / tw = 28.23	Compact		nnact in flow
Classification of flanges in fla Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Classification of web in flexue Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section	exure - Table B4 on section re - Table B4.1k on section	bf / (2 * tf) = 8.70 $\lambda_{pff} = 0.38 * \sqrt{[E / F_y]} = 9.15$ $\lambda_{rff} = 1.0 * \sqrt{[E / F_y]} = 24.08$ b (case 15) (d - 2 * k) / tw = 28.23 $\lambda_{pwf} = 3.76 * \sqrt{[E / F_y]} = 90.55$	Compact		npact in flexi
Classification of flanges in fla Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Classification of web in flexue Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Design of members for shear	exure - Table B4 on section re - Table B4.1k on section	bf / (2 * tf) = 8.70 $\lambda_{pff} = 0.38 * \sqrt{[E / F_y]} = 9.15$ $\lambda_{rff} = 1.0 * \sqrt{[E / F_y]} = 24.08$ b (case 15) (d - 2 * k) / tw = 28.23 $\lambda_{pwf} = 3.76 * \sqrt{[E / F_y]} = 90.55$ $\lambda_{rwf} = 5.70 * \sqrt{[E / F_y]} = 137.27$	Compact		npact in flexu
Classification of flanges in fla Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Classification of web in flexue Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Limiting ratio for non-compact section Required shear strength	exure - Table B4 on section re - Table B4.1k on section	bf / (2 * tf) = 8.70 $\lambda_{pff} = 0.38 * \sqrt{[E / F_y]} = 9.15$ $\lambda_{rff} = 1.0 * \sqrt{[E / F_y]} = 24.08$ b (case 15) (d - 2 * k) / tw = 28.23 $\lambda_{pwf} = 3.76 * \sqrt{[E / F_y]} = 90.55$ $\lambda_{rwf} = 5.70 * \sqrt{[E / F_y]} = 137.27$ Vr = max(abs(Vmax), abs(Vmin)) =	Compact		npact in flexi
Classification of flanges in fla Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Classification of web in flexue Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section	exure - Table B4 on section re - Table B4.1k on section	$b_{f} / (2 * t_{f}) = 8.70$ $\lambda_{pff} = 0.38 * \sqrt{[E / F_{y}]} = 9.15$ $\lambda_{rff} = 1.0 * \sqrt{[E / F_{y}]} = 24.08$ $b (case 15)$ $(d - 2 * k) / t_{w} = 28.23$ $\lambda_{pwf} = 3.76 * \sqrt{[E / F_{y}]} = 90.55$ $\lambda_{rwf} = 5.70 * \sqrt{[E / F_{y}]} = 137.27$ $V_{r} = max(abs(V_{max}), abs(V_{min})) =$ $A_{w} = d * t_{w} = 4.174 \text{ in}^{2}$	Compact		npact in flexi
Classification of flanges in fla Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Classification of web in flexue Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Limiting ratio for non-compa	exure - Table Ba on section re - Table B4.1k on section	$b_{f} / (2 * t_{f}) = 8.70$ $\lambda_{pff} = 0.38 * \sqrt{[E / F_{y}]} = 9.15$ $\lambda_{rff} = 1.0 * \sqrt{[E / F_{y}]} = 24.08$ $(d - 2 * k) / t_{w} = 28.23$ $\lambda_{pwf} = 3.76 * \sqrt{[E / F_{y}]} = 90.55$ $\lambda_{rwf} = 5.70 * \sqrt{[E / F_{y}]} = 137.27$ $V_{r} = max(abs(V_{max}), abs(V_{min})) =$ $A_{w} = d * t_{w} = 4.174 \text{ in}^{2}$ $k_{v} = 5.34$	Compact		npact in flexi
Classification of flanges in fla Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Classification of web in flexue Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Limiting ratio for non-compa	exure - Table B4 on section re - Table B4.1k on section - Chapter G	$b_{f} / (2 * t_{f}) = 8.70$ $\lambda_{pff} = 0.38 * \sqrt{[E / F_{y}]} = 9.15$ $\lambda_{rff} = 1.0 * \sqrt{[E / F_{y}]} = 24.08$ $(d - 2 * k) / t_{w} = 28.23$ $\lambda_{pwf} = 3.76 * \sqrt{[E / F_{y}]} = 90.55$ $\lambda_{rwf} = 5.70 * \sqrt{[E / F_{y}]} = 137.27$ $V_{r} = max(abs(V_{max}), abs(V_{min})) =$ $A_{w} = d * t_{w} = 4.174 \text{ in}^{2}$ $k_{v} = 5.34$ $C_{v1} = 1$	Compact		npact in flexi
Classification of flanges in fla Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Classification of web in flexue Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Limiting ratio for non-compa	exure - Table B4 on section re - Table B4.1k on section - Chapter G	$b_{f} / (2 * t_{f}) = 8.70$ $\lambda_{pff} = 0.38 * \sqrt{[E / F_{y}]} = 9.15$ $\lambda_{rff} = 1.0 * \sqrt{[E / F_{y}]} = 24.08$ $(d - 2 * k) / t_{w} = 28.23$ $\lambda_{pwf} = 3.76 * \sqrt{[E / F_{y}]} = 90.55$ $\lambda_{rwf} = 5.70 * \sqrt{[E / F_{y}]} = 137.27$ $V_{r} = max(abs(V_{max}), abs(V_{min})) =$ $A_{w} = d * t_{w} = 4.174 \text{ in}^{2}$ $k_{v} = 5.34$	Compact		npact in flexi

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Design of members for flexu	ire in the major a		U	ear strength e	xceeds required	l shear stren
Required flexural strength		-	-	(Ms1_min)) = 64.	229 kips ft	
		, ,		(· =	
-		, ,			· -	
Nominal flexural strength for y	ielding - eq F2-1	$M_{nyld} = M_p =$	F _y * Z _x = 324.5	83 kips_ft	. –	
Nominal flexural strength for y Nominal flexural strength	ielding - eq F2-1	$M_{nyid} = M_p =$ $M_n = M_{nyid} =$	F _y * Z _x = 324.5 324.583 kips_f	83 kips_ft t	. –	
Nominal flexural strength for y Nominal flexural strength	ielding - eq F2-1	$M_{nyld} = M_p =$ $M_n = M_{nyld} =$ $M_c = \phi_b * M_n$	Fy * Zx = 324.5 324.583 kips_f = 292.125 kips	83 kips_ft t :_ft		lexural stren
Nominal flexural strength for y Nominal flexural strength Design flexural strength		$M_{nyld} = M_p =$ $M_n = M_{nyld} =$ $M_c = \phi_b * M_n$	Fy * Zx = 324.5 324.583 kips_f = 292.125 kips	83 kips_ft t :_ft	ceeds required f	lexural stren
Yielding - Section F2.1 Nominal flexural strength for y Nominal flexural strength Design flexural strength Design of members for verti Consider deflection due to dea	cal deflection	Mnyld = Mp = Mn = Mnyld = Mc = φb * Mn PASS - I	Fy * Zx = 324.5 324.583 kips_f = 292.125 kips	83 kips_ft t :_ft		lexural stren
Nominal flexural strength for y Nominal flexural strength Design flexural strength Design of members for verti	cal deflection	M _{nyid} = M _P = M _n = M _{nyid} = M _c = φ _b * M _n PASS - I	Fy * Zx = 324.5 324.583 kips_f = 292.125 kips	83 kips_ft t :_ft		lexural stren
Nominal flexural strength for y Nominal flexural strength Design flexural strength Design of members for verti Consider deflection due to dea	cal deflection	$\begin{split} M_{nyld} &= M_{p} = \\ M_{n} &= M_{nyld} = \\ M_{c} &= \phi_{b} * M_{n} \\ PASS - I \\ PASS - I \\ hd snow loads \\ \delta_{lim} &= 2 * L_{s2} \end{split}$	Fy * Zx = 324.5 324.583 kips_f = 292.125 kips Design flexura	83 kips_ft t s_ft I strength exc		lexural strer

PASS - Maximum deflection does not exceed deflection limit

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		

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.
Beam length	L =	23.

ft .135 8.73 ft

SURFACE LOADS:

Dead load	DL =	0	psf		
Superimposed dead load	SDL =	30	psf		
Live load	LL avg =	46.9	psf	Deck = 60	Ball
Snow load	SL =	30	psf	Distance = 16.0	Distar
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		

Deck =	60	Ballast =	20
Distance =	16.0	Distance =	7.75

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:

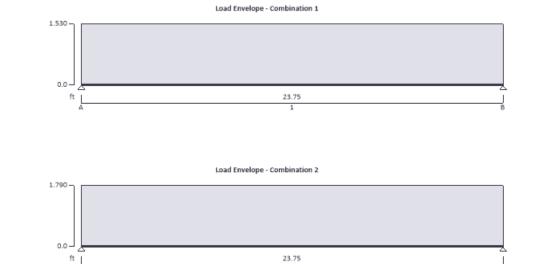
Dead load	DL =	0	plf	0	klf
Superimposed dead load	SDL =	130.156	plf	0.130	klf
Live load	LL =	173.542	plf	0.174	klf
Snow load	SL =	216.927	plf	0.217	klf
REACTIONS:	RDL =	0.00	kips		
Girder reaction	RSDL =	1.54	kips		
	RLL =	2.06	kips		
	RSL =	2.57	kips		

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

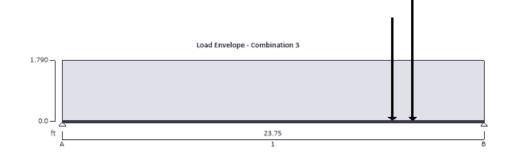
Tekla Tedds Fast + Epp					Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Supper Level Transfer Beam 1 (B9)			Sheet no./rev. 1		
	Calc. by Date Chk'd by Date A BJW 3/2/2021 Chk'd by Date A				App'd by	Date

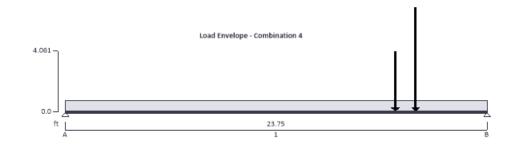
In accordance with AISC360-16 using the ASD method

Tedds calculation version 3.0.15

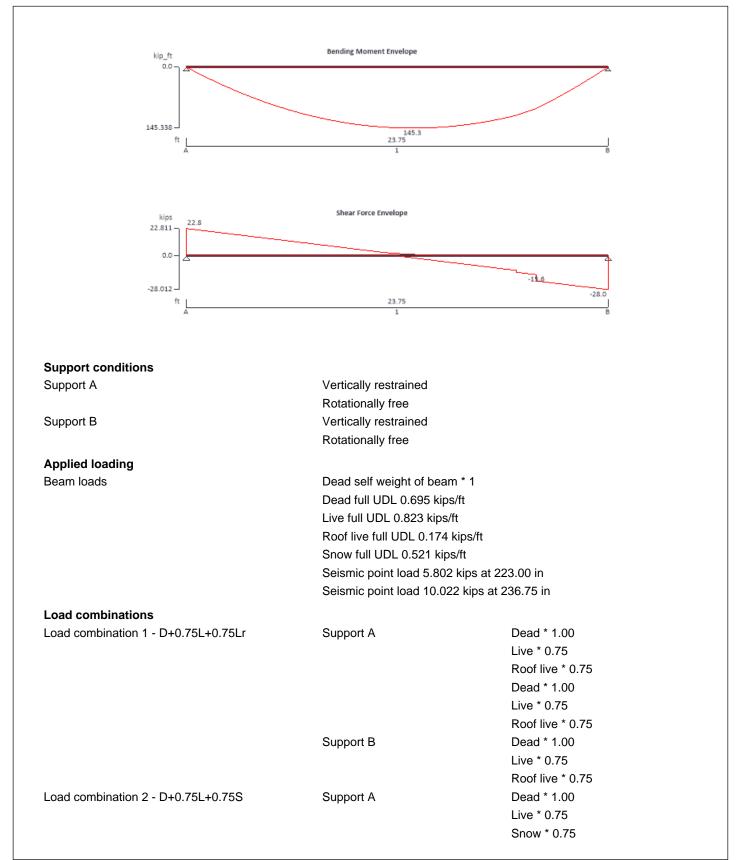


1



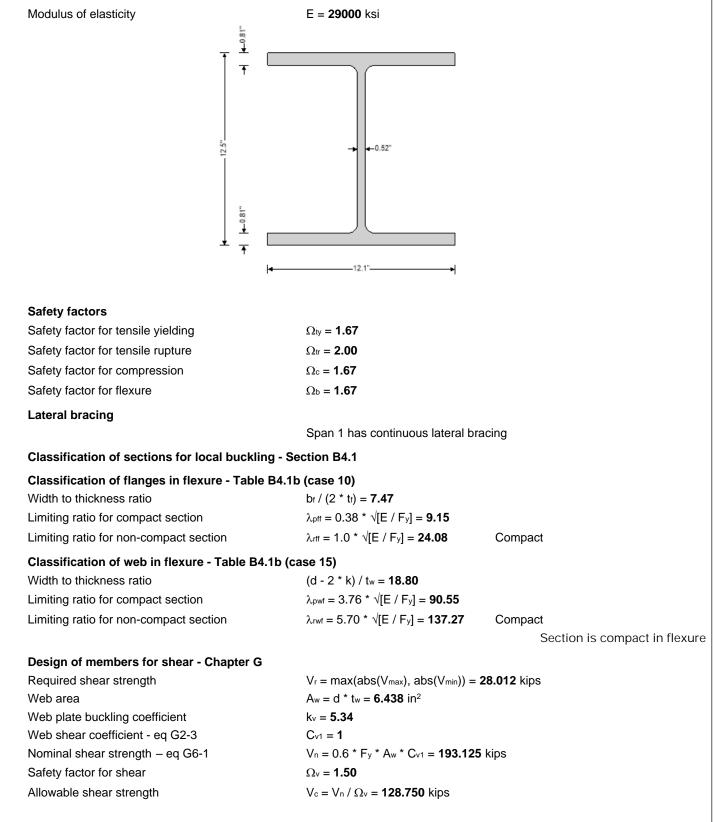


Fast + Epp					Job Ref. 8119	
Brooklyn NY 11217	Section Supper Level Transfer Beam 1 (B9)			Sheet no./rev. 2		
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date



Fast + Epp		Project Yaroslavsky Residence				Job Ref. 8119	
323 Dean Street, Suite #3	Section				Sheet no./rev	,	
Brooklyn, NY 11217		Transfer Beam	1 (B9)		3		
	Calc. by	Date	Chk'd by	Date	App'd by	Date	
	BJW	3/2/2021					
		•	•				
				Dead * Live * (
				Snow *			
		Support B		Dead *			
		Support B		Live * (
				Snow *			
Load combination 3 - D+0.75L+	0.525E+0.75S	Support A		Dead *			
				Live * (
				Snow *			
				Seismi	ic * 0.53		
				Dead *			
				Live * (0.75		
				Snow *	* 0.75		
				Seismi	ic * 0.53		
		Support B		Dead *	1.00		
				Live * (0.75		
				Snow *	* 0.75		
					ic * 0.53		
Load combination 4 - D+0.7E		Support A		Dead *			
					c * 0.70		
				Dead *			
		Querra ent D			ic * 0.70		
		Support B		Dead *			
				Seismi	c * 0.70		
Analysis results		NA 4454	king ft	Ν.Α	Okina ft		
Maximum moment Maximum shear		Mmax = 145.3 Vmax = 22.8	-		0 kips_ft -28 kips		
Deflection		δ _{max} = 22.6	-	$\nabla \min = -$ $\delta \min = 0$	-		
Maximum reaction at support A		$R_{A_{max}} = 0.0 III$			= 11.4 kips		
Unfactored dead load reaction a	at support A	RA_max = 22.	-	TXA_min -	– 11.4 Kips		
Unfactored live load reaction at		RA_Live = 9.8	-				
Unfactored roof live load reaction		R_{A} Roof live = 2	-				
Unfactored snow load reaction a		RA_Snow = 6.2					
Unfactored seismic load reactio	••	RA_Seismic = 3	-				
Maximum reaction at support B		RB_max = 28	-	RB_min =	= 18.2 kips		
Unfactored dead load reaction a	at support B	RB_Dead = 9.3	s kips				
Unfactored live load reaction at	support B	RB_Live = 9.8	kips				
Unfactored roof live load reaction	n at support B	$R_{B_Roof live} = 2$	2.1 kips				
Unfactored snow load reaction a		RB_Snow = 6.2	-				
Unfactored seismic load reactio	n at support B	$R_{B_Seismic} = 1$	2.9 kips				
Section details							
Section type		-	ISC 15th Edn ((v15.0))			
ASTM steel designation		A992					
Steel yield stress		F _y = 50 ksi					
Steel tensile stress		Fu = 65 ksi					

Fast + Epp					Job Ref. 8119	
Brooklyn NY 11217				Sheet no./rev. 4		
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date



PASS - Allowable shear strength exceeds required shear strength

Tekla Tedds Fast + Epp	Project Job Ref. Yaroslavsky Residence 8119					
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Upper Level Transfer Beam 1 (B9)			Sheet no./rev. 5		
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date

Design of members for flexure in the major ax	kis - Chapter F
Required flexural strength	Mr = max(abs(Ms1_max), abs(Ms1_min)) = 145.338 kips_ft
Yielding - Section F2.1	
Nominal flexural strength for yielding - eq F2-1	$M_{nyld} = M_p = F_y * Z_x = 550 \text{ kips_ft}$
Nominal flexural strength	Mn = Mnyld = 550.000 kips_ft
Allowable flexural strength	Mc = Mn / Ωb = 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 an	d snow loads
Lingthe state of a state	S 1 (040 4 400 in

Limiting deflection	δlim = Ls1 / 240 = 1.188 IN
Maximum deflection span 1	$\delta = max(abs(\delta_{max}), abs(\delta_{min})) = 0.767$ in
	DACC Maximum deflection deep not expected deflection limit

PASS - Maximum deflection does not exceed deflection limit

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		

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

ft

ft

NOTES: B10 - UPPER LEVEL DECK

GEOMETRY:

Tributary width	w _T =	7.177
Beam length	L =	30.604

SURFACE LOADS:

Dead load	DL =	0	psf			
Superimposed dead load	SDL =	30	psf			
Live load	LL avg =	50.7	psf	Deck =	60	Ва
Snow load	SL =	30	psf	Distance =	23.5	Dista
LINE LOADS:						
Dead load	DL =	0.0	plf	0.0 kli	ŕ	
Superimposed dead load	SDL =	215.3	plf	0.2 kli	f	
Live load	LL =	364.2	plf	0.4 kli	F	
Snow load	SL =	215.3	plf	0.2 kli	F	
REACTIONS:						
	RDL =	0.00	kips			
Girder reaction	RSDL =	3.29	kips			
	RLL =	5.57	kips			
	RSL =	3.29	kips			

Deck =	60	Ballast =	20
Distance =	23.5	Distance =	7.08

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

NOTES: B10 - HIGH ROOF

GEOMETRY:

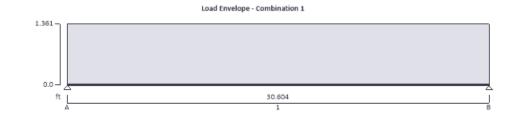
				Trib 1	Trib 2
Tributary width	w _T =	8.713	ft	9.76	7.61
Beam length	L =	30.604	ft	15.67	14.938
SURFACE LOADS:					
SURFACE LUADS.					
Dead load	DL =	0	psf		
Superimposed dead load	SDL =	15	psf		
Live load	LL =	20	psf		
Snow load	SL =	30	psf		
LINE LOADS:					
LINE LOADS.					
Dead load	DL =	0	plf	0	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 To	otal
SDL	0.607	klf
LL	0.713	klf
RL	0.174	klf
SL	0.477	klf

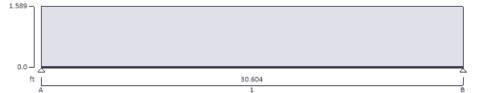
Tekla Tedds Fast + Epp	Project Yaroslavsky Re	esidence			Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Upper Level Tra	ansfer Beam 2 (B10)		Sheet no./rev. 1	
	,	Date 3/2/2021	Chk'd by	Date	App'd by	Date

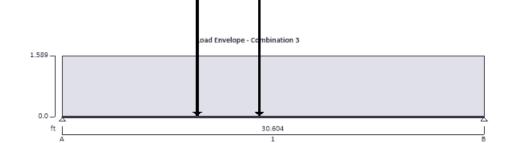
In accordance with AISC360-16 using the ASD method

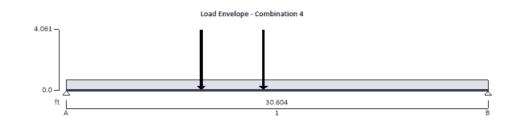
Tedds calculation version 3.0.15



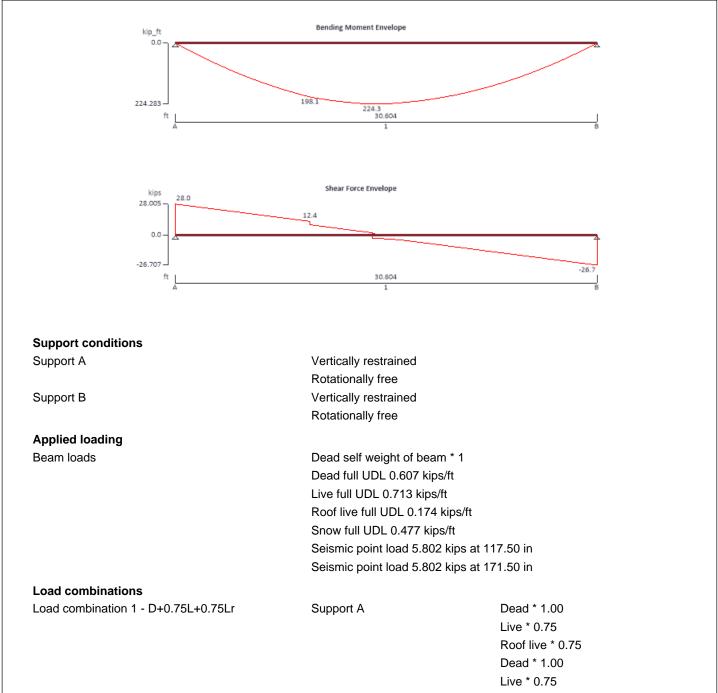
Load Envelope - Combination 2







Fast + Epp	Project Yaroslavsky Re	sidence			Job Ref. 8119	
Brooklyn NY 11217	Section Upper Level Tra	ansfer Beam 2 (I	B10)		Sheet no./rev. 2	
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date



Load combination 2 - D+0.75L+0.75S

Support A

Support B

Dead * 1.00 Live * 0.75 Roof live * 0.75 Dead * 1.00 Live * 0.75 Roof live * 0.75 Dead * 1.00 Live * 0.75 Snow * 0.75 Dead * 1.00

Tekla Tedds Fast + Epp	Project Yaroslavsky	Residence	Job Ref. 8119	Job Ref. 8119		
323 Dean Street, Suite #3	Section				Sheet no./rev	<i>.</i>
Brooklyn, NY 11217	Upper Level	Transfer Beam 2	2 (B10)		3	
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date
		•	•	Live * (0.75	•
				Snow *	* 0.75	
		Support B		Dead *	1.00	
				Live * 0	0.75	
				Snow *	* 0.75	
Load combination 3 - D+0.75L+	-0.525E+0.75S	Support A		Dead *	1.00	
				Live * 0		
				Snow *		
					c * 0.53	
				Dead *		
				Live * (
				Snow *	* 0.75 c * 0.53	
		Support B		Dead *		
		Support B		Live * (
				Snow *		
					c * 0.53	
Load combination 4 - D+0.7E		Support A		Dead * 1.00		
				Seismi	c * 0.70	
				Dead *	1.00	
				Seismi	c * 0.70	
		Support B		Dead *	1.00	
				Seismi	c * 0.70	
Analysis results						
Maximum moment		M _{max} = 224.3	•		0 kips_ft	
Maximum shear		V _{max} = 28 kip	S		-26.7 kips	
Deflection		δ _{max} = 1.1 in		δmin = 0		
Maximum reaction at support A		$R_{A_{max}} = 28 \text{ k}$	-	KA_min =	= 15.6 kips	
Unfactored dead load reaction at Unfactored live load reaction at		RA_Dead = 10.7 RA_Live = 10.9				
Unfactored roof live load reaction at		$R_{A_{Live}} = 10.3$ $R_{A_{Roof}}$ live = 2	-			
Unfactored snow load reaction		RA_ROON IVE = 2	-			
Unfactored seismic load reaction		$R_{A}Seismic = 7$	-			
Maximum reaction at support B		RB_max = 26.7	-	RB_min =	= 13.8 kips	
Unfactored dead load reaction		RB_Dead = 10.	-	_		
Unfactored live load reaction at		RB_Live = 10.9	-			
Unfactored roof live load reaction	on at support B	$R_{B_{Roof}} = 2$.7 kips			
Unfactored snow load reaction	at support B	RB_Snow = 7.3	kips			
Unfactored seismic load reaction	on at support B	RB_Seismic = 4.	6 kips			
Section details						
Section type		-	SC 15th Edn (v15.0))		
ASTM steel designation		A992				
Steel yield stress		F _y = 50 ksi				
Steel tensile stress		Fu = 65 ksi				
Modulus of elasticity		E = 29000 ks	i			

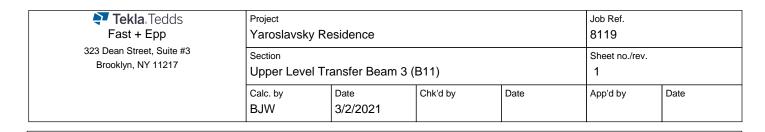
Fast + Epp	Project Yaroslavsk	y Residence			Job Ref. 8119		
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section		0 (D (0)	Sheet no			
•		el Transfer Beam		Data	4	Dete	
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date	
	lo)					
		·					
	1	ار ۲					
	.16.8"		- 0.53"				
	0.88						
	± ⊥ ₽ ±		`				
	·	4					
Safety factors							
Safety factor for tensile yielding		$\Omega_{ty} = 1.67$					
Safety factor for tensile rupture		$\Omega_{\rm tr} = 2.00$					
Safety factor for compression		Ωc = 1.67					
Safety factor for flexure		Ωb = 1.67					
Lateral bracing		Snan 1 has	continuous late	ral bracing			
Classification of continue for los	ool buokling	-		and bracing			
Classification of sections for lo	-						
Classification of flanges in flexu Width to thickness ratio	Ire - Table B4	4.10 (case 10) bf / (2 * tf) =	5 9/				
Limiting ratio for compact section			√[E / F _y] = 9.15				
Limiting ratio for non-compact sec	tion		$[E / F_y] = 24.08$	Compa	ct		
Classification of web in flexure			,]				
Width to thickness ratio		(d - 2 * k) / '	tw = 27.12				
Limiting ratio for compact section		· · · ·	* √[E / F _y] = 90.5	55			
Limiting ratio for non-compact sec	tion		√[E / F _y] = 137.		ct		
					Section is cor	mpact in flex	
Design of members for shear - (Chapter G						
Required shear strength		Vr = max(al	os(V _{max}), abs(V _m	nin)) = 28.005 kip	S		
Web area		A _w = d * t _w =	= 8.82 in ²				
Web plate buckling coefficient		$k_v = 5.34$					
		C _{v1} = 1					
Web shear coefficient - eq G2-3		$V_n = 0.6 * F$	⁷ y * A _w * C _{v1} = 26	4.600 kips			
Nominal shear strength - eq G6-1							
		Ω _v = 1.50	∉ = 176.400 kips				

Tekla Tedds Fast + Epp	^{Project} Yaroslavsky Re	esidence			Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Upper Level Transfer Beam 2 (B10)				Sheet no./rev. 5	
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date

Required flexural strength	Mr = max(abs(Ms1_max), abs(Ms1_min)) = 224.283 kips_ft
Yielding - Section F2.1	
Nominal flexural strength for yielding - eq F2-1	$M_{nyld} = M_p = F_y * Z_x = 729.167 \text{ kips_ft}$
Nominal flexural strength	Mn = Mnyid = 729.167 kips_ft
Allowable flexural strength	M _c = M _n / Ω _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 an	d snow loads

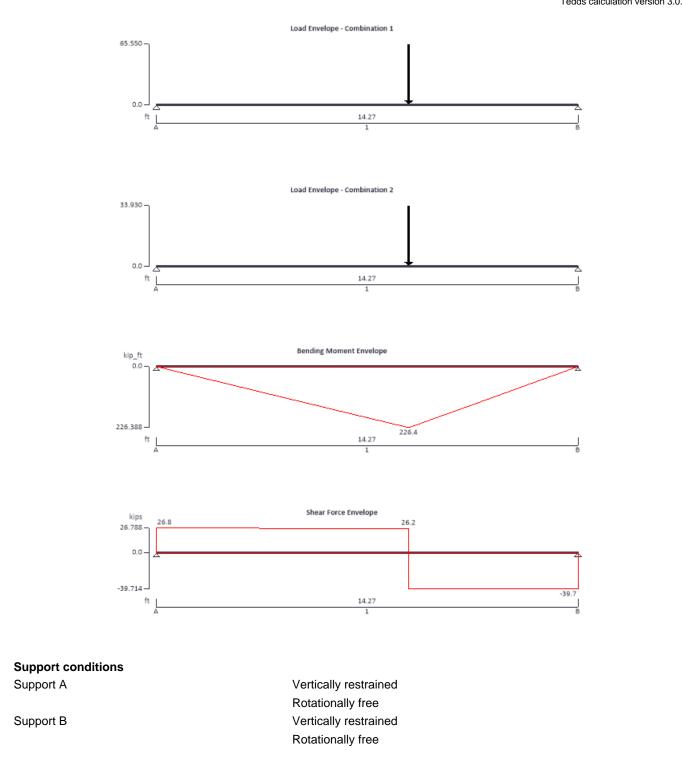
Limiting deflection $\delta_{lim} = L_{s1} / 240 = 1.53$ inMaximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = 1.079$ in

PASS - Maximum deflection does not exceed deflection limit



In accordance with AISC360-16 using the ASD method

Tedds calculation version 3.0.15



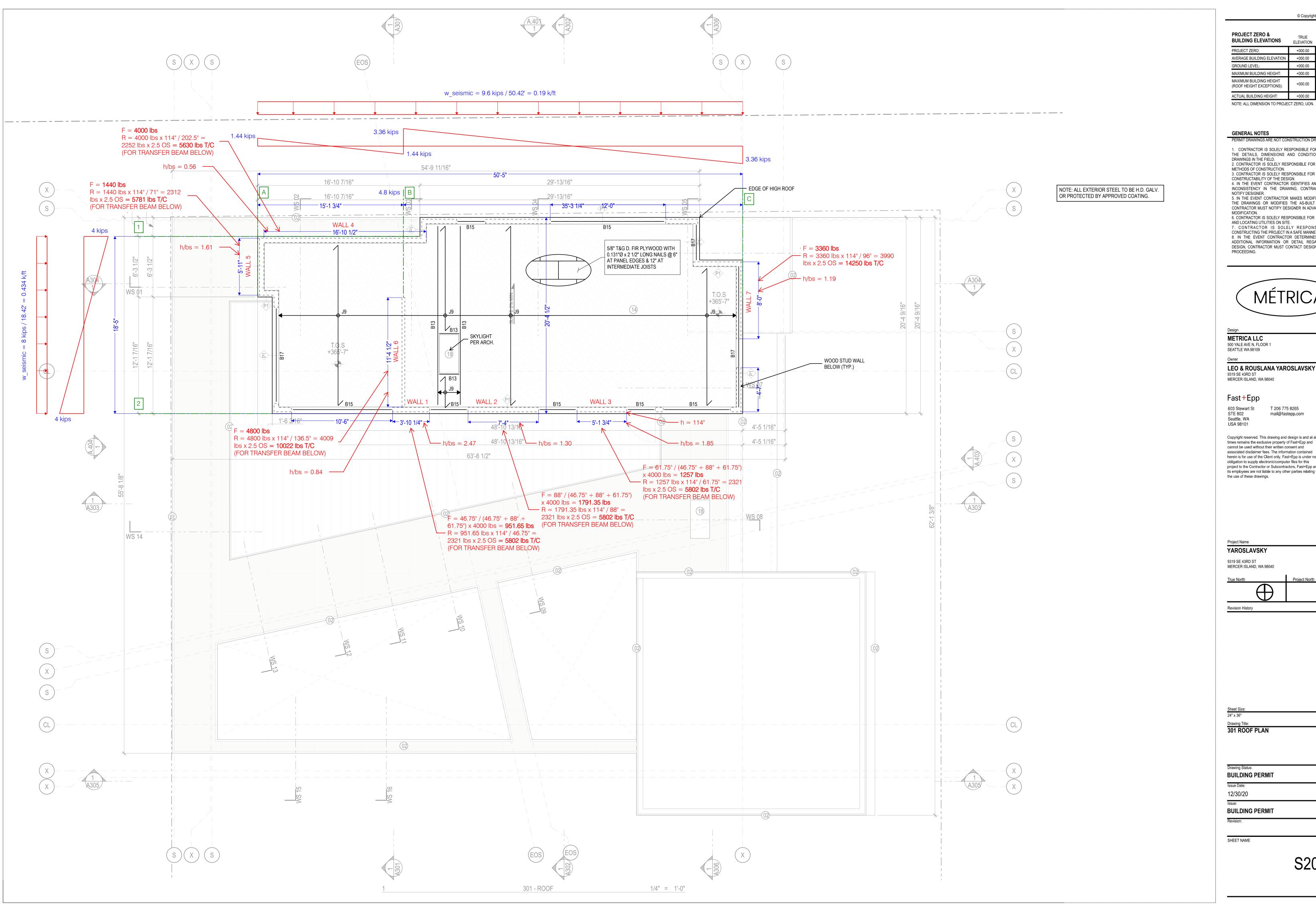
Tekla Tedds Fast + Epp	Project Yaroslavsky	Residence			Job Ref. 8119		
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Upper Level	Transfer Beam	3 (B11)		Sheet no./rev 2	'.	
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date	
Applied loading							
Beam loads		Dead self w	eight of beam *	1			
POINT LOADS FROM B9 AND B10			oad 20 kips at 2				
(SEE RESPECTIVE TEDDS OUTPUT)		-	ad 20.7 kips at				
		-	int load 4.8 kips				
		Snow point	oad 13.5 kips a	at 102.50 in			
		Seismic poir	nt load 19.9 kips	s at 102.50 in			
Load combinations		•					
Load combination 1 - D+0.75L+	J.525E+0.75S	Support A		Dead *			
				Live * (
				Snow Solomi			
				Dead *	ic * 1.00		
				Live * (
				Snow ³			
					ic * 1.00		
		Support B		Dead *			
		ouppoir D		Live * (
				Snow ³			
				Seismi	ic * 1.00		
Load combination 2 - D+0.7E		Support A		Dead *	* 1.00		
				Seismi	ic * 0.70		
				Dead *	* 1.00		
				Seismi	ic * 0.70		
		Support B		Dead *	* 1.00		
				Seismi	ic * 0.70		
Analysis results Maximum moment		M 000	line fi	5.4	Okine t		
Maximum moment Maximum shear		M _{max} = 226.4 V _{max} = 26.8	-		0 kips_ft - 39.7 kips		
Deflection		ν max = 20.0 δmax = 0.2 in	-	ν min = · δmin = (-		
Maximum reaction at support A					-		
Unfactored dead load reaction a	t support A	RA_max = 26. RA_Dead = 8.5	-	r t A_min :	= 14.1 kips		
Unfactored live load reaction at		RA_Dead = 6.3 RA_Live = 8.3	-				
Unfactored roof live load reaction at		$RA_Live = 0.3$ $RA_Roof live = '$	-				
Unfactored snow load reaction a		RA_ROOT INVE = RA_Snow = 5.4					
Unfactored seismic load reaction		RA_Seismic = 8					
Maximum reaction at support B		R _{B_max} = 39.	-	RB_min :	= 20.8 kips		
Unfactored dead load reaction a	t support B	RB_Dead = 12	-	_			
Unfactored live load reaction at		R _{B_Live} = 12.	-				
Unfactored roof live load reactio		$R_{B_{Roof live}} = 2$					
Unfactored snow load reaction a		RB_Snow = 8.1	-				
Unfactored seismic load reaction	n at support B	RB_Seismic = 1	1.9 kips				
Section details							
Section details							

Tekla Tedds Fast + Epp	Project Yaroslavsk	y Residence			Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Upper Leve	el Transfer Beam 3 (B11)		Sheet no./rev	
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date
ASTM steel designation		A992	•			•
Steel yield stress		F _y = 50 ksi				
Steel tensile stress		Fu = 65 ksi				
Modulus of elasticity		E = 29000 ksi				
Safety factors						
Safety factor for tensile yielding		$\Omega_{\rm ty} = 1.67$				
Safety factor for tensile rupture		$\Omega_{\rm tr} = 2.00$				
Safety factor for compression Safety factor for flexure		$Ω_{c} = 1.67$ $Ω_{b} = 1.67$				
-		S26 = 1.67				
Lateral bracing		Span 1 has as	ationala lataral	bracing		
			ntinuous lateral	Dracing		
	local buckling	- Section B4.1				
Classification of sections for	-					
Classification of flanges in fle	-	4.1b (case 10)				
Classification of flanges in fle Width to thickness ratio	exure - Table B4	4.1b (case 10) bf / (2 * tf) = 7.0				
Classification of flanges in fle Width to thickness ratio Limiting ratio for compact sectio	exure - Table B4	4.1b (case 10) bf / (2 * tf) = 7. λ _{pff} = 0.38 * √[[E / F _y] = 9.15			
Classification of flanges in fle Width to thickness ratio	exure - Table B4	4.1b (case 10) bf / (2 * tf) = 7.0	E / F _y] = 9.15	Compact		
Classification of flanges in fle Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact se Classification of web in flexur	exure - Table B4	4.1b (case 10) bf / (2 * tf) = 7.0 λpff = 0.38 * √[I λrff = 1.0 * √[E b (case 15)	E / F _y] = 9.15 / F _y] = 24.08	Compact		
Classification of flanges in fle Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact se Classification of web in flexur Width to thickness ratio	exure - Table B4 on ection re - Table B4.1k	4.1b (case 10) bf / (2 * tf) = 7.0 λpff = 0.38 * √[I λrff = 1.0 * √[E b (case 15) (d - 2 * k) / tw =	E / F _y] = 9.15 / F _y] = 24.08 : 35.85	Compact		
Classification of flanges in fle Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact section Classification of web in flexur Width to thickness ratio Limiting ratio for compact section	exure - Table Be on ection re - Table B4.1k	4.1b (case 10) $b_f / (2 * t_f) = 7.0$ $\lambda_{pff} = 0.38 * \sqrt{[I]}$ $\lambda_{rff} = 1.0 * \sqrt{[E]}$ (d - 2 * k) / tw = $\lambda_{pwf} = 3.76 * \sqrt{[I]}$	E / F _y] = 9.15 / F _y] = 24.08 : 35.85 E / F _y] = 90.55			
Classification of flanges in fle Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact se Classification of web in flexur Width to thickness ratio	exure - Table Be on ection re - Table B4.1k	4.1b (case 10) bf / (2 * tf) = 7.0 λpff = 0.38 * √[I λrff = 1.0 * √[E b (case 15) (d - 2 * k) / tw =	E / F _y] = 9.15 / F _y] = 24.08 : 35.85 E / F _y] = 90.55	Compact	oction is set	mpact in f
Classification of flanges in fle Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact sectio Classification of web in flexur Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact sectio	exure - Table Ba on ection re - Table B4.1k on ection	4.1b (case 10) $b_f / (2 * t_f) = 7.0$ $\lambda_{pff} = 0.38 * \sqrt{[I]}$ $\lambda_{rff} = 1.0 * \sqrt{[E]}$ (d - 2 * k) / tw = $\lambda_{pwf} = 3.76 * \sqrt{[I]}$	E / F _y] = 9.15 / F _y] = 24.08 : 35.85 E / F _y] = 90.55	Compact	ection is co	mpact in f
Classification of flanges in fle Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Classification of web in flexur Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Limiting ratio for non-compact section	exure - Table Ba on ection re - Table B4.1k on ection	4.1b (case 10) $b_f / (2 * t_f) = 7.0$ $\lambda_{pff} = 0.38 * \sqrt{[I]}$ $\lambda_{rff} = 1.0 * \sqrt{[E]}$ (d - 2 * k) / tw = $\lambda_{pwf} = 3.76 * \sqrt{[}$ $\lambda_{rwf} = 5.70 * \sqrt{[}$	E / F _y] = 9.15 / F _y] = 24.08 - 35.85 E / F _y] = 90.55 E / F _y] = 137.27	Compact S	ection is co	mpact in f
Classification of flanges in fle Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact section Classification of web in flexur Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Limiting ratio for non-compact section Required shear strength	exure - Table Ba on ection re - Table B4.1k on ection	4.1b (case 10) $b_f / (2 * t_f) = 7.4$ $\lambda_{pff} = 0.38 * \sqrt{[I]}$ $\lambda_{rff} = 1.0 * \sqrt{[E]}$ (d - 2 * k) / tw = $\lambda_{pwf} = 3.76 * \sqrt{[I]}$ $\lambda_{rwf} = 5.70 * \sqrt{[I]}$	E / F _y] = 9.15 / F _y] = 24.08 2 35.85 E / F _y] = 90.55 E / F _y] = 137.27 /max), abs(Vmin)	Compact	ection is co	mpact in f
Classification of flanges in fle Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact sectio Classification of web in flexur Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact sectio Design of members for shear Required shear strength Web area	exure - Table Ba on ection re - Table B4.1k on ection	4.1b (case 10) $b_f / (2 * t_f) = 7.0$ $\lambda_{pff} = 0.38 * \sqrt{[I]}$ $\lambda_{rff} = 1.0 * \sqrt{[E]}$ (d - 2 * k) / tw = $\lambda_{pwf} = 3.76 * \sqrt{[}$ $\lambda_{rwf} = 5.70 * \sqrt{[}$	E / F _y] = 9.15 / F _y] = 24.08 : 35.85 E / F _y] = 90.55 E / F _y] = 137.27 /max), abs(Vmin))	Compact S	ection is co	mpact in f
Classification of flanges in fle Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact section Classification of web in flexur Width to thickness ratio Limiting ratio for compact section Limiting ratio for non-compact section Limiting ratio for non-compact section Required shear strength	exure - Table Ba on ection re - Table B4.1k on ection - Chapter G	4.1b (case 10) bf / (2 * tf) = 7.0 $\lambda_{pff} = 0.38 * \sqrt{[I]}$ $\lambda_{rff} = 1.0 * \sqrt{[E]}$ b (case 15) (d - 2 * k) / tw = $\lambda_{pwf} = 3.76 * \sqrt{[I]}$ $\lambda_{rwf} = 5.70 * \sqrt{[I]}$ Vr = max(abs() Aw = d * tw = 6.	E / F _y] = 9.15 / F _y] = 24.08 : 35.85 E / F _y] = 90.55 E / F _y] = 137.27 /max), abs(Vmin))	Compact S	ection is co	mpact in f
Classification of flanges in fle Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact sectio Classification of web in flexur Width to thickness ratio Limiting ratio for compact sectio Limiting ratio for non-compact section Limiting ratio for non-compact sect	exure - Table Ba on ection re - Table B4.1k on ection - Chapter G	4.1b (case 10) bf / (2 * tf) = 7.0 $\lambda_{pff} = 0.38 * \sqrt{[I]}$ $\lambda_{rff} = 1.0 * \sqrt{[E]}$ b (case 15) (d - 2 * k) / tw = $\lambda_{pwf} = 3.76 * \sqrt{[I]}$ $\lambda_{rwf} = 5.70 * \sqrt{[I]}$ $V_r = max(abs(M_{aw} = d * tw = 6.1))$ $k_v = 5.34$	E / F _y] = 9.15 / F _y] = 24.08 2 35.85 E / F _y] = 90.55 E / F _y] = 137.27 /max), abs(Vmin)] 439 in ²	7 Compact S) = 39.714 kips	ection is co	mpact in f

Tekla Tedds Fast + Epp	Project Yaroslavsky	Residence	Project Yaroslavsky Residence				
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Upper Level	Section Upper Level Transfer Beam 3 (B11)			Sheet no./rev 4	Sheet no./rev. 4	
	Calc. by BJW	Date 3/2/2021	Chk'd by	Date	App'd by	Date	
Allowable shear strength		$V_c = V_n / \Omega_v$	= 128.770 kips				
		PASS -	Allowable she	ar strength e	exceeds required	d shear strengt	
				9		5	
Design of members for flexu	re in the major a	xis - Chapter F					
Design of members for flexu Required flexural strength	re in the major a	-	s(Ms1_max), abs(Ms1_min)) = 226	6 .388 kips_ft		
-	re in the major a	-	s(Ms1_max), abs(Ms1_min)) = 226	6 .388 kips_ft		
Required flexural strength		Mr = max(ab	s(Ms1_max), abs(6. 388 kips_ft		
Required flexural strength Yielding - Section F2.1		$M_r = max(ab)$ $M_{nyld} = M_p = 1$		57 kips_ft	6 .388 kips_ft		
Required flexural strength Yielding - Section F2.1 Nominal flexural strength for y		$M_r = max(ab)$ $M_{nyld} = M_p = 1$ $M_n = M_{nyld} = 3$	⁻ y * Z _x = 541.6 6	6 7 kips_ft	6 .388 kips_ft		
Required flexural strength Yielding - Section F2.1 Nominal flexural strength for yields Nominal flexural strength		$M_{r} = max(ab)$ $M_{nyld} = M_{p} = 1$ $M_{n} = M_{nyld} = 3$ $M_{c} = M_{n} / \Omega_{b}$		5 7 kips_ft _ft	6.388 kips_ft ceeds required f	lexural streng	
Required flexural strength Yielding - Section F2.1 Nominal flexural strength for yield Nominal flexural strength	elding - eq F2-1	$M_{r} = max(ab)$ $M_{nyld} = M_{p} = 1$ $M_{n} = M_{nyld} = 3$ $M_{c} = M_{n} / \Omega_{b}$		5 7 kips_ft _ft		lexural streng	
Required flexural strength Yielding - Section F2.1 Nominal flexural strength for yield Nominal flexural strength Allowable flexural strength	elding - eq F2-1 cal deflection	$M_r = max(ab)$ $M_{nyld} = M_p = 1$ $M_n = M_{nyld} = 3$ $M_c = M_n / \Omega_b$ $PASS - All c$		5 7 kips_ft _ft		lexural streng	
Required flexural strength Yielding - Section F2.1 Nominal flexural strength for yi Nominal flexural strength Allowable flexural strength Design of members for vertice	elding - eq F2-1 cal deflection	$M_r = max(ab)$ $M_{nyld} = M_p = 1$ $M_n = M_{nyld} = 3$ $M_c = M_n / \Omega_b$ $PASS - All c$	^F y * Z _x = 541.66 5 41.667 kips_ft = 324.351 kips wable flexural	5 7 kips_ft _ft		lexural streng	
Required flexural strength Yielding - Section F2.1 Nominal flexural strength for yield Nominal flexural strength Allowable flexural strength Design of members for verting Consider deflection due to deal	elding - eq F2-1 cal deflection	$M_{r} = max(ab)$ $M_{nyld} = M_{p} = 1$ $M_{n} = M_{nyld} = 3$ $M_{c} = M_{n} / \Omega_{b}$ PASS - Allo	^F y * Z _x = 541.66 5 41.667 kips_ft = 324.351 kips wable flexural	57 kips_ft _ft strength exc		lexural streng	

3 | LATERAL DESIGN

3.1 | WOOD FRAME SHEAR WALL DESIGN



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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	

PERMIT DRAWINGS ARE NOT CONSTRUCTION DRAWINGS

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5. IN THE EVENT CONTRACTOR MAKES MODIFICATIONS TO THE DRAWINGS OR MODIFIES THE AS-BUILT CONDITION, CONTRACTOR MUST NOTIFY DESIGNER IN ADVANCE OF THE 6. CONTRACTOR IS SOLELY RESPONSIBLE FOR IDENTIFYING

6. CONTRACTOR IS SOLELY RESPONSIBLE FOR IDENTIFYING AND LOCATING UTILITIES ON SITE.
7. CONTRACTOR IS SOLELY RESPONSIBLE FOR CONSTRUCTING THE PROJECT IN A SAFE MANNER.
8. IN THE EVENT CONTRACTOR DETERMINES IT NEEDS ADDITIONAL INFORMATION OR DETAIL REGARDING THE DEFICION CONTRACTOR MUST CONTACT DESIGNED AFFORE DESIGN, CONTRACTOR MUST CONTACT DESIGNER BEFORE



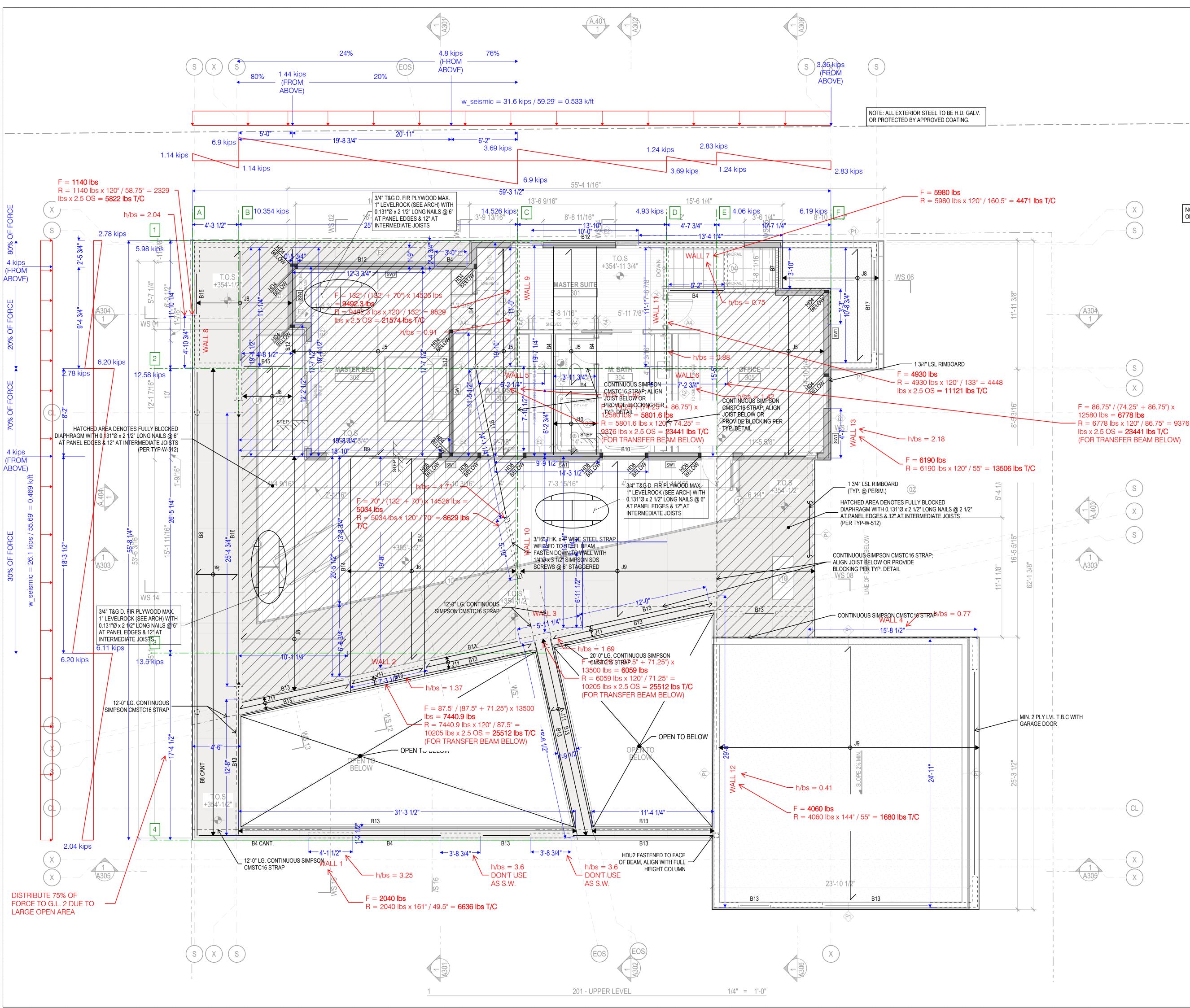
LEO & ROUSLANA YAROSLAVSKY

T 206 775 8265 mail@fastepp.com

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S204



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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 PROJE	CT ZERO, UON.	

GENERAL NOTES

PERMIT DRAWINGS ARE NOT CONSTRUCTION D

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 IN THE EVENT CONTRACTOR DETERMINES IT NEEDS ADDITIONAL INFORMATION OR DETAIL REGARDING THE DEDIDIO CONTRACTOR DETAIL REGARDING THE DESIGN, CONTRACTOR MUST CONTACT DESIGNER BEFORE



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PROCEEDING.

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Project Name YAROSLAVSKY

9319 SE 43RD ST MERCER ISLAND, WA 98040

(+)Revision History

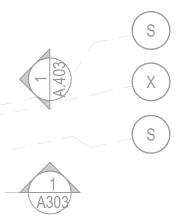
Sheet Size:	
24" x 36" Drawing Title:	
UPPER LEVEL PLAN	
Drawing Status:	
Drawing Status: BUILDING PERMIT	
BUILDING PERMIT	
BUILDING PERMIT Issue Date: 12/30/20	
BUILDING PERMIT Issue Date: 12/30/20 Issue:	
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BUILDING PERMIT Issue Date: 12/30/20 Issue: BUILDING PERMIT	0,

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

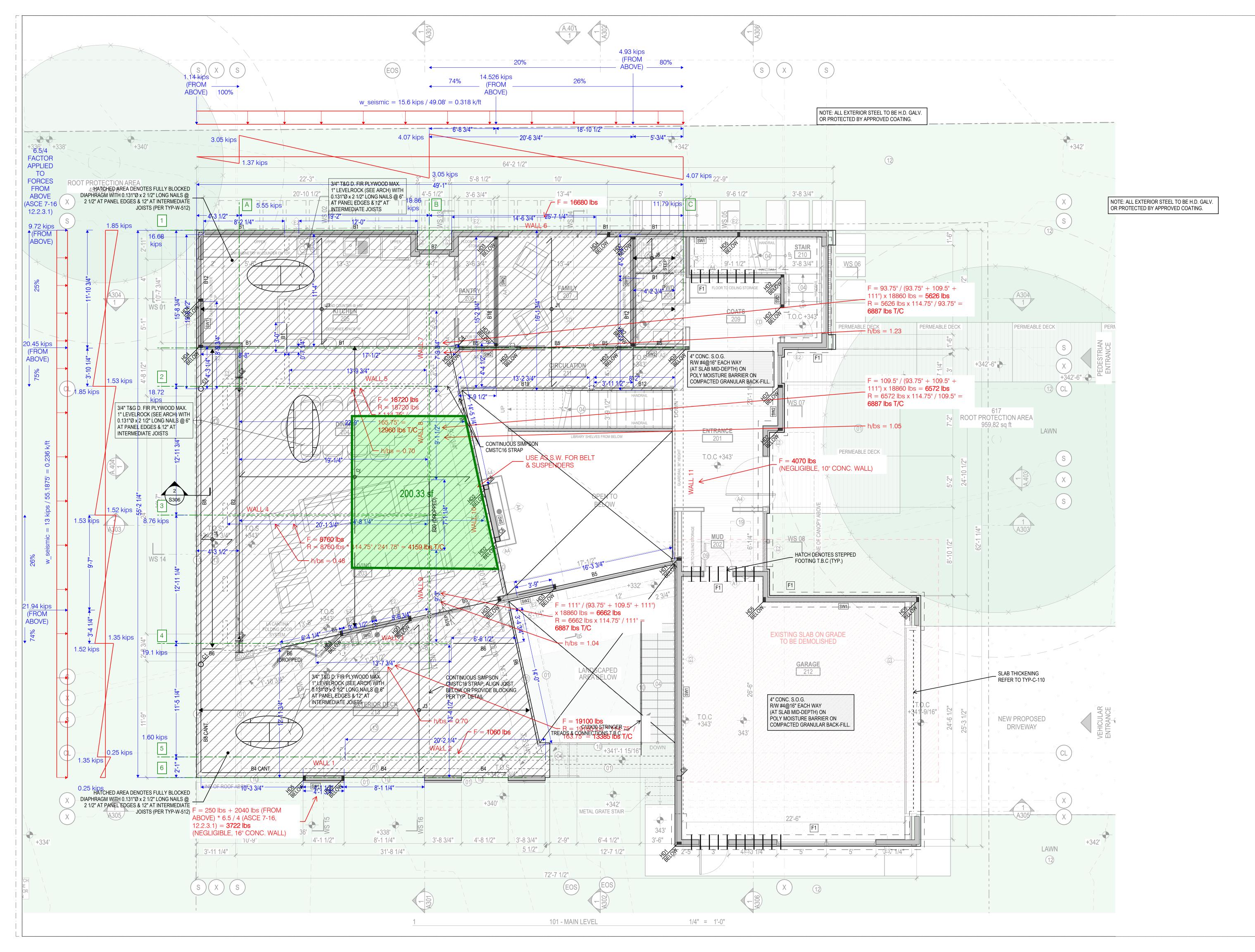




F = 86.75" / (74.25" + 86.75") x 12580 lbs = **6778 lbs** $-R = 6778 \text{ lbs x } 120^{\circ} / 86.75^{\circ} = 9376$ lbs x 2.5 OS = **23441 lbs T/C** (FOR TRANSFER BEAM BELOW)







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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 PROJE	CT ZERO, UON.	

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Project Name
YAROSLAVSKY

9319 SE 43RD ST

MERCER ISLAND, WA 98040

 True North
 Project North

 Revision History
 Image: Construction of the second secon

Sheet Size:	
24" x 36" Drawing Title:	
MAIN LEVEL PLAN	
BUILDING PERMIT	
12/30/20 Issue:	
BUILDING PERMIT Issue Date: 12/30/20	
BUILDING PERMIT Issue Date: 12/30/20 Issue:	01

S202

Tekla Tedds Fast + Epp					Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Wood Shear Wall - Supp. High Roof Wall 1			Sheet no./rev. 1		
	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date

WOOD SHEAR WALL DESIGN (NDS)

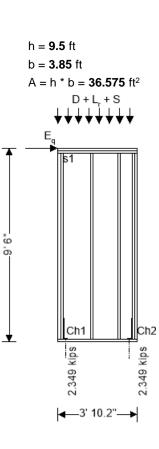
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Panel details

Structural wood panel sheathing on one side

Panel height Panel length

Total area of wall



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	As = 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	Ae = 16.5 in ²
Hole diameter	Dia = 1 in
Net cross-sectional area of end posts	Aen = 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	ka = 30000 lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)Species, grade and size classificationDouglas Fir-Larch, no.2 grade, 2" & wider

Tedds calculation version 1.2.04

Tekla Tedds Fast + Epp	Project Yaroslavsky Residence				Job Ref. 8119		
323 Dean Street, Suite #3	Section				Sheet no./rev	Sheet no./rev.	
Brooklyn, NY 11217	Wood Shear	Wall - Supp. Hig	h Roof Wall 1		2		
	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date	
Specific gravity		G = 0.50					
Tension parallel to grain		Ft = 575 lb/in	2				
Compression parallel to grain		Fc = 1350 lb/i	in²				
Modulus of elasticity		E = 1600000 lb/in ²					
Minimum modulus of elasticity		Emin = 58000	0 lb/in²				
Sheathing details							
Sheathing material		15/32" wood	d panel 3-ply	plywood she	athing		
Fastener type		8d common	nails at 3"ce	nters			
From SDPWS Table 4.3A Nomin	al Unit Shear C	Capacities for W	Vood-Frame S	hear Walls - V	Wood-based Pa	nels	
Nominal unit shear capacity fo		-					
Nominal unit shear capacity fo	-	V _w = 1370 lb,					
Apparent shear wall shear stiff	-	Ga = 15 kips					
		io inpo					
Loading details		D = 276.25 lb	\/ f +				
Dead load acting on top of panel Roof live load acting on top of par		D = 276.25 lt Lr = 369 lb/ft	D/IT				
Snow load acting on top of panel	IEI	S = 553 lb/ft					
Self weight of panel		$S = 333 \text{ lb/ft}^2$ Swt = 12 lb/ft ²					
In plane seismic load acting at he	ad of nanel	$E_q = 952 \text{ lbs}$					
Design spectral response accel. p	-						
From IBC 2018 cl.1605.2	. ,						
Load combination no.1		1 2D + 1 6(L	or S or R) + 0	5W			
Load combination no.2			0.5Lf + 0.5(Lr 0				
Load combination no.3		1.2D + E + 0.		/			
Load combination no.4		0.9D + W					
Load combination no.5		0.9D + E					
Adjustment factors							
Format conversion factor for tensi	on – Table N1						
		KFt = 2.70					
Format conversion factor for comp	pression – Table	e N1					
		KFc = 2.40					
Format conversion factor for mode	ulus of elasticity						
		Kfe = 1.76					
Resistance factor for tension – Ta	ible N2	$\phi t = 0.80$					
Resistance factor for compression		$\phi_c = 0.90$					
Resistance factor for modulus of e	elasticity – Table						
		$\phi_s = 0.85$					
Time effect factor – Table N3		$\lambda = 1.00$					
Sheathing resistance factor		φ D = 0.80					
Size factor for tension – Table 4A		CFt = 1.30					
	le 4A	CFc = 1.10					
Size factor for compression – Tab		· · · ·					
Size factor for compression $-$ Tab Wet service factor for tension $-$ Tab Wet service factor for compressio		Смt = 1.00 Смс = 1.00					

Tekla Tedds Fast + Epp	Project Yaroslavsky R	esidence			Job Ref. 8119		
323 Dean Street, Suite #3	Section				Sheet no./rev.		
Brooklyn, NY 11217	Wood Shear V	Vall - Supp. Hi	gh Roof Wall 1		3		
	Calc. by	Date	Chk'd by	Date	App'd by	Date	
	BJW	2/17/2021					
		Сме = 1.00					
Temperature factor for tension	– Table 2.3.3	Ctt = 1.00					
Temperature factor for compres							
	af alaatiaitu Tak	$C_{tc} = 1.00$					
Temperature factor for modulus	s of elasticity – Tabl	le 2.3.3 Ct∈ = 1.00					
Incising factor – cl.4.3.8		Ci = 1.00					
Buckling stiffness factor – cl.4.4	4.2	C⊤ = 1.00					
Adjusted modulus of elasticity		Emin' = Emin *	Кге * фs * Сме *	CtE * Ci * CT = 8	870000 psi		
Critical buckling design value		Fce = 0.822	× E _{min} ' / (h / d)²	= 1665 psi			
Reference compression design	value			Ctc * CFc * Ci = 3	3208 psi		
For sawn lumber		c = 0.8					
Column stability factor - eqr	1.3.7-1	C _P = (1 + (I	FcE / Fc*)) / (2 >	< c) – √([(1 + (Fce / Fc*)) / (2 ×	C)] ² - (F _{CE}	
		F_{c}^{*} / c) = 0.45					
From SDPWS Table 4.3.4 Max	kimum Shear Wall	Aspect Ratio	5				
Maximum shear wall aspect rat		3.5	-				
Shear wall length		b = 3.85 ft					
Shear wall aspect ratio		h / b = 2.468	}				
Segmented shear wall capac	ity						
Maximum shear force under	seismic loading	Vs_max = Eq	= 0.952 kips				
Shear capacity for seismic lo	bading	Vs = ϕ D * Vs	* b * (1.25 - 0	.125 * h / bs) =	= 2.842 kips		
		Vs_max / Vs =	0.335				
		PASS - She	ar capacity for	seismic load e	exceeds maxim	um shear fo	
Chord capacity for chords 1	and 2						
Shear wall aspect ratio		h / b = 2.468	}				
Load combination 5							
Choor force for manine t	20		E2 kine				
Shear force for maximum tension		$V = E_q = 0.9$	-				
Axial force for maximum ten	sion	P = 0 kips :	= 0 kips	ns			
Axial force for maximum ten Maximum tensile force in chord	sion	P = 0 kips = T = V * h / (t	= 0 kips b) - P = 2.349 ki	ps			
Axial force for maximum ten Maximum tensile force in chord Maximum applied tensile stress	sion	P = 0 kips = T = V * h / (k ft = T / A _{en} =	= 0 kips b) - P = 2.349 ki 174 lb/in²		15 lb/in²		
Axial force for maximum ten Maximum tensile force in chord	sion	P = 0 kips = T = V * h / (k ft = T / Aen = Ft' = Ft * KFt	= 0 kips b) - P = 2.349 ki 174 lb/in ² * φt * λ * CMt * C	ps tt * CFt * Ci = 16 '	15 lb/in²		
Axial force for maximum ten Maximum tensile force in chord Maximum applied tensile stress	sion	$P = 0 kips = T = V * h / (t)$ $f_t = T / A_{en} = F_t' = F_t * K_{Ft}$ $f_t / F_t' = 0.10$	= 0 kips b) - P = 2.349 ki 174 lb/in ² * φt * λ * C _{Mt} * C 8	tt * CFt * Ci = 16 '	15 lb/in² aximum applied	tensile str	
Axial force for maximum ten Maximum tensile force in chord Maximum applied tensile stress	sion	$P = 0 kips = T = V * h / (t)$ $f_t = T / A_{en} = F_t' = F_t * K_{Ft}$ $f_t / F_t' = 0.10$	= 0 kips b) - P = 2.349 ki 174 lb/in ² * φt * λ * C _{Mt} * C 8	tt * CFt * Ci = 16 '		tensile str	
Axial force for maximum ten Maximum tensile force in chord Maximum applied tensile stress Design tensile stress	sion I	$P = 0 kips = T = V * h / (t)$ $f_t = T / A_{en} = F_t' = F_t * K_{Ft}$ $f_t / F_t' = 0.10$	= 0 kips b) - P = 2.349 ki 174 lb/in ² * φt * λ * C _{Mt} * C 8 ign tensile stre	tt * CFt * Ci = 16 '		tensile str	
Axial force for maximum ten Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3	sion	P = 0 kips = T = V * h / (k + 1) + (k +	= 0 kips) - P = 2.349 ki 174 lb/in ² * φt * λ * CMt * C 8 ign tensile stre 52 kips	tt * CFt * Ci = 16 ess exceeds m			
Axial force for maximum ten Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp	sion	P = 0 kips = T = V * h / (k + 1) + (k +	= 0 kips b) - P = 2.349 ki 174 lb/in ² * φt * λ * CMt * C 8 ign tensile stre 52 kips D + Swt * h) + t	tt * CFt * Ci = 16 ess exceeds m	aximum applied		
Axial force for maximum ten Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp	sion I s pression npression	P = 0 kips = T = V * h / (k + 1) + (k +	= 0 kips b) - P = 2.349 ki 174 lb/in ² * φt * λ * CMt * C 8 ign tensile stre 52 kips D + Swt * h) + t	tt * CFt * Ci = 16 ess exceeds m 0.2 * S _{DS} * (D -	aximum applied		
Axial force for maximum ten Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp Axial force for maximum comp	sion I pression npression	P = 0 kips = T = V * h / (k + 1) + (k +	= 0 kips b) - P = 2.349 ki 174 lb/in ² * φt * λ * CMt * C 8 ign tensile stre 52 kips D + Swt * h) + t s b) + P = 2.968 k	tt * CFt * Ci = 16 ess exceeds m 0.2 * S _{DS} * (D -	aximum applied		
Axial force for maximum ten Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp Axial force for maximum comp Maximum compressive force in	sion I pression npression	P = 0 kips = T = V * h / (k + 1) + (k +	= 0 kips b) - P = 2.349 ki 174 lb/in ² * ϕ_t * λ * C _{Mt} * C 8 ign tensile stress 52 kips D + S _{Wt} * h) + t 5 b) + P = 2.968 k 180 lb/in ²	tt * CFt * Ci = 16 ess exceeds m 0.2 * S _{DS} * (D -	aximum applied + S _{wt} * h) + + C		

Tekla Tedds	Project	Job Ref.				
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Hold down force	
Chord 1	T ₁ = 2.349 kips
Chord 2	T ₂ = 2.349 kips
Seismic deflection	
Design shear force	$V_{\delta s} = E_q = 0.952 \text{ kips}$
Deflection limit	$\Delta_{s_allow} = 0.020 * h = 2.28$ in
Induced unit shear	v _{ðs} = V _{ðs} / b = 247.27 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * h) = 2.349 kips
Shear wall elastic deflection - Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_\delta / (k_a * b) = 0.367$ in
Deflection ampification factor	$C_{d\delta} = 4$
Seismic importance factor	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	δ_{sws} = Cd δ * δ_{swse} / Ie = 1.466 in
	$\delta_{sws} / \Delta_{s_allow} = 0.643$
	PASS - Shear wall deflection is less than deflection limit

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Tedds calculation version 1.2.04

WOOD SHEAR WALL DESIGN (NDS)

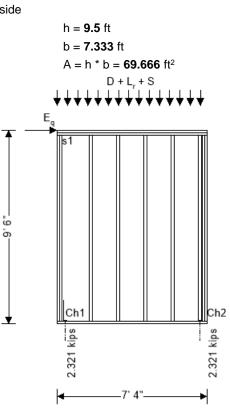
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Panel details

Structural wood panel sheathing on one side

Panel height Panel length

Total area of wall



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	As = 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	Ae = 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	ka = 30000 lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)Species, grade and size classificationDouglas Fir-Larch, no.2 grade, 2" & wider

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Brooklyn, NY 11217		Wall - Supp. Hig	h Roof Wall 2		2				
	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date			
Specific gravity		G = 0.50							
Tension parallel to grain		Ft = 575 lb/in	2						
Compression parallel to grain		Fc = 1350 lb/	in ²						
Modulus of elasticity		E = 1600000	lb/in ²						
Minimum modulus of elasticity		Emin = 58000	0 lb/in ²						
Sheathing details									
Sheathing material		15/32'' woo	d panel 3-ply	plywood she	athing				
Fastener type		8d commor	nails at 3"ce	nters					
From SDPWS Table 4.3A Nomi	nal Unit Shear (Capacities for V	Vood-Frame S	hear Walls - V	Nood-based Pa	nels			
Nominal unit shear capacity for		-							
Nominal unit shear capacity for		-							
Apparent shear wall shear stif	-	Ga = 15 kips	s/in						
Loading details		-							
Dead load acting on top of panel		D = 306 lb/ft							
Roof live load acting on top of participation	nel	$L_r = 408 \text{ lb/ft}$							
Snow load acting on top of panel			S = 611.25 lb/ft						
Self weight of panel			Swt = 12 lb/ft ²						
In plane seismic load acting at he	ad of panel	E ₉ = 1792 lbs	E _q = 1792 lbs						
Design spectral response accel.	-	ds SDS = 0.944							
From IBC 2018 cl.1605.2									
Load combination no.1		1.2D + 1.6(L	or S or R) + 0	.5W					
Load combination no.2		1.2D + W + ().5Lf + 0.5(Lr o	r S or R)					
Load combination no.3		1.2D + E + 0	.5Lf + 0.7S						
Load combination no.4		0.9D + W							
Load combination no.5		0.9D + E							
Adjustment factors									
Format conversion factor for tens	ion – Table N1								
		KFt = 2.70							
Format conversion factor for com	pression – Tabl	e N1							
		KFc = 2.40							
Format conversion factor for mod	lulus of elasticity	– Table N1							
		-	K _{FE} = 1.76						
Resistance factor for tension – T	able N2	$\phi_t = 0.80$							
Resistance factor for compressio		$\phi_c = 0.90$							
Resistance factor for modulus of	elasticity – Tabl								
		$\phi_s = 0.85$							
Time effect factor – Table N3		$\lambda = 1.00$							
Sheathing resistance factor		φD = 0.80							
	A	CFt = 1.30							
Size factor for tension - Table 44		CFc = 1.10							
Size factor for tension – Table 44 Size factor for compression – Ta	ble 4A	$C_{F_{C}} = 1.10$							
	able 4A	$C_{Fc} = 1.10$ $C_{Mt} = 1.00$ $C_{Mc} = 1.00$							

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	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date	
		Сме = 1.00					
Temperature factor for tension	– Table 2.3.3	Ctt = 1.00					
Temperature factor for compres		Ctc = 1.00					
Temperature factor for modulus	of elasticity – Tab						
		CtE = 1.00					
Incising factor – cl.4.3.8		Ci = 1.00					
Buckling stiffness factor – cl.4.4	1.2	C⊤ = 1.00					
Adjusted modulus of elasticity		Emin' = Emin *	Кге * фs * Сме *	CtE * Ci * CT = 8	70000 psi		
Critical buckling design value			< Emin' / (h / d) ²		-		
Reference compression design	value			$C_{tc} * C_{Fc} * C_i = 3$	3208 psi		
For sawn lumber		c = 0.8	T C		F - -		
Column stability factor - eqn	.3.7-1	$C_{P} = (1 + (F_{P}))^{2}$	FcE / Fc*)) / (2 >	< c) – √([(1 + (F	FcE / Fc*)) / (2 ×	c)]² - (Fc∈	
• • • • • • • • • • • • • • •		$C_{P} = (1 + (F_{cE} / F_{c}^{*})) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_{c}^{*})) / (2 \times c)]^{2} - (F_{cE} / F_{c}^{*})} / c) = 0.45$					
		, ,					
From SDPWS Table 4.3.4 Max		3.5	5				
Maximum shear wall aspect rati	0	5.5 b = 7.333 ft					
Shear wall aspect ratio		h / b = 1.295					
-	4	117.5 - 1.200					
Segmented shear wall capaci Maximum shear force under	-	Va may - Fa	= 1.792 kips				
	0			-			
Shear capacity for seismic lo	ading	•	* b = 5.749 kip	DS			
		Vs_max / Vs = PASS - Shea		seismic load e	xceeds maximi	ım shear fo	
		17100 01100					
	and O						
Chord capacity for chords 1 a	and 2	h/h – 1 205					
Shear wall aspect ratio	and 2	h / b = 1.295	i				
Shear wall aspect ratio Load combination 5	on	h / b = 1.295 V = E _q = 1.7 P = 0 kips =	92 kips				
Shear wall aspect ratio Load combination 5 Shear force for maximum tension	on sion	V = E _q = 1.7 9 P = 0 kips =	92 kips	ps			
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension	on sion	V = E _q = 1.7 9 P = 0 kips =	92 kips = 0 kips)) - P = 2.321 ki	ps			
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord	on sion	$V = E_q = 1.79$ P = 0 kips = T = V * h / (b) $f_t = T / A_{en} =$	92 kips = 0 kips ı) - P = 2.321 ki 172 lb/in²	ps tt * CFt * Ci = 161	5 lb/in ²		
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress	on sion	$V = E_q = 1.79$ P = 0 kips = T = V * h / (b) $f_t = T / A_{en} =$	92 kips = 0 kips)) - Ρ = 2.321 ki 172 lb/in ² * φt * λ * Cмt * C		5 lb/in ²		
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress	on sion	$V = E_q = 1.79$ $P = 0 \text{ kips} =$ $T = V * h / (b)$ $f_t = T / A_{en} =$ $F_t' = F_t * K_{Ft} *$ $f_t / F_t' = 0.100$	92 kips = 0 kips)) - Ρ = 2.321 ki 172 lb/in ² * φt * λ * C _{Mt} * C 7			tensile str	
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress	on sion	$V = E_q = 1.79$ $P = 0 \text{ kips} =$ $T = V * h / (b)$ $f_t = T / A_{en} =$ $F_t' = F_t * K_{Ft} *$ $f_t / F_t' = 0.100$	92 kips = 0 kips)) - Ρ = 2.321 ki 172 lb/in ² * φt * λ * C _{Mt} * C 7	tt * CFt * Ci = 161		tensile str	
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress	on sion	$V = E_q = 1.79$ $P = 0 \text{ kips} = 1.79$ $T = V * h / (b)$ $f_t = T / A_{en} = 1.79$ $F_t' = F_t * K_{Ft} * f_t / F_t' = 0.100$ $PASS - Des$ $V = E_q = 1.79$	92 kips = 0 kips)) - P = 2.321 ki 172 lb/in ² * φι * λ * C _{Mt} * C 7 ign tensile stre 92 kips	tt * CFt * Ci = 161 ess exceeds ma	aximum appliec		
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress	on sion	$V = E_q = 1.79$ $P = 0 \text{ kips} = 1.79$ $T = V * h / (b)$ $f_t = T / A_{en} = 1.79$ $F_t' = F_t * K_{Ft} * f_t / F_t' = 0.100$ $PASS - Des$ $V = E_q = 1.79$	92 kips = 0 kips)) - P = 2.321 ki 172 lb/in ² * φι * λ * C _{Mt} * C 7 ign tensile stre 92 kips	tt * CFt * Ci = 161	aximum appliec		
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp	on sion	$V = E_q = 1.79$ $P = 0 \text{ kips} = 1.79$ $T = V * h / (b)$ $f_t = T / A_{en} = 1.79$ $F_t' = F_t * K_{Ft} * f_t / F_t' = 0.100$ $PASS - Des$ $V = E_q = 1.79$	92 kips = 0 kips b) - P = 2.321 ki 172 lb/in ² * φι * λ * CMt * C 7 ign tensile stre 92 kips D + Swt * h) + t	tt * CFt * Ci = 161 ess exceeds ma	aximum appliec		
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp	on sion ression npression	$V = E_q = 1.79$ $P = 0 \text{ kips} = 1.79$ $T = V * h / (b)$ $f_t = T / A_{en} = 1.79$ $F_t' = F_t * K_{Ft} * f_t / F_t' = 0.100$ $PASS - Des$ $V = E_q = 1.79$ $P = (1.2 * (I) = 0.674 \text{ kips})$	92 kips = 0 kips b) - P = 2.321 ki 172 lb/in ² * φι * λ * CMt * C 7 ign tensile stre 92 kips D + Swt * h) + t	tt * CFt * Ci = 161 ess exceeds ma 0.2 * Sds * (D +	aximum appliec		
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp Axial force for maximum comp	on sion ression npression chord	$V = E_q = 1.79$ $P = 0 \text{ kips} = 1.79$ $T = V * h / (b)$ $f_t = T / A_{en} = 1.79$ $F_t' = F_t * K_{Ft} * f_t / F_t' = 0.100$ $PASS - Des$ $V = E_q = 1.79$ $P = (1.2 * (I) = 0.674 \text{ kips})$	92 kips = 0 kips b) - P = 2.321 ki 172 lb/in ² * φι * λ * CMt * C 7 ign tensile stre 92 kips D + Swt * h) + F 5 b) + P = 2.996 k	tt * CFt * Ci = 161 ess exceeds ma 0.2 * Sds * (D +	aximum appliec		
Shear wall aspect ratio Load combination 5 Shear force for maximum tensio Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp Axial force for maximum comp	on sion ression npression chord	$V = E_q = 1.79$ $P = 0 \text{ kips} = 1.79$ $T = V * h / (b)$ $f_t = T / A_{en} = 1.79$ $F_t = F_t * K_{Ft} * f_t / F_t = 0.100$ $PASS - Des$ $V = E_q = 1.79$ $P = (1.2 * (l))$ $P = (1.2 * (l))$ $C = V * h / (b)$ $f_c = C / A_e = 1.79$	92 kips = 0 kips b) - P = 2.321 ki 172 lb/in ² * φt * λ * CMt * C 7 ign tensile stre 92 kips D + Swt * h) + t 5 b) + P = 2.996 k 182 lb/in ²	tt * CFt * Ci = 161 ess exceeds ma 0.2 * Sds * (D +	aximum appliec - Swt * h) + + 0		

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Hold down force	
Chord 1	T1 = 2.321 kips
Chord 2	T ₂ = 2.321 kips
Seismic deflection	
Design shear force	V _{ðs} = E _q = 1.792 kips
Deflection limit	$\Delta_{s_allow} = 0.020 * h = 2.28$ in
Induced unit shear	v _{ðs} = V _{ðs} / b = 244.36 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * h) = 2.321 kips
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_{\delta} / (k_a * b) = 0.264$ in
Deflection ampification factor	$C_{d\delta} = 4$
Seismic importance factor	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	δ_{sws} = Cdô * δ_{swse} / Ie = 1.055 in
	$\delta_{sws} / \Delta_{s_allow} = 0.463$
	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

Panel details

Structural wood panel sheathing on one side

Panel height Panel length

Total area of wall

Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	As = 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	Ae = 16.5 in ²
Hole diameter	Dia = 1 in
Net cross-sectional area of end posts	Aen = 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	ka = 30000 lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)Species, grade and size classificationDouglas Fir-Larch, no.2 grade, 2" & wider

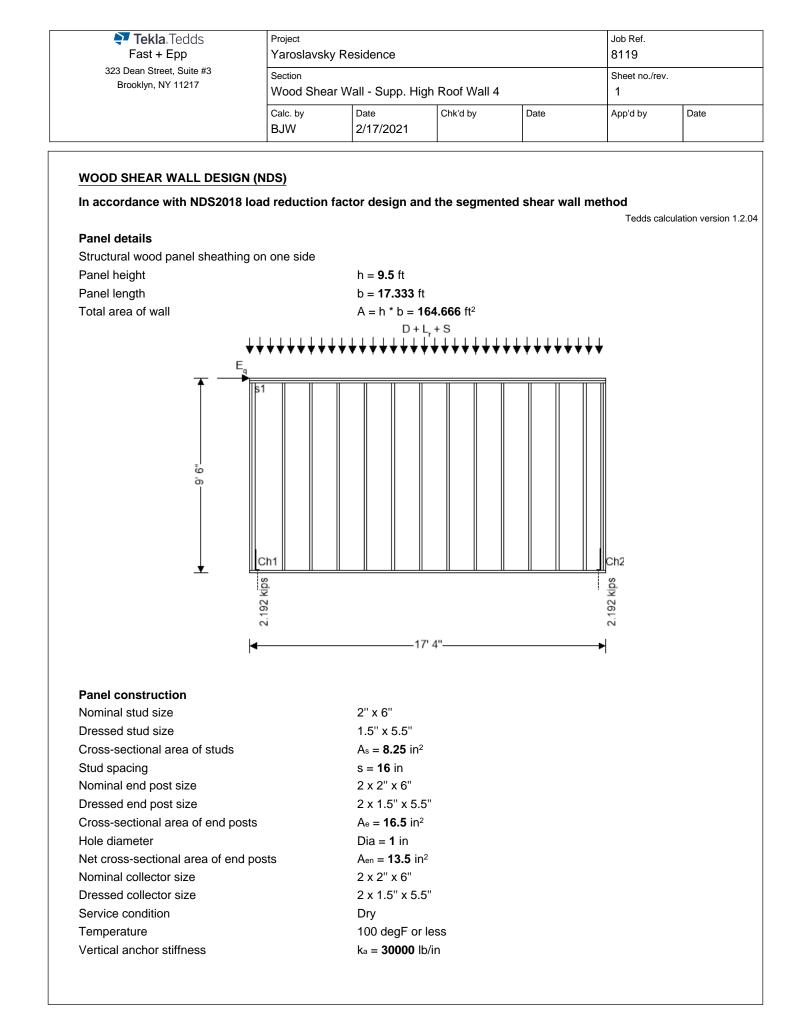
Tedds calculation version 1.2.04

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Brooklyn, NY 11217		Wall - Supp. Hig	h Roof Wall 3		2				
	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date			
Specific gravity		G = 0.50							
Tension parallel to grain		Ft = 575 lb/in	2						
Compression parallel to grain		Fc = 1350 lb/	in²						
Modulus of elasticity		E = 1600000	lb/in ²						
Minimum modulus of elasticity		Emin = 58000	0 lb/in ²						
Sheathing details									
Sheathing material		15/32" woo	d panel 3-ply	plywood she	eathing				
Fastener type		8d common	nails at 3"ce	enters					
From SDPWS Table 4.3A Nomina	l Unit Shear	Capacities for V	Vood-Frame S	Shear Walls -	Wood-based Pa	nels			
Nominal unit shear capacity for		-							
Nominal unit shear capacity for	wind design	v _w = 1370 lb	/ft						
Apparent shear wall shear stiffne	-	Ga = 15 kips	s/in						
Loading details		D = 306 lb/ft							
Dead load acting on top of panel Roof live load acting on top of pane	I	D = 300 ID/It Lr = 408 Ib/ft							
Snow load acting on top of panel			S = 611.25 lb/ft						
Self weight of panel			$S_{wt} = 12 \text{ lb/ft}^2$						
In plane seismic load acting at head	of nanel	$E_q = 1257$ lbs							
Design spectral response accel. par	-								
From IBC 2018 cl.1605.2	, i								
Load combination no.1		1 2D + 1 6(L	or S or R) + 0	5W					
Load combination no.2		-).5Lf + 0.5(Lr 0						
Load combination no.3		1.2D + E + 0	-	,					
Load combination no.4		0.9D + W							
Load combination no.5		0.9D + E							
Adjustment factors									
Format conversion factor for tension	n – Table N1								
		KFt = 2.70							
Format conversion factor for compre	ession – Tab	le N1							
		KFc = 2.40							
Format conversion factor for module	us of elasticity								
		Kfe = 1.76							
Resistance factor for tension – Tab		$\phi t = 0.80$							
Resistance factor for compression -		$\phi_c = 0.90$	$\phi_{\rm c} = 0.90$						
Resistance factor for modulus of ela	sticity – Tab								
		$\phi_s = 0.85$							
Time effect factor – Table N3		$\lambda = 1.00$							
Sheathing resistance factor		φ D = 0.80							
Size factor for tension – Table 4A		CFt = 1.30							
Size factor for compression – Table		CFc = 1.10							
Wet service factor for tension – Tak		C _{Mt} = 1.00							
Wet service factor for compression		C _{Mc} = 1.00							
Wet service factor for modulus of el	asticity – Tab	ole 4A							

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	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date		
		Сме = 1.00	•					
Temperature factor for tension	– Table 2.3.3	Ctt = 1.00						
Temperature factor for compres	sion – Table 2.3.3							
Temperature factor for modulus	of elasticity – Tab	$C_{tc} = 1.00$						
		CtE = 1.00						
Incising factor – cl.4.3.8		Ci = 1.00						
Buckling stiffness factor – cl.4.4	1.2	C⊤ = 1.00						
Adjusted modulus of elasticity		Emin' = Emin *	Кге * фs * Сме *	CtE * Ci * CT = 8	370000 psi			
Critical buckling design value		Fce = 0.822	× E _{min} ' / (h / d)²	= 1665 psi				
Reference compression design	value			$C_{tc} * C_{Fc} * C_i = 3$	3208 psi			
For sawn lumber		c = 0.8						
Column stability factor - eqn	.3.7-1	C _P = (1 + (F	= _{cE} / Fc*)) / (2 >	× c) – √([(1 + (F	FcE / Fc*)) / (2 ×	c)] ² - (F _{cE}		
		Fc*) / C) = 0.45						
From SDPWS Table 4.3.4 Max	kimum Shear Wall	Aspect Ratio	S					
Maximum shear wall aspect rat		3.5						
Shear wall length		b = 5.146 ft						
Shear wall aspect ratio		h / b = 1.846						
Segmented shear wall capaci	ty							
Maximum shear force under	seismic loading	$V_{s_max} = E_q$	= 1.257 kips					
Shear capacity for seismic lo	bading	Vs = φD * vs * b = 4.034 kips						
		$V_{s_max} / V_s =$	0.312					
		PASS - Shea	ar capacity for	seismic load e	exceeds maximu	um shear fo		
Chord capacity for chords 1 a	and 2							
Shear wall aspect ratio		h / b = 1.846	5					
Load combination 5								
Load combination 5 Shear force for maximum tension		V = E _q = 1.2	57 kips					
Load combination 5 Shear force for maximum tension Axial force for maximum tension	sion	P = 0 kips =	57 kips = 0 kips	inc				
Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord	sion	P = 0 kips = T = V * h / (t	57 kips = 0 kips b) - P = 2.321 ki	ips				
Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress	sion	P = 0 kips = T = V * h / (b ft = T / A _{en} =	57 kips = 0 kips b) - P = 2.321 ki 172 lb/in ²		1 5 lb/in²			
Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord	sion	P = 0 kips = T = V * h / (t ft = T / A _{en} = Ft' = Ft * K _{Ft}	57 kips = 0 kips b) - Ρ = 2.321 ki 172 lb/in ² * φt * λ * Cмt * C	ips Stt * CFt * Ci = 161	15 lb/in²			
Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress	sion	P = 0 kips = T = V * h / (k + 1) kip = 0 ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.10	57 kips = 0 kips b) - Ρ = 2.321 ki 172 lb/in ² * φt * λ * C _{Mt} * C 6	Ctt * CFt * Ci = 161	15 lb/in² aximum appliec	I tensile str		
Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress	sion	P = 0 kips = T = V * h / (k + 1) kip = 0 ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.10	57 kips = 0 kips b) - Ρ = 2.321 ki 172 lb/in ² * φt * λ * C _{Mt} * C 6	Ctt * CFt * Ci = 161		I tensile str		
Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress	sion	P = 0 kips = T = V * h / (k + 1) kip = 0 ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.10	57 kips = 0 kips b) - P = 2.321 ki 172 lb/in ² * φt * λ * CMt * C 6 ign tensile stre	Ctt * CFt * Ci = 161		I tensile str		
Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress	ression	$P = 0 \text{ kips} = T = V * h / (b + ft) = T / A_{en} = Ft' = Ft * K_{Ft}$ $ft / Ft' = 0.10$ $PASS - Des$ $V = E_q = 1.2$	57 kips = 0 kips b) - P = 2.321 ki 172 lb/in ² * φt * λ * C _{Mt} * C 6 ign tensile stre 57 kips	ess exceeds ma				
Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp	ression	$P = 0 \text{ kips} = T = V * h / (b + ft) = T / A_{en} = Ft' = Ft * K_{Ft}$ $ft / Ft' = 0.10$ $PASS - Des$ $V = E_q = 1.2$	57 kips = 0 kips b) - P = 2.321 ki 172 lb/in ² * φt * λ * CMt * C 6 ign tensile stre 57 kips D + Swt * h) + t	ess exceeds ma	aximum appliec			
Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp	ression npression	P = 0 kips = T = V * h / (k + 1) + (k +	57 kips = 0 kips b) - P = 2.321 ki 172 lb/in ² * φt * λ * CMt * C 6 ign tensile stre 57 kips D + Swt * h) + t	ett * CFt * Ci = 161 ess exceeds ma 0.2 * Sps * (D +	aximum appliec			
Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp Axial force for maximum comp	ression npression chord	P = 0 kips = T = V * h / (b + b) + (b +	57 kips = 0 kips b) - P = 2.321 ki 172 lb/in ² * φt * λ * CMt * C 6 ign tensile stre 57 kips D + Swt * h) + t s b) + P = 2.995 k 181 lb/in ²	ess exceeds ma 0.2 * S _{DS} * (D + kips	aximum appliec + Swt * h) + + 0			
Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp Axial force for maximum comp Maximum compressive force in	ression npression chord	P = 0 kips = T = V * h / (b + b) + (b +	57 kips = 0 kips b) - P = 2.321 ki 172 lb/in ² * φt * λ * CMt * C 6 ign tensile stre 57 kips D + Swt * h) + t s b) + P = 2.995 k 181 lb/in ²	ett * CFt * Ci = 161 ess exceeds ma 0.2 * Sps * (D +	aximum appliec + Swt * h) + + 0			

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Hold down force	
Chord 1	T ₁ = 2.321 kips
Chord 2	T ₂ = 2.321 kips
Seismic deflection	
Design shear force	$V_{\delta s} = E_q = 1.257 \text{ kips}$
Deflection limit	$\Delta_{s_allow} = 0.020 * h = 2.28$ in
Induced unit shear	$v_{\delta s} = V_{\delta s} / b = 244.28 \text{ lb/ft}$
Anchor tension force	T _δ = max(0 kips,v _{δs} * 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	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	δ_{sws} = Cdô * δ_{swse} / Ie = 1.239 in
	$\delta_{sws} / \Delta_{s_allow} = 0.544$
	PASS - Shear wall deflection is less than deflection limit



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Brooklyn, NY 11217 Wood Shear V		ar Wall - Supp. Hi	gh Roof Wall 4	2				
	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date		
	ł	l	ł	ļ				
From NDS Supplement Table	4A - Reference	e design values f	or visually gra	ded dimensio	on lumber (2" - 4	" thick)		
Species, grade and size classifi	cation	Douglas Fir-	Larch, no.2 gra	de, 2" & wider				
Specific gravity		G = 0.50						
Tension parallel to grain		Ft = 575 lb/in						
Compression parallel to grain		Fc = 1350 lb						
Modulus of elasticity		E = 160000						
Minimum modulus of elasticity		Emin = 58000	10 lb/in ²					
Sheathing details								
Sheathing material			d panel 3-ply		athing			
Fastener type		8d commoi	n nails at 3"ce	nters				
From SDPWS Table 4.3A Non	ninal Unit Shea	r Capacities for V	Nood-Frame S	hear Walls - \	Nood-based Pa	nels		
Nominal unit shear capacity	for seismic de	sign vs = 980 lb/	ft					
Nominal unit shear capacity	for wind desig	n v _w = 1370 lk	o/ft					
Apparent shear wall shear st	tiffness	Ga = 15 kip	s/in					
Loading details								
Dead load acting on top of pane	el	D = 276.25	D = 276.25 lb/ft					
Roof live load acting on top of p	anel	Lr = 369 lb/ft						
Snow load acting on top of panel	el	S = 553 lb/ft						
Self weight of panel		S _{wt} = 12 lb/ft	2					
In plane seismic load acting at h	-		E _q = 4000 lbs					
Design spectral response accel	. par., short peri	ods Sps = 0.944	s Sds = 0.944					
From IBC 2018 cl.1605.2								
Load combination no.1		-	1.2D + 1.6(Lr or S 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 Load combination no.5			0.9D + W 0.9D + E					
		0.9D + E						
Adjustment factors		4						
Format conversion factor for ter	ISION - Table N	1 KFt = 2.70						
Format conversion factor for co	mpression – Ta							
		KFc = 2.40						
Format conversion factor for mo	odulus of elastici							
		KFE = 1.76						
Resistance factor for tension -	Table N2	$\phi_t = 0.80$						
Resistance factor for compressi	ion – Table N2	$\phi_c = 0.90$						
Resistance factor for modulus of		ble N2						
		φs = 0.85						
Time effect factor – Table N3		$\lambda = 1.00$						
Sheathing resistance factor		φD = 0.80						
-		φ _D = 0.80 C _{Ft} = 1.30						
Size factor for tension – Table 4	Size factor for tension – Table 4A							

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		Wall - Supp. Hi	gh Roof Wall 4	3	3			
	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date		
Wet service factor for compressi	on – Table 4A	Смс = 1.00						
Wet service factor for modulus o	f elasticity – Tab	ole 4A						
		Cme = 1.00						
Temperature factor for tension -		Ctt = 1.00						
Temperature factor for compress	sion – Table 2.3.							
-	6 H 10 H T	$C_{tc} = 1.00$						
Temperature factor for modulus	of elasticity – La	ble 2.3.3 Ct∈ = 1.00						
Incising factor – cl.4.3.8		CtE = 1.00 Ci = 1.00						
Buckling stiffness factor – cl.4.4.	2	C⊤ = 1.00						
Adjusted modulus of elasticity	2		Кге * фs * Сме *	Сте * Сі * Ст =	870000 psi			
Critical buckling design value			× Emin' / (h / d) ²					
Reference compression design v	value		∝ tennin / (11 / u) → c*φc*λ*CMc*	•	3208 pei			
For sawn lumber	alue	C = 0.8	ε ψε λ Ome		5200 psi			
	3 7-1		Ξ _{οΓ} / Ε _ο *)) / (2 \	$(c) = \sqrt{(1/1 + (1))}$	Έ _{οΓ} / Ε _ο *)) / (2 ·	√ c)12 - (F		
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 Maxi		-	S					
Maximum shear wall aspect ratio)	3.5						
Shear wall length		b = 17.333 f	b = 17.333 h h / b = 0.548					
Shear wall aspect ratio		n / D = 0.540	•					
Segmented shear wall capacit	-		4 I.:					
Maximum shear force under	-	-	•					
Shear capacity for seismic loa	ading	$V_{s} = \phi_{D} * v_{s} * b = $ 13.589 kips						
		$V_{s_max} / V_s =$						
		PASS - She	ar capacity for	seismic load e	exceeds maxim	ium snear to		
Chord capacity for chords 1 a	nd 2							
Shear wall aspect ratio		h / b = 0.548	5					
Load combination 5 Shear force for maximum tensio	2		ino					
Axial force for maximum tension		V = Eq = 4 kips P = 0 kips = 0 kips						
Maximum tensile force in chord			•	D C				
Maximum applied tensile stress		-	T = V * h / (b) - P = 2.192 kips ft = T / A _{en} = 162 lb/in ²					
Design tensile stress		$F_{t} = F_{t} * K_{Ft} * \phi_{t} * \lambda * C_{Mt} * C_{Ft} * C_{Ft} * C_{i} = 1615 \text{ lb/in}^{2}$						
		$f_t / F_t' = 0.101$						
		PASS - Design tensile stress exceeds maximum applied tensile stre						
Load combination 3			-					
Shear force for maximum compr	ession	$V = E_q = 4$ kips						
Axial force for maximum compression		P = (1.2 * (D + S _{wt} * h) + (0.2 * SDS * (D	+ S _{wt} * h) + +	0.7 * S) * s		
		= 0.619 kips						
	shord		o) + P = 2.812 k	tips				
Maximum compressive force in o	noru	$f_c = C / A_e = 170 \text{ lb/in}^2$						
Maximum compressive force in o Maximum applied compressive s		$f_c = C / A_e =$	170 lb/in ²					
-				Ctc * CFc * Ci * (CP = 1433 lb/in ²			

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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	$V_{\delta s} = E_q = 4 \text{ kips}$
Deflection limit	$\Delta_{s_{allow}} = 0.020 * h = 2.28 in$
Induced unit shear	v _{õs} = V _{õs} / b = 230.77 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * 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	le = 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

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	Calc. by	ar Wall - Supp. Hig	Chk'd by	Date	1 App'd by	Date
	BJW	2/19/2021				
WOOD SHEAR WALL DESIGN			14			
In accordance with NDS2018 I	bad reduction	factor design an	d the segment	ed shear wa		culation version
Panel details						
Structural wood panel sheathing Panel height	on one side	h = 9.5 ft				
Panel length		b = 5.917 ft				
Total area of wall		A = h * b = 5	6.209 ft ²			
		E,				
						
	- 9					
	6-					
	<u> </u>	Ch1	<u> </u>	Ch2		
		312 kips				
		2		× 1		
		2.3		N N		
		5	11"			
		- 0				
Panel construction						
Nominal stud size		2" x 6"				
Dressed stud size		1.5" x 5.5"				
Cross-sectional area of studs		As = 8.25 in ²	1			
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 post	S	A _e = 16.5 in ²	2			
Hole diameter		Dia = 1 in	_			
Net cross-sectional area of end	oosts	A _{en} = 13.5 in	2			
Nominal collector size		2 x 2" x 6"	="			
Dressed collector size Service condition		2 x 1.5" x 5.5	C			
Temperature		Dry 100 degF or	000			
Vertical anchor stiffness		ka = 80000 ll				

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Brooklyn, NY 11217	Wood Shea	r Wall - Supp. Hiç	Vall - Supp. High Roof Wall 5					
	Calc. by BJW	Date 2/19/2021	Chk'd by	Date	App'd by	Date		
From NDS Supplement Table	44 - Reference	design values fr	or visually gra	ded dimensio	n lumber (2" - 4	" thick)		
Species, grade and size classif		-	Larch, no.2 gra		•	, mony		
Specific gravity		G = 0.50						
Tension parallel to grain		Ft = 575 lb/in	2					
Compression parallel to grain		Fc = 1350 lb/	′in²					
Modulus of elasticity		E = 160000	lb/in ²					
Minimum modulus of elasticity		Emin = 58000	0 lb/in ²					
Sheathing details								
Sheathing material		15/32" woo	d panel 3-ply	plywood shea	athing			
Fastener type		8d commor	nails at 3"ce	enters				
From SDPWS Table 4.3A Non		-		Shear Walls - W	Vood-based Pa	nels		
Nominal unit shear capacity		•						
Nominal unit shear capacity	•							
Apparent shear wall shear s	tiffness	Ga = 15 kips	s/in					
Loading details								
Self weight of panel		$S_{wt} = 12 \text{ lb/ft}^2$						
In plane seismic load acting at		Eq = 1440 lbs	S					
Design spectral response acce	I. par., short perio	ods Sps = 0.944						
From IBC 2018 cl.1605.2			0					
Load combination no.1		-	1.2D + 1.6(Lr or S 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 Load combination no.4			1.2D + E + 0.5Lf + 0.7S 0.9D + W					
Load combination no.5			0.9D + E					
		0.9D + E						
Adjustment factors Format conversion factor for ter	nsion – Table N1							
		KFt = 2.70						
Format conversion factor for co	mpression – Tab							
		KFc = 2.40						
Format conversion factor for me	odulus of elasticit	ty – Table N1						
		Kfe = 1.76						
Resistance factor for tension -	Table N2	$\varphi t = \textbf{0.80}$						
Resistance factor for compress	ion – Table N2	$\phi_c = 0.90$						
Resistance factor for modulus of	of elasticity – Tab	ble N2						
		φs = 0.85						
Time effect factor – Table N3		$\lambda = 1.00$						
Sheathing resistance factor		φ D = 0.80						
Size factor for tension – Table	4A	CFt = 1.30						
Size factor for compression – T	able 4A	CFc = 1.10						
Wet service factor for tension -		C _{Mt} = 1.00						
Wet service factor for compress	sion – Table 4A	Смс = 1.00						

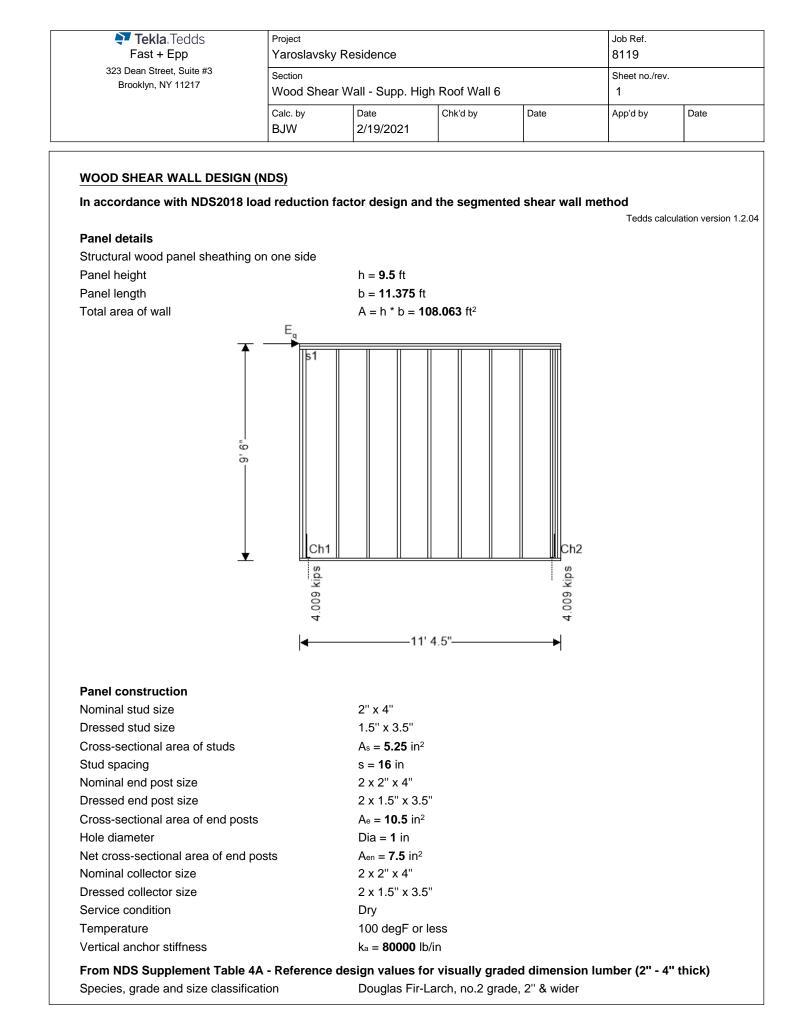
Tekla Tedds Fast + Epp	Project Yaroslavsk	y Residence		Job Ref. 8119					
323 Dean Street, Suite #3 Brooklyn, NY 11217 Wood Shear W					Sheet no./rev				
		ear Wall - Supp. High Roof Wall 5 3							
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Temperature factor for tension -	- Table 2.3.3	Ctt = 1.00			·	•			
Temperature factor for compres	sion – Table 2.	3.3							
		Ctc = 1.00							
Temperature factor for modulus	of elasticity - T	able 2.3.3							
		CtE = 1.00							
Incising factor – cl.4.3.8	_	Ci = 1.00							
Buckling stiffness factor – cl.4.4	.2	CT = 1.00							
Adjusted modulus of elasticity			′ Кге * фs * Сме *		870000 psi				
Critical buckling design value		$F_{cE} = 0.822$	\times E _{min} ' / (h / d) ²	= 1665 psi					
Reference compression design	value	$F_{c}^{*} = F_{c} * K_{F}$	с* фс*λ*Смс*	$C_{tc} * C_{Fc} * C_i =$	3208 psi				
For sawn lumber		c = 0.8							
Column stability factor - eqn	.3.7-1	CP = (1 + (FcE / Fc*)) / (2 :	× c) – √([(1 +	(FcE / Fc*)) / (2	× c)] ² - (F _{CE}			
		Fc*) / C) = 0	Fc*) / C) = 0.45						
From SDPWS Table 4.3.4 Max	imum Shear W	all Aspect Ratio	S						
Maximum shear wall aspect rati		3.5							
Shear wall length		b = 5.917 ft	b = 5.917 ft						
Shear wall aspect ratio		h / b = 1.60	h / b = 1.606						
Segmented shear wall capaci	tv								
Maximum shear force under	-	ng Vs_max = Eq	= 1.44 kips						
Shear capacity for seismic lo		-	$V_{s} = \phi_{D} * v_{s} * b = 4.639$ kips						
	aang	Vs_max / Vs =							
				seismic load	exceeds maxin	num shear i			
Chord capacity for chords 1 a	ind 2								
Shear wall aspect ratio		h / b = 1.60	6						
Load combination 5									
Shear force for maximum tension	on	V = Eq = 1.4	V = E _q = 1.44 kips						
Axial force for maximum tens	sion	P = 0 kips	P = 0 kips = 0 kips						
Maximum tensile force in chord		•	b) - P = 2.312 k	ips					
Maximum applied tensile stress			$f_t = T / A_{en} = 171 \text{ lb/in}^2$						
Design tensile stress			$F_t' = F_t * K_{Ft} * \varphi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i = 1615 \text{ lb/in}^2$						
			ft / Ft' = 0.106						
Landarsech' (* C		PASS - Des	sign tensile stre	ess exceeds n	naximum applie	d tensile s			
Load combination 3			Aling						
Shear force for maximum comp			$V = E_q = 1.44$ kips $P = (1.2 + S_{10} + h_{10}, 2 + S_{20} + S_{10} + h_{10}) + c_1(2 - 0.106$ kips						
Axial force for maximum con	•	$P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2 = 0.106 \text{ kips}$							
Maximum compressive force in			C = V * h / (b) + P = 2.418 kips f _c = C / A _e = 147 lb/in ²						
Maximum applied compressive stress Design compressive stress				C. * C- * C *					
		$F_{c}' = F_{c} * K_{Fc} * \phi_{c} * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_{i} * C_{P} = 1433 \text{ lb/in}^{2}$							
	PASS -	fc / Fc' = 0.1 Design compres		ceeds maximi	um applied com	pressive s			
Hold down force					111-11-0-001	,			
Chord 1		T₁ = 2.312 k	kips						
Chord 2			$T_1 = 2.312$ kips $T_2 = 2.312$ kips						

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Seismic deflection

Design shear force	$V_{\delta s} = E_q = 1.44 \text{ kips}$
Deflection limit	$\Delta_{s_allow} = 0.020 * h = 2.28$ in
Induced unit shear	v _{õs} = V _{õs} / b = 243.38 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * 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	le = 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



Fast + Epp	Project Yaroslavsky F	Residence			Job Ref. 8119			
323 Dean Street, Suite #3	Section				Sheet no./rev	·.		
Brooklyn, NY 11217	Wood Shear \	Nall - Supp. Hig	gh Roof Wall 6	2	2			
	Calc. by BJW	Date 2/19/2021	Chk'd by	App'd by	Date			
Specific gravity		G = 0.50						
Tension parallel to grain		Ft = 575 lb/ir	1 ²					
Compression parallel to grain		Fc = 1350 lb/	/in²					
Modulus of elasticity		E = 160000) lb/in²					
Minimum modulus of elasticity		Emin = 58000	0 lb/in ²					
Sheathing details								
Sheathing material		15/32'' woo	d panel 3-ply	plywood she	athing			
Fastener type		8d commor	n nails at 3"ce	enters				
From SDPWS Table 4.3A Nor	ninal Unit Shear C	apacities for V	Nood-Frame S	Shear Walls - V	Nood-based Pa	nels		
Nominal unit shear capacity		•						
Nominal unit shear capacity		Vw = 1370 lb						
Apparent shear wall shear st	•	Ga = 15 kip:						
			6,111					
Loading details			0					
Self weight of panel		Swt = 12 lb/ft						
In plane seismic load acting at h	-	Eq = 4800 lb	S					
Design spectral response accel	. par., snort period	S 5DS = 0.944						
From IBC 2018 cl.1605.2								
Load combination no.1			r or S or R) + 0					
Load combination no.2		1.2D + W + 0.5Lf + 0.5(Lr or S or R) 1.2D + E + 0.5Lf + 0.7S						
Load combination no.3 Load combination no.4		0.9D + W						
Load combination no.4		0.9D + E						
		0.30 + L						
Adjustment factors	aion Tabla Nd							
Format conversion factor for ter	ISION - LADIE N1	KFt = 2.70						
Format conversion factor for con	moression - Table							
		KFc = 2.40						
Format conversion factor for mo	odulus of elasticity							
		KFE = 1.76						
Resistance factor for tension –	Table N2	$\phi_{t} = 0.80$						
Resistance factor for compressi	ion – Table N2	$\phi_{\rm c} = 0.90$						
Resistance factor for modulus o								
	5	φs = 0.85						
Time effect factor – Table N3		λ = 1.00						
Sheathing resistance factor		φD = 0.80						
Size factor for tension – Table 4	1A	CFt = 1.50						
Size factor for compression – T		CFc = 1.15						
Wet service factor for tension –		C _{Mt} = 1.00						
Wet service factor for compress	ion – Table 4A	Смс = 1.00						
Wet service factor for modulus of	of elasticity – Table	e 4A						
		Сме = 1.00						

Fast + Epp	Project Yaroslavsl	ky Residence			Job Ref. 8119					
323 Dean Street, Suite #3	Section				Sheet no./rev.					
Brooklyn, NY 11217		ar Wall - Supp. H	igh Roof Wall 6		3					
	Calc. by BJW	Date 2/19/2021	Chk'd by	Date	App'd by	Date				
		Ctc = 1.00								
Temperature factor for modulus	of elasticity -	Table 2.3.3								
		CtE = 1.00								
Incising factor – cl.4.3.8		Ci = 1.00								
Buckling stiffness factor – cl.4.4	.2	C⊤ = 1.00								
Adjusted modulus of elasticity		Emin' = Emin	* Кге * фѕ * Сме *	* CtE * Ci * Cт =	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 * фс * λ * Смс *	$C_{tc} * C_{Fc} * C_i =$	- 3353 psi					
For sawn lumber		c = 0.8								
Column stability factor - eqn.	.3.7-1	C _P = (1 + ((F _{cE} / F _c *)) / (2	× c) – √([(1 +	(F _{cE} / F _c *)) / (2 >	< c)]² - (F c∈				
		Fc*) / C) = (0.19							
From SDPWS Table 4.3.4 Max	imum Shear V	Vall Aspect Ratio	os							
Maximum shear wall aspect ration	D	3.5	-							
Shear wall length		b = 11.375	ft							
Shear wall aspect ratio		h / b = 0.83	5							
Segmented shear wall capacit	y									
Maximum shear force under	seismic loadi	ng Vs_max = Ed	$V_{s_{max}} = E_{q} = 4.8 \text{ kips}$							
Shear capacity for seismic lo	ading	Vs = \$\$D * V	Vs = \phi_D * vs * b = 8.918 kips							
		Vs_max / Vs =	- 0.538							
		PASS - She	ear capacity for	seismic load	exceeds maxim	um shear f				
Chord capacity for chords 1 a	nd 2									
Shear wall aspect ratio		h / b = 0.83	5							
Load combination 5										
Shear force for maximum tensio			$V = E_q = 4.8 \text{ kips}$							
Axial force for maximum tens	sion	•	P = 0 kips = 0 kips							
Maximum tensile force in chord			T = V * h / (b) - P = 4.009 kips							
Maximum applied tensile stress			$f_t = T / A_{en} = 535 \text{ lb/in}^2$							
Design tensile stress			$F_{t}' = F_{t} * K_{Ft} * \phi_{t} * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_{i} = 1863 \text{ lb/in}^{2}$							
			ft / Ft' = 0.287 PASS - Design tensile stress exceeds maximum applied tensile stre							
Load combination 3		r A33 - D6	ราฐาา เอาราเฮ รแ	USS EXLEEUS I		u iensiie si				
Shear force for maximum compl	ression	V = E _q = 4.	3 kips							
Axial force for maximum com			$P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2 = 0.106 kips$							
Maximum compressive force in			(b) + P = 4.114			•				
Maximum applied compressive			$f_c = C / A_e = 392 \text{ lb/in}^2$							
Design compressive stress		Fc' = Fc * Kr	тс*фс*λ*Смс*	Ctc * CFc * Ci *	C _P = 644 lb/in ²					
· ·		fc / Fc' = 0.6								
	PASS -	Design compre	ssive stress ex	ceeds maxim	um applied com	pressive st				
Hold down force										
Chord 1	1			T ₁ = 4.009 kips						
Chord 2		T ₂ = 4.009	kips							
Seismic deflection										

Tekla Tedds Fast + Epp	Project Yaroslavsky Re	Job Ref. 8119				
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Wood Shear Wall - Supp. High Roof Wall 6					
	,	Date 2/19/2021	Chk'd by	Date	App'd by	Date

Deflection limit	$\Delta_{s_{allow}} = 0.020 * h = 2.28$ in
Induced unit shear	νδs = Vδs / b = 421.98 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * h) = 4.009 kips
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_\delta / (k_a * b) = 0.324$ in
Deflection ampification factor	$C_{d\delta} = 4$
Seismic importance factor	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e = 1.297$ in
	$\delta_{sws} / \Delta_{s_allow} = 0.569$
	DASS. Shear wall deflection is less than deflection limit

PASS - Shear wall deflection is less than deflection limit

Tekla Tedds Fast + Epp	Project Yaroslavsł	ky Residence			Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Sheet no./rev. Wood Shear Wall - Supp. High Roof Wall 6 1					
	Calc. by BJW	Date 2/19/2021	Chk'd by	Date	App'd by	Date
	Dow	2/10/2021				
WOOD SHEAR WALL DESIGN	<u>(NDS)</u>					
In accordance with NDS2018 lo	oad reduction	factor design an	d the segmente	ed shear wall		
Panel details					l edds cald	culation version
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 = 7	6 ft ²			
	E	a				
			<u> </u>	╗		
	-					
	9.0 "0					
	Ĩ					
		Ch1		Ch2		
	<u> </u>			- -		
		kipe		19 kips		
		99 kips		66		
		(r)		τ, σ		
		I	8'			
			0			
Panel construction						
Nominal stud size		2" x 6"				
Dressed stud size		1.5" x 5.5"				
Cross-sectional area of studs		As = 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	5"			
Cross-sectional area of end post	S	A _e = 16.5 in ²				
Hole diameter		Dia = 1 in				
Net cross-sectional area of end p	oosts	A _{en} = 13.5 in	2			
Nominal collector size		2 x 2" x 6"				
Dressed collector size		2 x 1.5" x 5.5	5"			
Service condition		Dry				
Temperature		100 degF or	less			
remperature						
Vertical anchor stiffness		ka = 80000 lk				

Tekla Tedds Fast + Epp	Project Yaroslavsky R	esidence			Job Ref. 8119			
323 Dean Street, Suite #3	Section				Sheet no./rev	·.		
Brooklyn, NY 11217	Wood Shear W	/all - Supp. Hig	2					
	Calc. by BJW	Date 2/19/2021	Chk'd by	Date	App'd by	Date		
Specific gravity		G = 0.50						
Tension parallel to grain		Ft = 575 lb/in	2					
Compression parallel to grain		Fc = 1350 lb/i	in²					
Modulus of elasticity		E = 1600000	lb/in ²					
Minimum modulus of elasticity		Emin = 58000	0 lb/in ²					
Sheathing details								
Sheathing material		15/32" wood	d panel 3-ply	plywood she	athing			
Fastener type		8d common	nails at 3"ce	nters				
From SDPWS Table 4.3A Nom	inal Unit Shear Ca	apacities for W	Vood-Frame S	hear Walls - \	Nood-based Pa	nels		
Nominal unit shear capacity	for seismic desigr	n vs = 980 lb/ft	t					
Nominal unit shear capacity	for wind design	v _w = 1370 lb,	/ft					
Apparent shear wall shear st	iffness	Ga = 15 kips	/in					
Loading details								
Self weight of panel		$S_{wt} = 12 \text{ lb/ft}^2$						
In plane seismic load acting at h	lead of panel	$E_q = 3360 \text{ lbs}$						
Design spectral response accel	-	SDS = 0.944						
From IBC 2018 cl.1605.2								
Load combination no.1		1.2D + 1.6(Lr	or S or R) + 0	.5W				
Load combination no.2		1.2D + W + 0	0.5Lf + 0.5(Lr o	r S or R)				
Load combination no.3		1.2D + E + 0.5Lf + 0.7S						
Load combination no.4		0.9D + W						
Load combination no.5		0.9D + E						
Adjustment factors								
Format conversion factor for ten	sion – Table N1							
		KFt = 2.70						
Format conversion factor for con	mpression – Table							
Format conversion factor for mo	dulus of elasticity	KFc = 2.40						
		KFE = 1.76						
Resistance factor for tension -	Table N2	$\phi_t = 0.80$						
Resistance factor for compressi		$\Phi_{\rm c}=0.90$						
Resistance factor for modulus o		1						
	,	φs = 0.85						
Time effect factor – Table N3		λ = 1.00						
Sheathing resistance factor		φ D = 0.80						
Size factor for tension – Table 4	A	CFt = 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 compress		C _{Mc} = 1.00						
Wet service factor for modulus of	of elasticity – Table							
		Сме = 1.00						
Temperature factor for tension -		Ctt = 1.00						
Temperature factor for compres	sion – Table 2.3.3							

Tekla Tedds Fast + Epp	Project Yaroslavsk	y Residence			Job Ref. 8119				
323 Dean Street, Suite #3	Section					Sheet no./rev.			
Brooklyn, NY 11217		ar Wall - Supp. H	igh Roof Wall 6		3				
	Calc. by BJW	Date 2/19/2021	Chk'd by	Date	App'd by	Date			
		Ctc = 1.00							
Temperature factor for modulus	of elasticity -	Fable 2.3.3							
		CtE = 1.00							
Incising factor – cl.4.3.8		Ci = 1.00							
Buckling stiffness factor - cl.4.4	.2	C⊤ = 1.00							
Adjusted modulus of elasticity			* Кге * фѕ * Сме *		870000 psi				
Critical buckling design value			\times E _{min} ' / (h / d) ²	-					
Reference compression design	value		-с*фс*λ*Смс*	$C_{tc} * C_{Fc} * C_i =$	= 3208 psi				
For sawn lumber		c = 0.8		1					
Column stability factor – eqn.	3.7-1		<i>,,</i> , , , , , , , , , , , , , , , , , ,	× c) – √([(1 +	(F _{cE} / F _c *)) / (2 >	< C)] ² - (FcE			
		Fc*) / C) = ().45						
From SDPWS Table 4.3.4 Maxi		•	S						
Maximum shear wall aspect ratio	0	3.5							
Shear wall length		b = 8 ft							
Shear wall aspect ratio		h / b = 1.18	h / b = 1.188						
Segmented shear wall capacit	-	–							
Maximum shear force under		0	= 3.36 kips						
Shear capacity for seismic lo	ading	$V_s = \phi_D * v_s$	$V_{s} = \phi_{D} * v_{s} * b = 6.272$ kips						
		Vs_max / Vs =							
		PASS - She	ear capacity for	seismic load	exceeds maxim	um shear fo			
Chord capacity for chords 1 a	nd 2								
Shear wall aspect ratio		h / b = 1.18	8						
Load combination 5	_								
Shear force for maximum tensio			$V = E_q = 3.36 \text{ kips}$						
Axial force for maximum tens	ION	•	P = 0 kips = 0 kips						
Maximum tensile force in chord			T = V * h / (b) - P = 3.990 kips ft = T / A _{en} = 296 lb/in ²						
Maximum applied tensile stress			Tt = T / Aen = 296 ID/IN ² Ft' = Ft * KFt * φt * λ * CMt * Ctt * CFt * Ci = 1615 Ib/in ²						
Design tensile stress			$F_{t} = F_{t}^{*} K_{Ft}^{*} \phi_{t}^{*} \lambda^{*} C_{Mt}^{*} C_{Ft}^{*} C_{Ft}^{*} G = 1613 \text{ id}/\text{In}^{2}$ $f_{t} / F_{t}^{*} = 0.183$						
			-	ess exceeds n	naximum applie	d tensile st			
Load combination 3			agin tonone oth						
Shear force for maximum compr	ession	V = Eq = 3. 3	86 kips						
Axial force for maximum com			•	SDS * Swt * h) *	* s / 2 = 0.106 ki	ps			
Maximum compressive force in o	-		b) + P = 4.096	,					
Maximum applied compressive s		fc = C / Ae =							
Design compressive stress		Fc' = Fc * KFc * φc * λ * CMc * Ctc * CFc * Ci * CP = 1433 lb/in ²							
		fc / Fc' = 0.1	73						
	PASS -	Design compres	ssive stress ex	ceeds maximi	um applied com	pressive str			
Hold down force									
Chord 1		T1 = 3.99 ki	T ₁ = 3.99 kips						
Chord 2		T2 = 3.99 ki	ps						
Seismic deflection									
Design shear force		$V_{\delta s} = E_q = 0$	3.36 kips						

Tekla Tedds Fast + Epp	Project Yaroslavsky Re	sidence	Job Ref. 8119			
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Wood Shear Wall - Supp. High Roof Wall 6					
	Calc. by BJW	Date 2/19/2021	Chk'd by	Date	App'd by	Date

Deflection limit	$\Delta_{s_{allow}} = 0.020 * h = 2.28 in$
Induced unit shear	v _{ðs} = V _{ðs} / b = 420 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * h) = 3.990 kips
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_\delta / (k_a * b) = 0.339$ in
Deflection ampification factor	$C_{d\delta} = 4$
Seismic importance factor	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	δ_{sws} = Cd δ * δ_{swse} / Ie = 1.355 in
	$\delta_{sws} / \Delta_{s_allow} = 0.595$
	DASS Shaar wall deflection is loss than deflection limit

PASS - Shear wall deflection is less than deflection limit

Fast + Epp	Project Job Yaroslavsky Residence 811					
Brooklyn NY 11217	Section Wood Shear Wall - Supp. Upper Level Wall 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

Panel details

Structural wood panel sheathing on one side

Panel height

Panel length Total area of wall

Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	As = 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	Ae = 16.5 in ²
Hole diameter	Dia = 1 in
Net cross-sectional area of end posts	Aen = 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	ka = 80000 lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)Species, grade and size classificationDouglas Fir-Larch, no.2 grade, 2" & wider

Tedds calculation version 1.2.04

Tekla Tedds Fast + Epp	Project Yaroslavsky Residence				Job Ref. 8119		
323 Dean Street, Suite #3	Section	tion Sheet no./re					
Brooklyn, NY 11217	Wood Shear W	/all - Supp. Upp	er Level Wall	1	2		
	Calc. by BJW	Date 2/19/2021	Chk'd by	Date	App'd by	Date	
Specific gravity		G = 0.50					
Tension parallel to grain		Ft = 575 lb/in ²	2				
Compression parallel to grain		Fc = 1350 lb/ii					
Modulus of elasticity		E = 160000					
Minimum modulus of elasticity		Emin = 580000) lb/in ²				
Sheathing details							
Sheathing material		15/32" wood	l panel 3-ply	plywood she	athing		
Fastener type		8d common	nails at 3"ce	enters			
From SDPWS Table 4.3A Nomin	al Unit Shear Ca	apacities for W	ood-Frame S	Shear Walls -	Wood-based Pa	nels	
Nominal unit shear capacity for		-					
Nominal unit shear capacity for	wind design	v _w = 1370 lb/	ft				
Apparent shear wall shear stiff	•	Ga = 15 kips/	/in				
Loading details							
Dead load acting on top of panel		D = 19 lb/ft					
Roof live load acting on top of panel	عا	$L_r = 13 \text{ lb/ft}$					
Snow load acting on top of panel		S = 19 lb/ft					
Self weight of panel		$S_{wt} = 12 \text{ lb/ft}^2$					
In plane seismic load acting at hea	d of panel	E _q = 2040 lbs					
Design spectral response accel. pa	-	SDS = 0.944					
From IBC 2018 cl.1605.2							
Load combination no.1		1.2D + 1.6(Lr	or S or R) + 0	.5W			
Load combination no.2		1.2D + W + 0.					
Load combination no.3		1.2D + E + 0.	5Lf + 0.7S				
Load combination no.4		0.9D + W					
Load combination no.5		0.9D + E					
Adjustment factors							
Augustinent lactors							
Format conversion factor for tension	on – Table N1						
•	on – Table N1	KFt = 2.70					
•		N1					
Format conversion factor for tension Format conversion factor for comp	ression – Table	N1 KFc = 2.40					
Format conversion factor for tension	ression – Table	N1 K _{Fc} = 2.40 - Table N1					
Format conversion factor for tension Format conversion factor for comp Format conversion factor for modu	ression – Table lus of elasticity -	N1 K _{Fc} = 2.40 - Table N1 K _{FE} = 1.76					
Format conversion factor for tension Format conversion factor for comp Format conversion factor for modu Resistance factor for tension – Ta	ression – Table lus of elasticity - ble N2	N1 K _{Fc} = 2.40 - Table N1 K _{FE} = 1.76 φt = 0.80					
Format conversion factor for tension Format conversion factor for comp Format conversion factor for modu Resistance factor for tension – Ta Resistance factor for compression	ression – Table lus of elasticity - ole N2 – Table N2	N1 $K_{Fc} = 2.40$ - Table N1 $K_{FE} = 1.76$ $\phi_t = 0.80$ $\phi_c = 0.90$					
Format conversion factor for tension Format conversion factor for comp Format conversion factor for modu Resistance factor for tension – Ta	ression – Table lus of elasticity - ole N2 – Table N2	N1 $K_{Fc} = 2.40$ - Table N1 $K_{FE} = 1.76$ $\phi_t = 0.80$ $\phi_c = 0.90$ N2					
Format conversion factor for tension Format conversion factor for comp Format conversion factor for modu Resistance factor for tension – Ta Resistance factor for compression Resistance factor for modulus of e	ression – Table lus of elasticity - ole N2 – Table N2	N1 $K_{Fc} = 2.40$ - Table N1 $K_{FE} = 1.76$ $\phi_t = 0.80$ $\phi_c = 0.90$ N2 $\phi_s = 0.85$					
Format conversion factor for tension Format conversion factor for comp Format conversion factor for modu Resistance factor for tension – Ta Resistance factor for compression Resistance factor for modulus of e Time effect factor – Table N3	ression – Table lus of elasticity - ole N2 – Table N2	N1 $K_{Fc} = 2.40$ - Table N1 $K_{FE} = 1.76$ $\phi_t = 0.80$ $\phi_c = 0.90$ N2 $\phi_s = 0.85$ $\lambda = 1.00$					
Format conversion factor for tension Format conversion factor for comp Format conversion factor for modu Resistance factor for tension – Ta Resistance factor for compression Resistance factor for modulus of e Time effect factor – Table N3 Sheathing resistance factor	ression – Table lus of elasticity - ole N2 – Table N2	N1 $K_{Fc} = 2.40$ - Table N1 $K_{FE} = 1.76$ $\phi_t = 0.80$ $\phi_c = 0.90$ N2 $\phi_s = 0.85$ $\lambda = 1.00$ $\phi_D = 0.80$					
Format conversion factor for tension Format conversion factor for comp Format conversion factor for modul Resistance factor for tension – Ta Resistance factor for compression Resistance factor for modulus of e Time effect factor – Table N3 Sheathing resistance factor Size factor for tension – Table 4A	ression – Table lus of elasticity - ole N2 – Table N2 lasticity – Table	N1 $K_{Fc} = 2.40$ - Table N1 $K_{FE} = 1.76$ $\phi_t = 0.80$ $\phi_c = 0.90$ N2 $\phi_s = 0.85$ $\lambda = 1.00$ $\phi_D = 0.80$ $C_{Ft} = 1.30$					
Format conversion factor for tension Format conversion factor for comp Format conversion factor for modul Resistance factor for tension – Ta Resistance factor for compression Resistance factor for modulus of e Time effect factor – Table N3 Sheathing resistance factor Size factor for tension – Table 4A Size factor for compression – Table 4A	ression – Table lus of elasticity - ole N2 – Table N2 lasticity – Table	N1 $K_{Fc} = 2.40$ - Table N1 $K_{FE} = 1.76$ $\phi_t = 0.80$ $\phi_c = 0.90$ N2 $\phi_s = 0.85$ $\lambda = 1.00$ $\phi_D = 0.80$ $C_{Fc} = 1.30$ $C_{Fc} = 1.10$					
Format conversion factor for tension Format conversion factor for comp Format conversion factor for modul Resistance factor for tension – Ta Resistance factor for compression Resistance factor for modulus of e Time effect factor – Table N3 Sheathing resistance factor Size factor for tension – Table 4A	ression – Table lus of elasticity - ole N2 – Table N2 lasticity – Table e 4A ible 4A	N1 $K_{Fc} = 2.40$ - Table N1 $K_{FE} = 1.76$ $\phi_t = 0.80$ $\phi_c = 0.90$ N2 $\phi_s = 0.85$ $\lambda = 1.00$ $\phi_D = 0.80$ $C_{Ft} = 1.30$					

Tekla Tedds Fast + Epp	,	Project Yaroslavsky Residence					
323 Dean Street, Suite #3	Section	Section					
Brooklyn, NY 11217	Wood Shear V	Vall - Supp. Up	1	3			
	Calc. by BJW	Date 2/19/2021	Chk'd by	Date	App'd by	Date	
		Сме = 1.00					
Temperature factor for tension	– Table 2.3.3	$C_{tt} = 1.00$					
Temperature factor for compres							
	of classicity. Tab	$C_{tc} = 1.00$					
Temperature factor for modulus	s of elasticity - Tabl	le ∠.3.3 Ct∈ = 1.00					
Incising factor – cl.4.3.8		CtE = 1.00 Ci = 1.00					
Buckling stiffness factor – cl.4.4	1.2	C⊤ = 1.00					
Adjusted modulus of elasticity			Кге * фs * Сме *	CtE * Ci * CT = 87	70000 psi		
Critical buckling design value			× Emin' / (h / d) ²				
Reference compression design	value		· · · ·	$C_{tc} * C_{Fc} * C_i = 3$	208 psi		
For sawn lumber		C = 0.8	φο το Οινιο				
Column stability factor – eqn			F _{cE} / Fc*)) / (2 >	< c) – √([(1 + (F		c)]² - (F _⊂ ⊧	
		$C_{P} = (1 + (F_{cE} / F_{c^*})) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_{c^*})) / (2 \times c)]^2 - (F_{cE} / F_{c^*}) / c)} = 0.33$					
		, ,					
From SDPWS Table 4.3.4 Max		-	5				
Maximum shear wall aspect rat	10	3.5 b = 4.125 ft					
Shear wall length Shear wall aspect ratio		b = 4.125 m h / b = 2.768	2				
-		11/ D = 2.700	•				
Segmented shear wall capaci	-	V _ E	- 2.04 kipo				
Maximum shear force under	0	Vs_max = Eq	-				
Shear capacity for seismic lo	bading			.125 * h / bs) =	2.924 KIPS		
		$V_{s_max} / V_s =$				un obser fo	
		PA32 - 206	al capacity for	seismic load ex	xceeus maximi	um snear to	
Chord capacity for chords 1 a	and 2						
Shear wall aspect ratio	and 2	h / b = 2.76 8	}				
Shear wall aspect ratio Load combination 5							
Shear wall aspect ratio Load combination 5 Shear force for maximum tensio	on	V = E _q = 2.0	4 kips				
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension	on sion	V = E _q = 2.0 P = 0 kips =	4 kips = 0 kips	25			
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord	on sion	V = E _q = 2.0 P = 0 kips : T = V * h / (b	4 kips = 0 kips)) - P = 5.646 ki	ps			
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress	on sion	$V = E_q = 2.0$ P = 0 kips = T = V * h / (t) $f_t = T / A_{en} =$	4 kips = 0 kips)) - P = 5.646 ki 418 lb/in ²		5 lh/in²		
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord	on sion	$V = E_q = 2.0$ P = 0 kips = T = V * h / (k ft = T / A_{en} = F_t' = F_t * K_{Ft}	4 kips = 0 kips)) - Ρ = 5.646 ki 418 lb/in ² * φt * λ * Cмt * C	ps tt * CFt * Ci = 161 5	5 lb/in²		
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress	on sion	$V = E_q = 2.0$ P = 0 kips = T = V * h / (k $f_t = T / A_{en} = $ $F_t' = F_t * K_{Ft}$ $f_t / F_t' = 0.25$	4 kips = 0 kips b) - P = 5.646 ki 418 lb/in ² * φt * λ * C _{Mt} * C 9	tt * CFt * Ci = 161		d tensile str	
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress	on sion	$V = E_q = 2.0$ P = 0 kips = T = V * h / (k $f_t = T / A_{en} = $ $F_t' = F_t * K_{Ft}$ $f_t / F_t' = 0.25$	4 kips = 0 kips b) - P = 5.646 ki 418 lb/in ² * φt * λ * C _{Mt} * C 9			d tensile str	
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tensile Maximum tensile force in chord Maximum applied tensile stress Design tensile stress	on sion	$V = E_q = 2.0$ P = 0 kips = T = V * h / (k ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.25	4 kips = 0 kips b) - P = 5.646 ki 418 lb/in ² * φι * λ * C _{Mt} * C 9 ign tensile stre	tt * CFt * Ci = 161		d tensile str	
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress	on sion s	$V = E_q = 2.0$ P = 0 kips = $T = V * h / (k_f = T / A_{en} = $ $F_t' = F_t * K_{Ft} = $ $f_t / F_t' = 0.25$ PASS - Des $V = E_q = 2.0$	4 kips = 0 kips b) - P = 5.646 ki 418 lb/in ² * φt * λ * C _{Mt} * C 9 ign tensile stre 4 kips	tt * CFt * Ci = 161	iximum applied		
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp	on sion s	$V = E_q = 2.0$ P = 0 kips = $T = V * h / (k_f = T / A_{en} = $ $F_t' = F_t * K_{Ft} = $ $f_t / F_t' = 0.25$ PASS - Des $V = E_q = 2.0$	4 kips = 0 kips b) - P = 5.646 ki 418 lb/in ² * φt * λ * CMt * C 9 ign tensile stre 4 kips D + Swt * h) + 0	tt * CFt * Ci = 161 ess exceeds ma	iximum applied		
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp	on sion s pression npression	$V = E_q = 2.0$ P = 0 kips = 1 T = V * h / (h + 1) + (h	4 kips = 0 kips b) - P = 5.646 ki 418 lb/in ² * φt * λ * CMt * C 9 ign tensile stre 4 kips D + Swt * h) + 0	tt * CFt * Ci = 161 ess exceeds ma 0.2 * Sps * (D +	iximum applied		
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp Axial force for maximum comp	on sion s pression npression chord	$V = E_q = 2.0$ P = 0 kips = 1 T = V * h / (h + 1) + (h	4 kips = 0 kips b) - P = 5.646 ki 418 lb/in ² * φt * λ * CMt * C 9 ign tensile stre 4 kips D + Swt * h) + f s b) + P = 5.799 k	tt * CFt * Ci = 161 ess exceeds ma 0.2 * Sps * (D +	iximum applied		
Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum comp Axial force for maximum comp Axial force for maximum comp	on sion s pression npression chord	$V = E_q = 2.0$ $P = 0 \text{ kips} =$ $T = V * h / (k$ $f_t = T / A_{en} =$ $F_t' = F_t * K_{Ft}$ $f_t / F_t' = 0.25$ $PASS - Des$ $V = E_q = 2.0$ $P = (1.2 * ($ $= 0.153 \text{ kips}$ $C = V * h / (k$ $f_c = C / A_e =$	4 kips = 0 kips b) - P = 5.646 ki 418 lb/in ² * φι * λ * CMι * C 9 ign tensile stre 4 kips D + Swι * h) + 0 5 b) + P = 5.799 k 351 lb/in ²	tt * CFt * Ci = 161 ess exceeds ma 0.2 * Sps * (D +	iximum applied ∙ Swt * h) + + C		

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Hold down force	
Chord 1	T1 = 5.646 kips
Chord 2	T2 = 5.646 kips
Seismic deflection	
Design shear force	$V_{\delta s} = E_q = 2.04 \text{ kips}$
Deflection limit	$\Delta_{s_allow} = 0.020 * h = 2.74 in$
Induced unit shear	v _{ðs} = V _{ðs} / b = 494.55 lb/ft
Anchor tension force	T ₈ = max(0 kips,v _{8s} * h) = 5.646 kips
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_\delta / (k_a * b) = 0.626$ in
Deflection ampification factor	$C_{d\delta} = 4$
Seismic importance factor	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	δ_{sws} = Cdô * δ_{swse} / Ie = 2.503 in
	$\delta_{sws} / \Delta_{s_allow} = 0.914$
	PASS - Shear wall deflection is less than deflection limit

Tekla Tedds Fast + Epp	Project Yaroslavsky Residence Section Wood Shear Wall - Supp. Upper Level Wall 2					
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WOOD SHEAR WALL DESIGN (NDS)

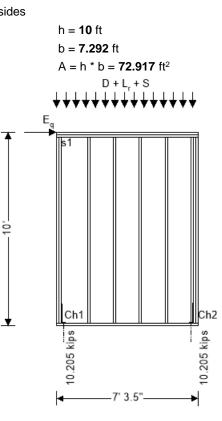
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Panel details

Structural wood panel sheathing on both sides

Panel height

Panel length Total area of wall



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	As = 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	Ae = 16.5 in ²
Hole diameter	Dia = 1 in
Net cross-sectional area of end posts	Aen = 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	ka = 80000 lb/in

Tekla Tedds Fast + Epp		Project Yaroslavsky Residence Section				
323 Dean Street, Suite #3						<u>.</u>
Brooklyn, NY 11217	Wood Shear Wall - Supp. Upper Level Wall 2				2	
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date
Specific gravity		G = 0.50				
Tension parallel to grain		Ft = 575 lb/in	2			
Compression parallel to grain		Fc = 1350 lb/				
Modulus of elasticity		E = 1600000				
Minimum modulus of elasticity		Emin = 58000	0 lb/in²			
Sheathing details		45/00				
Sheathing material			d panel 3-ply 1 nails at 2"ce		auning	
Fastener type						
From SDPWS Table 4.3A Nomin		-		Shear Walls -	Wood-based Pa	nels
Nominal unit shear capacity fo		•				
Nominal unit shear capacity fo Apparent shear wall shear stiff	•	n ∨w = 1790 lb Ga = 20 kips				
		Ga = 20 Kips	5/111			
Combined unit shear capacities						
Combined nominal unit shear	capacity for s	-	acco lb/ft			
Combined nominal unit chear	conocity for y	$V_{sc} = 2 * V_{s} =$	= 2360 ID/II			
Combined nominal unit shear		$V_{wc} = 2 * V_w$	- 2520 lb/ft			
Combined apparent shear wal	l shear stiffne			/in		
Loading details						
Dead load acting on top of panel		D = 295 lb/ft				
Roof live load acting on top of par	nel	Lr = 200 lb/ft				
Snow load acting on top of panel		S = 295 lb/ft				
Self weight of panel		$S_{wt} = 12 \text{ lb/ft}^2$	2			
In plane seismic load acting at he	-	Eq = 7441 lbs	3			
Design spectral response accel. p	ar., short perio	ods Sps = 0.944				
From IBC 2018 cl.1605.2				514/		
Load combination no.1 Load combination no.2		-	r or S or R) + 0).5Lf + 0.5(Lr o			
Load combination no.3		1.2D + W + 0	-			
Load combination no.4		0.9D + W				
Load combination no.5		0.9D + E				
Adjustment factors						
Format conversion factor for tensi	on – Table N1					
_	. —	K _{Ft} = 2.70				
Format conversion factor for comp	pression – Tal					
Format conversion factor for mode	ulus of elastici	K _{Fc} = 2.40 tv – Table N1				
		KFE = 1.76				
Resistance factor for tension – Ta	ble N2	$\phi_t = 0.80$				
Resistance factor for compression		$\phi_c = 0.90$				
Resistance factor for modulus of e		·				
		φs = 0.85				

Fast + Epp	Project Yaroslavsky I	Residence	Job Ref. 8119			
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Brooklyn, NY 11217	Wood Shear	Wall - Supp. Up	per Level Wall	2	3	
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date
Time effect factor – Table N3		$\lambda = 1.00$				
Sheathing resistance factor		φD = 0.80				
Size factor for tension – Table 4	A	CFt = 1.30				
Size factor for compression – Ta	able 4A	CFc = 1.10				
Wet service factor for tension -		C _{Mt} = 1.00				
Wet service factor for compressi		Смс = 1.00				
Wet service factor for modulus o	f elasticity – Tabl					
		Сме = 1.00				
Temperature factor for tension – Temperature factor for compress						
		Ctc = 1.00				
Temperature factor for modulus	or elasticity – Tat					
Inciding factor of 4.2.9		CtE = 1.00 Ci = 1.00				
Incising factor – cl.4.3.8 Buckling stiffness factor – cl.4.4.	2	Ci = 1.00 C⊤ = 1.00				
Adjusted modulus of elasticity	2		Кге * фs * Сме *		870000 nei	
					0,0000 poi	
Critical buckling design value			< Emin' / (h / d) ²	-	2200	
Reference compression design v For sawn lumber	alue	$Fc^{*} = Fc^{*} KFc$ C = 0.8	:* φc * λ * C Mc *		JZUO PSI	
	271		/ L .*\\ / (0 .	$(0) \sqrt{(1/1)}$	(FcE / Fc*)) / (2 >	(0)12 (E -
Column stability factor – eqn.	5.7-1	CP = (1 + (P + P + P + P + P + P + P + P + P +	,, ,	< C) — \([(I + ((FcE / Fc)) / (Z >	(C)] (FcE
From SDPWS Table 4.3.4 Maxi	mum Shear Wal	Aspect Ratios	5			
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 capacit	у					
Maximum shear force under	seismic loading	$V_{s_max} = E_q$	= 7.441 kips			
Shear capacity for seismic loa	ading	$V_s = \phi D * V_{sc}$	* b = 14.933	kips		
		$V_{s_max} / V_s =$	0.498			
		PASS - Shea	ar capacity for	seismic load	exceeds maxim	um shear f
Chord capacity for chords 1 a	nd 2					
Shear wall aspect ratio		h / b = 1.371				
Load combination 5						
Shear force for maximum tensio	n	V = Eq = 7.4	41 kips			
Axial force for maximum tens	ion	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		$F_t = F_t * K_{Ft}$	* φt * λ * Cмt * C	tt * CFt * Ci = 16	615 lb/in ²	
		ft / Ft' = 0.468				
		PASS - Desi	ign tensile stre	ess exceeds m	naximum applie	d tensile st
Load combination 3 Shear force for maximum compr		V = Eq = 7.4 4				

Tekla Tedds Fast + Epp	Project Yaroslavsk	y Residence	Job Ref. 8119					
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	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date		
Axial force for maximum co	mpression	P = (1.2 * (D	0 + S _{wt} * h) +	0.2 * Sps * (D) + S _{wt} * h) + +	0.7 * S) * s /		
		= 0.522 kips						
Maximum compressive force in	n chord	C = V * h / (b) + P = 10.727	kips				
Maximum applied compressive	e stress	$f_c = C / A_e = 6$	6 50 lb/in ²					
Design compressive stress		$F_{c}' = F_{c} * K_{Fc} * \phi_{c} * \lambda * C_{Mc} * C_{tc} * C_{Fc} * C_{i} * C_{P} = \textbf{1318} \text{ Ib/in}^{2}$						
		fc / Fc' = 0.493						
	PASS -	Design compress	sive stress exe	ceeds maxim	um applied com	pressive stre		
Hold down force								
Chord 1		T1 = 10.205 k	kips					
Chord 2		T ₂ = 10.205 k	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		$v_{\delta s} = V_{\delta s} / b = 1020.48 \text{ lb/ft}$						
Anchor tension force		T _δ = max(0 kips,v _{δs} * 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		le = 1						
Amp. seis. deflection – ASCE	7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_s$	wse / le = 1.89 in	n				
		δ_{sws} / Δ_s allow =	- 0.787					

Tekla .Tedds Fast + Epp						Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Wood Shear Wall - Supp. Upper Level Wall 3				Sheet no./rev. 1	Sheet no./rev. 1	
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date	

WOOD SHEAR WALL DESIGN (NDS)

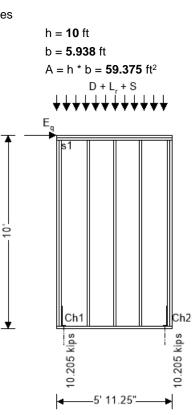
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Panel details

Structural wood panel sheathing on both sides

Panel height

Panel length Total area of wall



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	As = 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	Ae = 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	ka = 80000 lb/in

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323 Dean Street, Suite #3	Section					<u>.</u>
Brooklyn, NY 11217	Wood Shear Wall - Supp. Upper Level Wall 3				2	
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date
Specific gravity		G = 0.50				
Tension parallel to grain		Ft = 575 lb/in	2			
Compression parallel to grain		Fc = 1350 lb/				
Modulus of elasticity		E = 1600000				
Minimum modulus of elasticity		Emin = 58000	U Ib/in²			
Sheathing details		45/2011				
Sheathing material			d panel 3-ply nails at 2"ce		auning	
Fastener type						
From SDPWS Table 4.3A Nomin		-		hear Walls -	Wood-based Pa	nels
Nominal unit shear capacity for Nominal unit shear capacity for		•				
	-	$G_a = 20$ kips				
Apparent shear wall shear stiff	1622	$G_a = 20$ kips	5/111			
Combined unit shear capacities	ana situ far s					
Combined nominal unit shear of	apacity for s	-	arca lh/ft			
Combined nominal unit chear of	ennacity for y	$V_{sc} = 2 * V_{s} =$	= 2360 ID/IL			
Combined nominal unit shear of	apacity 101 v	$V_{wc} = 2 * V_w$	- 3590 lb/ft			
Combined apparent shear wall	shear stiffne			/in		
Loading details						
Dead load acting on top of panel		D = 245 lb/ft				
Roof live load acting on top of pan	el	Lr = 164 lb/ft				
Snow load acting on top of panel		S = 245 lb/ft				
Self weight of panel		$S_{wt} = 12 \text{ lb/ft}^2$	2			
In plane seismic load acting at hea	-	Eq = 6059 lbs	3			
Design spectral response accel. pa	ar., short perio	ods Sps = 0.944				
From IBC 2018 cl.1605.2						
Load combination no.1 Load combination no.2			• or S or R) + 0).5Lf + 0.5(Lr o			
Load combination no.3		1.2D + W + C	-			
Load combination no.4		0.9D + W				
Load combination no.5		0.9D + E				
Adjustment factors						
Format conversion factor for tension	on – Table N1					
-		KFt = 2.70				
Format conversion factor for comp	ression – Tat	ble N1 KFc = 2.40				
Format conversion factor for modu	lus of elasticit					
		KFE = 1.76				
Resistance factor for tension - Ta	ble N2	$\phi t = 0.80$				
Resistance factor for compression	– Table N2	φc = 0.90				
Resistance factor for modulus of e	lasticity – Tał	ble N2				
		φs = 0.85				

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Brooklyn, NY 11217	Wood Shear W	Wood Shear Wall - Supp. Upper Level Wall 33						
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date		
Time effect factor – Table N3		$\lambda = 1.00$						
Sheathing resistance factor		φD = 0.80						
Size factor for tension – Table 4A	N Contraction of the second se	CFt = 1.30						
Size factor for compression – Tal		CFc = 1.10						
Wet service factor for tension – T		CMt = 1.00						
Wet service factor for compression		Смс = 1.00						
Wet service factor for modulus of	elasticity – Table	е 4А Сме = 1.00						
Temperature factor for tension –	Table 2.3.3	$C_{ME} = 1.00$ $C_{tt} = 1.00$						
Temperature factor for compress		0 1.00						
		Ctc = 1.00						
Temperature factor for modulus of	of elasticity – Tab	le 2.3.3						
		CtE = 1.00						
Incising factor – cl.4.3.8		Ci = 1.00						
Buckling stiffness factor - cl.4.4.2	2	CT = 1.00						
Adjusted modulus of elasticity		$E_{min}' = E_{min} *$	Кге * фs * Сме *	$C_{tE} * C_i * C_T = C_{tE}$	870000 psi			
Critical buckling design value		$F_{cE} = 0.822 \times E_{min}' / (h / d)^2 = 1502 \text{ psi}$						
Reference compression design v	alue	$F_{c}^{*} = F_{c} * K_{Fc}$	* φc * λ * Cмc *	$C_{tc} * C_{Fc} * C_i =$	3208 psi			
For sawn lumber		c = 0.8						
Column stability factor - eqn.3	3.7-1	CP = (1 + (F	^F cE / Fc*)) / (2 >	< c) – √([(1 + ((FcE / Fc*)) / (2 ×	c)]² - (Fc∈		
		Fc*) / C) = 0 .	41					
From SDPWS Table 4.3.4 Maxir	num Shear Wall	Aspect Ratios	5					
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 s	-	Vs_max = Eq :	-					
Shear capacity for seismic loa	ding	$V_s = \phi_D * V_{sc}$	* b = 12.16 ki	ps				
		$V_{s_max} / V_s =$						
		PASS - Shea	ar capacity for	seismic load e	exceeds maxim	um shear f		
Chord capacity for chords 1 an	d 2							
Shear wall aspect ratio Load combination 5		h / b = 1.684						
Shear force for maximum tension	I Contraction of the second	V = Eq = 6.0	59 kips					
Axial force for maximum tensi	on	P = 0 kips =	= 0 kips					
		T = V * h / (b) - P = 10.205	kips				
Maximum tensile force in chord		$f_t = T / A_{en} = T$						
Maximum applied tensile stress				+ * C=+ * C: - 16	15 lb/in2			
		Ft' = Ft * KFt *	-					
Maximum applied tensile stress		ft / Ft' = 0.468	3					
Maximum applied tensile stress		ft / Ft' = 0.468	3		naximum applied	d tensile si		

Tekla Tedds Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsk	Project Yaroslavsky Residence					
	Section Wood Shea	Section Wood Shear Wall - Supp. Upper Level Wall 3				Sheet no./rev. 4	
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date	
Axial force for maximum co	mpression	P = (1.2 * ([0 + S _{wt} * h) +	0.2 * Sps * (C) + S _{wt} * h) + +	0.7 * S) * s /	
		= 0.452 kips					
Maximum compressive force in	n chord	C = V * h / (b) + P = 10.657	kips			
Maximum applied compressive	e stress	$f_c = C / A_e = 0$	646 lb/in ²				
Design compressive stress		F_{c} ' = F_{c} * K_{Fc} * ϕ_{c} * λ * C_{Mc} * C_{tc} * C_{Fc} * C_{i} * C_{P} = 1318 lb/in ²					
		fc / Fc' = 0.49	D				
	PASS -	Design compress	sive stress exe	ceeds maxim	um applied com	pressive stre	
Hold down force							
Chord 1		T1 = 10.205	kips				
Chord 2		T ₂ = 10.205	kips				
Seismic deflection							
Design shear force		$V_{\delta s} = E_q = 6.059 \text{ kips}$					
Deflection limit		$\Delta_{s_{allow}} = 0.020 * h = 2.4 in$					
Induced unit shear		ν _{δs} = V _{δs} / b = 1020.46 lb/ft					
Anchor tension force		T _δ = max(0 kips,v _{δs} * h) = 10.205 kips					
Shear wall elastic deflection -	Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s}$	* h ³ / (3 * E * A	Ae * b) + vδs * h	n / (Gac) + h * Τδ /	(ka * b) = 0.52	
		in					
Deflection ampification factor		$C_{d\delta} = 4$					
Seismic importance factor		le = 1					
Amp. seis. deflection – ASCE	7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_s$	wse / le = 2.088	in			
		δ_{sws} / Δ_s allow =	0.87				
		USWS / AS_allow =	- 0.01				

	Project Yaroslavsky Re	Job Ref. 8119				
323 Dean Street, Suite #3 Brooklyn, NY 11217	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)

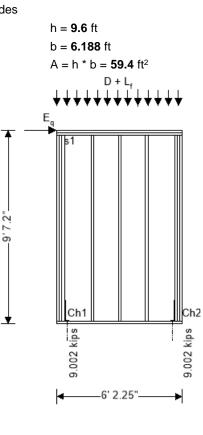
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Panel details

Structural wood panel sheathing on both sides

Panel height

Panel length Total area of wall



Panel	construction
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Nominal stud size	2" x 4"
Dressed stud size	1.5" x 3.5"
Cross-sectional area of studs	As = 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	Ae = 15.75 in ²
Hole diameter	Dia = 1 in
Net cross-sectional area of end posts	Aen = 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	ka = 80000 lb/in

Fast + Epp	^{Project} Yaroslavsky	Residence	Job Ref. 8119			
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section	· Wall - Supp. Up	Sheet no./rev 2	Sheet no./rev.		
		Date				Data
	Calc. by BJW	2/23/2021	Chk'd by	Date	App'd by	Date
Specific gravity		G = 0.50				
Tension parallel to grain		Ft = 575 lb/in	2			
Compression parallel to grain		Fc = 1350 lb/	'in²			
Modulus of elasticity		E = 160000				
Minimum modulus of elasticity		Emin = 58000	0 lb/in ²			
Sheathing details						
Sheathing material			d panel 3-ply		athing	
Fastener type		8d common	nails at 3"ce	enters		
From SDPWS Table 4.3A Nomina	I Unit Shear	Capacities for V	Vood-Frame S	Shear Walls - V	Wood-based Pa	nels
Nominal unit shear capacity for	seismic desi	ign vs = 980 lb/f	t			
Nominal unit shear capacity for	wind design	v _w = 1370 lb	/ft			
Apparent shear wall shear stiffn	ess	Ga = 15 kips	s/in			
Combined unit shear capacities						
Combined nominal unit shear ca	apacity for se	eismic design				
		$v_{sc} = 2 * v_s =$	= 1960 lb/ft			
Combined nominal unit shear ca	apacity for w	ind design				
		$V_{wc} = 2 * V_w$	= 2740 lb/ft			
Combined apparent shear wall	shear stiffne	ss Gac = Ga1 +	Ga2 = 30 kips	/in		
Loading details			·			
Dead load acting on top of panel		D = 294 lb/ft				
Floor live load acting on top of panel	el	$L_f = 392 \text{ lb/ft}$				
Self weight of panel		$S_{wt} = 12 \text{ lb/ft}^2$				
In plane seismic load acting at head	d of panel	E _q = 5802 lbs	S			
Design spectral response accel. pa	r., short perio	ds 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).5Lf + 0.5(Lr 0	r S or R)		
Load combination no.3		1.2D + E + 0	.5Lf + 0.7S			
Load combination no.4		0.9D + W				
Load combination no.5		0.9D + E				
Adjustment factors						
Format conversion factor for tension	n – Table N1					
		KFt = 2.70				
Format conversion factor for compr	ession – Tab	le N1 K⊧c = 2.40				
Format conversion factor for modul	us of elasticity					
		KFE = 1.76				
Resistance factor for tension – Tab	le N2	$\phi_t = 0.80$				
Resistance factor for compression		$\phi_{\rm c} = 0.90$				
Resistance factor for modulus of ela						
	,	φs = 0.85				

Fast + Epp	y Residence	Residence							
323 Dean Street, Suite #3 Section					Sheet no./rev				
Brooklyn, NY 11217	Wood Shea	Wood Shear Wall - Supp. Upper Level Wall 5							
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date			
Sheathing resistance factor		φD = 0.80							
Size factor for tension – Table 4	A	CFt = 1.50							
Size factor for compression – Ta	ble 4A	CFc = 1.15							
Wet service factor for tension -	Table 4A	C _{Mt} = 1.00							
Wet service factor for compressi	on – Table 4A	C _{Mc} = 1.00							
Wet service factor for modulus o	f elasticity – Ta	able 4A							
		Сме = 1.00							
Temperature factor for tension -	Table 2.3.3	Ctt = 1.00							
Temperature factor for compress	sion – Table 2.3	3.3							
		Ctc = 1.00							
Temperature factor for modulus	of elasticity – T								
		CtE = 1.00							
Incising factor – cl.4.3.8	•	Ci = 1.00							
Buckling stiffness factor – cl.4.4.	Z	CT = 1.00	V * · * ~ · ·	0 * 0 * 0	070000 ·				
Adjusted modulus of elasticity			Кге * фs * Сме *		870000 psi				
Critical buckling design value			< Emin' / (h / d)²	-					
Reference compression design	/alue	$F_{c}^{*} = F_{c} * K_{Fc}$	Fc* = Fc * KFc * φc * λ * CMc * Ctc * CFc * Ci = 3353 psi						
For sawn lumber		c = 0.8							
Column stability factor - eqn.	3.7-1	C _P = (1 + (F	^F cE / Fc*)) / (2 >	< c) – √([(1 + ((FcE / Fc*)) / (2 :	× c)]² - (Fc∈			
		$F_{c}^{*}) / c) = 0$.19						
From SDPWS Table 4.3.4 Maxi	mum Shear W	all Aspect Ratios	6						
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 capacit									
Maximum shear force under		0	= 5.802 kips						
Shear capacity for seismic loa	ading	$V_s = \phi_D * v_{so}$	e * b = 9.702 ki	ps					
		$V_{s_max} / V_s =$							
		PASS - Shea	ar capacity for	seismic load	exceeds maxim	num shear f			
Chord capacity for chords 1 and	nd 2								
Shear wall aspect ratio		h / b = 1.552							
Load combination 5									
Shear force for maximum tension		V = E _q = 5.8	-						
Axial force for maximum tens	ion	P = 0 kips = 0 kips							
Maximum tensile force in chord) - P = 9.002 ki	ps					
Maximum applied tensile stress		$f_t = T / A_{en} =$							
Design tensile stress			* φt * λ * Cмt * C	tt * CFt * Ci = 18	363 lb/in ²				
5		$f_t / F_t' = 0.430$				-14 ¹¹ -			
C C		PASS - Des	ian tensile stre	ess exceeds m	naximum applie	a tensile st			
-			9						
Load combination 3	ession		-						
-		V = E _q = 5.8	02 kips	ם).2 * Sps * (D	+ S _{wt} * h) + 0.5	5 * Lf) * s / 2			

Tekla Tedds Fast + Epp	Project Yaroslavsky	y Residence	Job Ref. 8119			
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Wood Shea	ar Wall - Supp. Upp	Sheet no./rev. 4			
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date
Maximum compressive force in	chord	C = V * h / (b)) + P = 9.511 k	kips		
Maximum applied compressive	stress	$f_c = C / A_e = 6$	604 lb/in ²			
Design compressive stress		$F_{c}' = F_{c} * K_{Fc}$	* фс * λ * Смс *	Ctc * CFc * Ci *	CP = 631 lb/in ²	
		fc / Fc' = 0.95	7			
	PASS - I	Design compress	ive stress ex	ceeds maxim	um applied com	pressive stre
Hold down force						
Chord 1		T1 = 9.002 kip	os			
Chord 2		T2 = 9.002 kip	os			
Seismic deflection						
Seismic deflection Design shear force		Võs = Eq = 5 .	802 kips			
			802 kips 20 * h = 2.304	in		
Design shear force			20 * h = 2.304	in		
Design shear force Deflection limit		Δ s_allow= 0.02 v _{δs} = V _{δs} / b =	20 * h = 2.304			
Design shear force Deflection limit Induced unit shear	Eqn. 4.3-1	Δ s_allow= 0.02 v _{os} = V _{os} / b = T _o = max(0 ki	20 * h = 2.304 937.7 lb/ft ps,v _{õs} * h) = 9.	002 kips	∩ / (Gac) + h * Tδ / (ka * b) = 0.5 1
Design shear force Deflection limit Induced unit shear Anchor tension force	Eqn. 4.3-1	Δ s_allow= 0.02 v _{os} = V _{os} / b = T _o = max(0 ki	20 * h = 2.304 937.7 lb/ft ps,v _{õs} * h) = 9.	002 kips	n / (Gac) + h * T _ð / (ka * b) = 0.51
Design shear force Deflection limit Induced unit shear Anchor tension force	Eqn. 4.3-1	$\Delta s_{allow} = 0.02$ $v_{\delta s} = V_{\delta s} / b =$ $T_{\delta} = max(0 \text{ ki})$ $\delta_{swse} = 2 * v_{\delta s}$	20 * h = 2.304 937.7 lb/ft ps,v _{õs} * h) = 9.	002 kips	n / (Gac) + h * T _ð / (ka * b) = 0.51
Design shear force Deflection limit Induced unit shear Anchor tension force Shear wall elastic deflection – F	Eqn. 4.3-1	$\Delta s_{allow} = 0.02$ $v_{\delta s} = V_{\delta s} / b =$ $T_{\delta} = max(0 \text{ ki})$ $\delta_{swse} = 2 * v_{\delta s}$ in	20 * h = 2.304 937.7 lb/ft ps,v _{õs} * h) = 9.	002 kips	n / (Gac) + h * Tõ / (<u>k</u> a * b) = 0.51
Design shear force Deflection limit Induced unit shear Anchor tension force Shear wall elastic deflection – E Deflection ampification factor		$\Delta s_allow= 0.02$ $v_{\delta s} = V_{\delta s} / b =$ $T_{\delta} = max(0 \text{ ki})$ $\delta_{swse} = 2 * v_{\delta s}$ in $C_{d\delta} = 4$ $I_{e} = 1$	20 * h = 2.304 937.7 lb/ft ps,v _{õs} * h) = 9.	002 kips A _e * b) + v _{õs} * h	n / (Gac) + h * T _ð / ('ka * b) = 0.51
Design shear force Deflection limit Induced unit shear Anchor tension force Shear wall elastic deflection – E Deflection ampification factor Seismic importance factor		$\Delta s_allow= 0.02$ $v_{\delta s} = V_{\delta s} / b =$ $T_{\delta} = max(0 \text{ ki})$ $\delta_{swse} = 2 * v_{\delta s}$ in $C_{d\delta} = 4$ $I_{e} = 1$	20 * h = 2.304 937.7 lb/ft ps,v _{ðs} * h) = 9. * h ³ / (3 * E * A wse / le = 2.069	002 kips A _e * b) + v _{õs} * h	n / (Gac) + h * T _ð / (ka * b) = 0.51

Tekla .Tedds Fast + Epp	Project Yaroslavsky Residence					
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WOOD SHEAR WALL DESIGN (NDS)

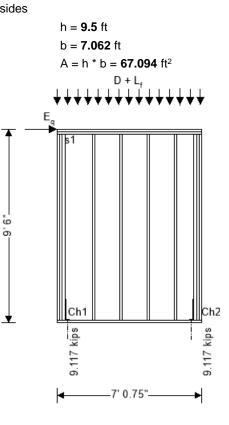
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Panel details

Structural wood panel sheathing on both sides

Panel height

Panel length Total area of wall



ranel construction	Panel	construction
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Nominal stud size	2" x 4"
Dressed stud size	1.5" x 3.5"
Cross-sectional area of studs	As = 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	Ae = 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	ka = 80000 lb/in

Tekla. Tedds Project Fast + Epp Yaroslavsk 323 Dean Street, Suite #3 Section Brooklyn, NY 11217 Ware of Observed		Residence	Job Ref. 8119			
			Sheet no./rev	Sheet no./rev.		
	Wood Shear	Wall - Supp. Up	per Level Wall	6	2	
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date
Specific gravity		G = 0.50				
Tension parallel to grain		Ft = 575 lb/ir	2			
Compression parallel to grain		Fc = 1350 lb/	ïn²			
Modulus of elasticity		E = 160000	lb/in ²			
Minimum modulus of elasticity		Emin = 58000	0 lb/in ²			
Sheathing details						
Sheathing material			d panel 3-ply		athing	
Fastener type		8d commor	nails at 3"ce	nters		
From SDPWS Table 4.3A Nomin	nal Unit Shear C	Capacities for V	Vood-Frame S	hear Walls - W	Vood-based Pa	nels
Nominal unit shear capacity for	r seismic desig	gn vs = 980 lb/f	t			
Nominal unit shear capacity for	r wind design	v _w = 1370 lb	/ft			
Apparent shear wall shear stif	iness	Ga = 15 kips	s/in			
Combined unit shear capacities	6					
Combined nominal unit shear	capacity for se	ismic design				
		$v_{sc} = 2 * v_s :$	= 1960 lb/ft			
Combined nominal unit shear	capacity for wi	nd design				
		$v_{wc} = 2 * v_w$	= 2740 lb/ft			
Combined apparent shear wal	l shear stiffnes	$G_{ac} = G_{a1} + G_{a1}$	Ga2 = 30 kips/	/in		
Loading details						
Dead load acting on top of panel		D = 231.25	o/ft			
Floor live load acting on top of pa	nel	Lf = 309 lb/ft				
Self weight of panel		$S_{wt} = 12 \text{ lb/ft}$				
In plane seismic load acting at he	•	Eq = 6778 lb	S			
Design spectral response accel.	oar., snort period	IS SDS = 0.944				
From IBC 2018 cl.1605.2				-		
Load combination no.1			or S or R) + 0			
Load combination no.2 Load combination no.3		1.2D + W + 0 1.2D + E + 0).5Lf + 0.5(Lr 0I 5L	501 K)		
Load combination no.4		0.9D + W	.521 + 0.70			
Load combination no.5		0.9D + W				
Adjustment factors						
Format conversion factor for tens	ion – Table N1					
		KFt = 2.70				
Format conversion factor for com	pression – Table	e N1				
		KFc = 2.40				
Format conversion factor for mod	ulus of elasticity					
Desistance factor for torrais. T		KFE = 1.76				
Resistance factor for tension – Ta		$\phi_{t} = 0.80$				
Resistance factor for compression Resistance factor for modulus of		φc = 0.90				
		φs = 0.85				

Fast + Epp	Project Yaroslavsk	y Residence			Job Ref. 8119					
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section		Sheet no./rev.							
Brooklyn, NY 11217	Wood Shea	ar Wall - Supp. Up	per Level Wall	6	3					
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date				
Sheathing resistance factor		φ D = 0.80								
Size factor for tension - Table 4	A	CFt = 1.50								
Size factor for compression - Ta	able 4A	CFc = 1.15								
Wet service factor for tension -	Table 4A	C _{Mt} = 1.00								
Wet service factor for compress	ion – Table 4A	C _{Mc} = 1.00								
Wet service factor for modulus of	of elasticity – Ta	ble 4A								
		Cme = 1.00								
Temperature factor for tension -	- Table 2.3.3	Ctt = 1.00								
Temperature factor for compres	sion – Table 2.3	3.3								
		Ctc = 1.00								
Temperature factor for modulus	of elasticity – T	able 2.3.3								
		CtE = 1.00								
Incising factor – cl.4.3.8		Ci = 1.00								
Buckling stiffness factor - cl.4.4	.2	C⊤ = 1.00								
Adjusted modulus of elasticity		Emin' = Emin *	Кге * фѕ * Сме *	$C_{tE} * C_i * C_T =$	870000 psi					
Critical buckling design value		F _{cE} = 0.822 >	< Emin' / (h / d)²	= 674 psi						
Reference compression design	value	$F_{c}^{*} = F_{c} * K_{Fc}$	F_{c}^{*} = F_{c} * K_{Fc} * ϕ_{c} * λ * C_{Mc} * C_{tc} * C_{Fc} * C_{i} = 3353 psi							
For sawn lumber		c = 0.8	c = 0.8							
Column stability factor - eqn.	3.7-1	C _P = (1 + (F		< c) – √([(1 +)	(FcE / Fc*)) / (2 >	< c)]² - (F c∈				
		F_{c^*} / c) = 0.	,, ,			、				
From ODDMO Table 40.4 M		, ,								
From SDPWS Table 4.3.4 Max		-								
Maximum shear wall aspect rational Shear wall length	U	3.5 b = 7.062 ft								
Shear wall aspect ratio		b = 7.062 n h / b = 1.345								
·		117 D - 1.343								
Segmented shear wall capacit Maximum shear force under	-	g Vs_max = Eq	= 6.778 kips							
Shear capacity for seismic lo		e	* b = 11.074	kips						
	- 3	$V_{s_max} / V_{s} =$		r =						
				seismic load	exceeds maxim	um shear f				
Chard canacity for shards 4 -	nd 2									
Chord capacity for chords 1 a		h / b = 1.345								
Shear wall aspect ratio		117 D = 1.343								
Load combination 5										
Load combination 5	n		/ X kine							
Shear force for maximum tensio		V = E _q = 6.7 7 P = 0 kips =	-							
Shear force for maximum tension Axial force for maximum tension		P = 0 kips =	0 kips	06						
Shear force for maximum tensio Axial force for maximum tens Maximum tensile force in chord		P = 0 kips = T = V * h / (b	= 0 kips) - P = 9.117 ki	ps						
Shear force for maximum tensio Axial force for maximum tens Maximum tensile force in chord Maximum applied tensile stress		P = 0 kips = T = V * h / (b ft = T / A _{en} =	= 0 kips) - P = 9.117 ki 810 lb/in²		863 Ih/in ²					
Shear force for maximum tensio Axial force for maximum tens Maximum tensile force in chord		P = 0 kips = T = V * h / (b ft = T / A _{en} = Ft' = Ft * K _{Ft} *	= 0 kips) - Ρ = 9.117 ki 810 lb/in ² φt * λ * C _{Mt} * C		363 lb/in²					
Shear force for maximum tensio Axial force for maximum tens Maximum tensile force in chord Maximum applied tensile stress		P = 0 kips = T = V * h / (b) ft = T / Aen = Ft' = Ft * KFt * ft / Ft' = 0.43	= 0 kips) - P = 9.117 ki 810 lb/in ² ⁴ φt * λ * C _{Mt} * C 5	tt * CFt * Ci = 18		d tancila at				
Shear force for maximum tensio Axial force for maximum tens Maximum tensile force in chord Maximum applied tensile stress Design tensile stress		P = 0 kips = T = V * h / (b) ft = T / Aen = Ft' = Ft * KFt * ft / Ft' = 0.43	= 0 kips) - P = 9.117 ki 810 lb/in ² ⁴ φt * λ * C _{Mt} * C 5	tt * CFt * Ci = 18	363 lb/in² naximum applied	d tensile st				
Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3	sion	P = 0 kips = T = V * h / (bft = T / Aen = Ft' = Ft * KFt * ft / Ft' = 0.43PASS - Desi	= 0 kips) - P = 9.117 ki 810 lb/in ² φt * λ * C _{Mt} * C 5 gn tensile stre	tt * CFt * Ci = 18		d tensile st				
Shear force for maximum tensio Axial force for maximum tens Maximum tensile force in chord Maximum applied tensile stress Design tensile stress	ression	P = 0 kips = T = V * h / (b + ft) = T / Aen = Ft' = Ft * KFt * ft / Ft' = 0.43! $PASS - Deside V = Eq = 6.77$	= 0 kips) - P = 9.117 ki 810 lb/in ² φt * λ * C _{Mt} * C 5 gn tensile stre 7 8 kips	tt * CFt * Ci = 18 ess exceeds n						

Tekla Tedds Fast + Epp	Project Yaroslavsky	y Residence	Job Ref. 8119				
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Wood Shea	ar Wall - Supp. Upp	ber Level Wall	6	Sheet no./rev. 4	Sheet no./rev. 4	
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date	
Maximum compressive force in	chord	C = V * h / (b)) + P = 9.540 k	kips			
Maximum applied compressive	stress	$f_c = C / A_e = 6$	06 lb/in ²				
Design compressive stress		$F_{c}' = F_{c} * K_{Fc}$	* фс * λ * Смс *	Ctc * CFc * Ci *	CP = 644 lb/in ²		
		fc / Fc' = 0.94	I				
	PASS - I	Design compress	ive stress ex	ceeds maxim	um applied com	pressive stre	
Hold down force							
Chord 1		T1 = 9.117 kip	os				
Chord 2		T2 = 9.117 kip	os				
Opionnia deflection							
Seismic deflection							
Design shear force		$V_{\delta s} = E_q = 6.$	778 kips				
			778 kips 20 * h = 2.28 i	'n			
Design shear force			20 * h = 2.28 i	'n			
Design shear force Deflection limit		Δ s_allow= 0.02 v _{\deltas} = V _{\deltas} / b =	20 * h = 2.28 i				
Design shear force Deflection limit Induced unit shear	Eqn. 4.3-1	Δ s_allow= 0.02 v _{\deltas} = V _{\deltas} / b = T _{\delta} = max(0 ki	20 * h = 2.28 i 959.72 lb/ft ps,v _{õs} * h) = 9.	117 kips	η / (Gac) + h * Τδ /	(ka * b) = 0.49	
Design shear force Deflection limit Induced unit shear Anchor tension force	Eqn. 4.3-1	Δ s_allow= 0.02 v _{\deltas} = V _{\deltas} / b = T _{\delta} = max(0 ki	20 * h = 2.28 i 959.72 lb/ft ps,v _{õs} * h) = 9.	117 kips	n / (Gac) + h * T _δ /	(ka * b) = 0.49	
Design shear force Deflection limit Induced unit shear Anchor tension force	Eqn. 4.3-1	$\Delta s_{allow} = 0.02$ $v_{\delta s} = V_{\delta s} / b =$ $T_{\delta} = max(0 \text{ ki})$ $\delta_{swse} = 2 * v_{\delta s}$	20 * h = 2.28 i 959.72 lb/ft ps,v _{õs} * h) = 9.	117 kips	n / (Gac) + h * Tõ /	(ka * b) = 0.49	
Design shear force Deflection limit Induced unit shear Anchor tension force Shear wall elastic deflection – I	Eqn. 4.3-1	$\Delta s_allow= 0.02$ $v_{\delta s} = V_{\delta s} / b =$ $T_{\delta} = max(0 \text{ ki})$ $\delta_{swse} = 2 * v_{\delta s}$ in	20 * h = 2.28 i 959.72 lb/ft ps,v _{õs} * h) = 9.	117 kips	n / (Gac) + h * T _ð /	(ka * b) = 0.49	
Design shear force Deflection limit Induced unit shear Anchor tension force Shear wall elastic deflection – I Deflection ampification factor		$\Delta s_allow= 0.02$ $v_{\delta s} = V_{\delta s} / b =$ $T_{\delta} = max(0 \text{ ki})$ $\delta_{swse} = 2 * v_{\delta s}$ in $C_{d\delta} = 4$ $I_{e} = 1$	20 * h = 2.28 i 959.72 lb/ft ps,v _{õs} * h) = 9.	117 kips Ae * b) + vδs * h	n / (Gac) + h * T _ð /	(k _a * b) = 0.49	
Design shear force Deflection limit Induced unit shear Anchor tension force Shear wall elastic deflection – I Deflection ampification factor Seismic importance factor		$\Delta s_allow= 0.02$ $v_{\delta s} = V_{\delta s} / b =$ $T_{\delta} = max(0 \text{ ki})$ $\delta_{swse} = 2 * v_{\delta s}$ in $C_{d\delta} = 4$ $I_{e} = 1$	20 * h = 2.28 i 959.72 lb/ft ps,v _{ðs} * h) = 9. * h ³ / (3 * E * /	117 kips Ae * b) + vδs * h	n / (Gac) + h * T _ð /	(ka * b) = 0.49	

Tekla Tedds Fast + Epp	Project Yaroslavsky	Residence			Job Ref. 8119		
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Wood Shea	SectionSheet no./rev.Wood Shear Wall - Supp. Upper Level Wall 71					
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date	
WOOD SHEAR WALL DESIG	N (NDS)	I					
In accordance with NDS2018	load reduction f	actor design an	d the segment	ed shear wall			
Panel details					Tedds cai	culation versio	
Structural wood panel sheathin	g on one side						
Panel height		h = 10 ft					
Panel length		b = 13.354 ft					
Total area of wall		A = h * b = 1					
	\pm 1 1 1	• D • • • • • • <u>• + + + + + + + +</u>	ᆠᇈ _ᠻ ᠘᠘᠘᠘᠘᠘᠘᠘᠘				
	E _g	********	******	****			
	4.478 kips	13'	4.25"	Ch2			
Panel construction							
Nominal stud size		2" x 6"					
Dressed stud size		1.5" x 5.5"					
Cross-sectional area of studs		As = 8.25 in ²					
Stud spacing		s = 16 in					
Nominal end post size		2 x 2" x 6"	-11				
Dressed end post size	- 4 -	2 x 1.5" x 5.5					
Cross-sectional area of end po Hole diameter	SIS	A₀ = 16.5 in² Dia = 1 in					
Net cross-sectional area of end	l poste	Dia = 1 in A _{en} = 13.5 in	2				
Nominal collector size	μυδιδ	Aen = 13.5 IN 2 x 2'' x 6''					
Dressed collector size		2 x 2 x 0 2 x 1.5" x 5.5	5"				
Service condition		Dry	-				
Temperature		100 degF or	less				
Vertical anchor stiffness		ka = 80000 lk					

Tekla Tedds Fast + Epp	Project Yaroslavsky	Residence			Job Ref. 8119	
323 Dean Street, Suite #3	Section				Sheet no./rev	
Brooklyn, NY 11217	Wood Shear	Wall - Supp. Up	per Level Wall	7	2	
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date
Specific gravity		G = 0.50				
Tension parallel to grain		Ft = 575 lb/ir	2			
Compression parallel to grain		Fc = 1350 lb/	'in²			
Modulus of elasticity		E = 1600000	lb/in²			
Minimum modulus of elasticity		Emin = 58000	0 lb/in ²			
Sheathing details						
Sheathing material		15/32'' woo	d panel 3-ply	plywood she	athing	
Fastener type		8d commor	nails at 3"ce	nters	-	
From SDPWS Table 4.3A Nom	inal Unit Shear	Canacities for V	Vood-Frame S	hoar Walls - V	Nood-based Pa	nole
Nominal unit shear capacity		-				
Nominal unit shear capacity	for wind design	v _w = 1370 lb	/ft			
Apparent shear wall shear st	iffness	Ga = 15 kips	s/in			
Loading details						
Dead load acting on top of pane	I	D = 60 lb/ft				
Floor live load acting on top of p		L _f = 80 lb/ft				
Self weight of panel		Swt = 12 lb/ft ²	2			
In plane seismic load acting at h	ead of panel	E _q = 5980 lbs	S			
Design spectral response accel.	par., short perio	ds 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	0.5Lf + 0.5(Lr o	r S or R)		
Load combination no.3		1.2D + E + 0	.5Lf + 0.7S			
Load combination no.4		0.9D + W				
Load combination no.5		0.9D + E				
Adjustment factors						
Format conversion factor for ten	sion – Table N1					
		KFt = 2.70				
Format conversion factor for cor	mpression – Tabl					
-		KFc = 2.40				
Format conversion factor for mo	dulus of elasticity					
Desistance for the fort		KFE = 1.76				
Resistance factor for tension –		$\phi_t = 0.80$				
Resistance factor for compressi		$\phi_c = 0.90$				
Resistance factor for modulus o	t elasticity – Tab					
-		φs = 0.85				
Time effect factor – Table N3		$\lambda = 1.00$				
Sheathing resistance factor		φ D = 0.80				
Size factor for tension – Table 4		CFt = 1.30				
Size factor for compression – Ta		CFc = 1.10				
Wet service factor for tension -		C _{Mt} = 1.00				
		() 4 00				
Wet service factor for compress Wet service factor for modulus of		C _{Mc} = 1.00				

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Brooklyn, NY 11217	Wood She	ar Wall - Supp. Up	oper Level Wall	7	3	
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date
Temperature factor for tension	- Table 2.3.3	Ctt = 1.00				
Temperature factor for compres	sion – Table 2.					
- · · · · · · · ·	.	$C_{tc} = 1.00$				
Temperature factor for modulus	of elasticity - 1	CtE = 1.00				
Incising factor – cl.4.3.8		Ci⊨ = 1.00 Ci = 1.00				
Buckling stiffness factor – cl.4.4	.2	CT = 1.00				
Adjusted modulus of elasticity		Emin' = Emin *	Кге * фs * Сме *	* CtE * Ci * CT =	870000 psi	
Critical buckling design value			× Emin' / (h / d)²		·	
Reference compression design	value			Ctc * CFc * Ci =	3208 psi	
For sawn lumber		c = 0.8	- +- ··			
Column stability factor - eqn	.3.7-1	C _P = (1 + (I	F _{cE} / F _c *)) / (2	× c) – √([(1 + ((FcE / Fc*)) / (2 >	≺ c)]² - (F cE
, , , , , , , , , , , , , , , , , , , ,		F_{c}^{*}) / c) = 0				
From CDDWC Table 4.2.4 Max	imum Cheer M	, ,				
From SDPWS Table 4.3.4 Max Maximum shear wall aspect rat		3.5	5			
Shear wall length	0	b = 13.354 f	t			
Shear wall aspect ratio		h / b = 0.74 9				
Segmented shear wall capaci	tv					
Maximum shear force under	-	ng Vs_max = Eq	= 5.98 kips			
Shear capacity for seismic lo		-	* b = 10.47 ki	ns		
	lading	Vs_max / Vs =		20		
				seismic load	exceeds maxim	ium shear f
Chord capacity for chords 1 a	ind 2					
Shear wall aspect ratio Load combination 5		h / b = 0.749)			
Shear force for maximum tension	on	V = E _q = 5.9	8 kips			
Axial force for maximum tens	sion	P = 0 kips :	= 0 kips			
Maximum tensile force in chord		T = V * h / (t	o) - P = 4.478 k	ips		
Maximum applied tensile stress		$f_t = T / A_{en} =$				
Design tensile stress				$C_{tt} * C_{Ft} * C_i = 16$	15 lb/in ²	
		$f_t / F_t' = 0.20$				-1.4
Load combination 3		PASS - Des	ign tensile str	ess exceeds m	naximum applie	a tensile si
Shear force for maximum comp	ression	V = Eq = 5.9	8 kins			
Axial force for maximum comp			•	0.2 * Sds * (D	+ S _{wt} * h) + 0.5	5 * L _f) * s / 2
		0.193 kips				
Maximum compressive force in	chord	C = V * h / (I	b) + P = 4.671 ∣	kips		
Maximum applied compressive	stress	$f_c = C / A_e =$	283 lb/in ²			
Design compressive stress		$F_{c}' = F_{c} * K_{Fc}$	с* фс*λ*Смс*	Ctc * CFc * Ci * (CP = 1318 lb/in ²	
		fc / Fc' = 0.2 1	-			
	PASS -	Design compres	sive stress ex	ceeds maximu	im applied com	pressive st
Hold down force						
Chord 1		T1 = 4.478 k	ips			

Tekla Tedds Fast + Epp						Job Ref. 8119	
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Chord 2	T ₂ = 4.478 kips
Seismic deflection	
Design shear force	V₀s = Eq = 5.98 kips
Deflection limit	$\Delta_{s_allow} = 0.020 * h = 2.4 in$
Induced unit shear	v _{ðs} = V _{ðs} / b = 447.8 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * h) = 4.478 kips
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_{\delta} / (k_a * b) = 0.351 \text{ in}$
Deflection ampification factor	$C_{d\delta} = 4$
Seismic importance factor	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	δ_{sws} = Cdô * δ_{swse} / Ie = 1.402 in
	$\delta_{sws} / \Delta_{s_allow} = 0.584$
	PASS - Shear wall deflection is less than deflection limit

Tekla .Tedds Fast + Epp	Project Yaroslavsky Residence					
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WOOD SHEAR WALL DESIGN (NDS)

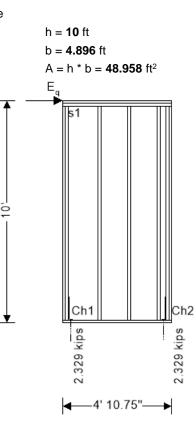
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Panel details

Structural wood panel sheathing on one side

Panel height Panel length

Total area of wall



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	As = 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	Ae = 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	ka = 80000 lb/in

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323 Dean Street, Suite #3	Section				Sheet no./rev	
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	Calc. by BJW	Date 2/19/2021	Chk'd by	Date	App'd by	Date
From NDS Supplement Table 4. Species, grade and size classifica Specific gravity Tension parallel to grain Compression parallel to grain Modulus of elasticity Minimum modulus of elasticity Sheathing details Sheathing material Fastener type From SDPWS Table 4.3A Nomin Nominal unit shear capacity for Nominal unit shear capacity for Apparent shear wall shear stift	nal Unit Shear or seismic des or wind design	Douglas Fir-I G = 0.50 $F_t = 575$ lb/in $F_c = 1350$ lb/ E = 1600000 $E_{min} = 58000$ 15/32'' woo 8d common Capacities for V sign v _s = 980 lb/f	Larch, no.2 gra ² ² ¹ Ib/in ² 0 Ib/in ² 0 Ib/in ² 1 d panel 3-ply 1 nails at 3"ce Vood-Frame S t /ft	ade, 2" & wider plywood she enters	athing	
Loading details Self weight of panel In plane seismic load acting at he Design spectral response accel.		$S_{wt} = 12 \text{ lb/ft}^2$ $E_q = 1140 \text{ lbs}^2$ ods $S_{DS} = 0.944$				
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 + (0.5Lf + 0.5(Lr o	r S or R)		
Load combination no.3		1.2D + E + 0	.5Lf + 0.7S			
Load combination no.4		0.9D + W				
Load combination no.5		0.9D + E				
Adjustment factors						
Format conversion factor for tens	ion – Table N1					
		K _{Ft} = 2.70				
Format conversion factor for com	pression – Tab					
		KFc = 2.40				
Format conversion factor for mod	ulus of elasticit	-				
		KFE = 1.76				
Resistance factor for tension – Ta		$\varphi_t = 0.80$				
Resistance factor for compression		$\varphi_{\rm c}=\boldsymbol{0.90}$				
Resistance factor for modulus of	elasticity – Tab					
		$\phi_s = 0.85$				
Time effect factor – Table N3		$\lambda = 1.00$				
Sheathing resistance factor		φD = 0.80				
Size factor for tension – Table 4A	L.	CFt = 1.30				
Size factor for compression - Tak	ole 4A	CFc = 1.10				
Wet service factor for tension – T	able 4A	C _{Mt} = 1.00				
Wet service factor for compression Wet service factor for modulus of		С _{Мс} = 1.00 Ые 4А				
viol service racion for mounius of	clasticity - ra	Сме = 1.00				

Fast + Epp	Project Yaroslavsk	y Residence			Job Ref. 8119				
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Brooklyn, NY 11217		ar Wall - Supp. U	pper Level Wall	8	3				
	Calc. by BJW	Date 2/19/2021	Chk'd by	Date	App'd by	Date			
Temperature factor for tension -	- Table 2.3.3	Ctt = 1.00	I	I		I			
Temperature factor for compress		3.3							
		Ctc = 1.00							
Temperature factor for modulus	of elasticity – T								
Inciding factor of 4.2.0		CtE = 1.00 Ci = 1.00							
Incising factor – cl.4.3.8 Buckling stiffness factor – cl.4.4	2	C₁ = 1.00 C⊤ = 1.00							
Adjusted modulus of elasticity	.2		* Кге * фѕ * Сме *	Сн= * Сі * Ст –	870000 nsi				
					Groude par				
Critical buckling design value			× Emin' / (h / d) ²	-					
Reference compression design	value		с* фс*λ*Смс*	Utc " UFc * Ci =	3208 psi				
For sawn lumber	074	c = 0.8				a)12 (F			
Column stability factor – eqn.	3.7-1		<i>,,</i> , , , , , , , , , , , , , , , , , ,	\times C) - $\sqrt{((1 + 1)^{2})^{2}}$	(FcE / Fc*)) / (2	× C)]² - (FcE			
		Fc*) / C) = ().41						
From SDPWS Table 4.3.4 Maxi		-	S						
Maximum shear wall aspect ratio	C	3.5							
Shear wall length		b = 4.896 ft							
Shear wall aspect ratio		h / b = 2.04 3	3						
Segmented shear wall capacit	-								
Maximum shear force under	seismic loadir	ng Vs_max = Eq	= 1.14 kips						
Shear capacity for seismic lo	ading	$V_s = \phi_D * v_s$	s * b * (1.25 - 0	0.125 * h / bs)	= 3.818 kips				
		Vs_max / Vs =	0.299						
		PASS - She	ar capacity for	seismic load	exceeds maxim	num shear f			
Chord capacity for chords 1 a	nd 2								
Shear wall aspect ratio Load combination 5		h / b = 2.04 3	3						
Shear force for maximum tensio	n	V = Eq = 1.1	4 kips						
Axial force for maximum tens	ion	P = 0 kips	= 0 kips						
Maximum tensile force in chord		T = V * h / (l	b) - P = 2.329 ki	ips					
Maximum applied tensile stress		ft = T / A _{en} =	172 lb/in ²						
Design tensile stress		$F_t = F_t * K_{Ft}$	* фt * λ * Смt * С	Ctt * CFt * Ci = 16	615 lb/in ²				
		ft / Ft' = 0.10							
		PASS - Des	sign tensile stre	ess exceeds n	naximum applie	d tensile st			
Load combination 3									
Shear force for maximum compr		$V = E_q = 1.1$	•						
Axial force for maximum com	-				* s / 2 = 0.111 k	ips			
Maximum compressive force in o			b) + P = 2.440 k	kips					
Maximum applied compressive s	stress	$f_c = C / A_e =$		0 10 10					
Design compressive stress				Ctc * CFc * Ci *	CP = 1318 lb/in ²				
	DVCC	f₀ / F₀' = 0.1 Design compres		coode movim	im applied com	proseivo d			
	FA32 -	Design compres	SSIVE SUESS EX	CEEUS MAXIM	ani applieu com	pressive SI			
Hold down force		T1 = 2.329 k							
Chord 1									

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Seismic o	deflection
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Design shear force	$V_{\delta s} = E_q = 1.14 \text{ kips}$
Deflection limit	$\Delta_{s_{allow}} = 0.020 * h = 2.4 in$
Induced unit shear	v _{ðs} = V _{ðs} / b = 232.85 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * h) = 2.329 kips
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_{\delta} / (k_a * b) = 0.229 \text{ in}$
Deflection ampification factor	$C_{d\delta} = 4$
Seismic importance factor	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	δ_{sws} = Cdô * δ_{swse} / Ie = 0.916 in
	$\delta_{sws} / \Delta_{s_allow} = 0.382$

PASS - Shear wall deflection is less than deflection limit

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WOOD SHEAR WALL DESIG		ctor design an	d the segmen	ted shear wall	method	
		-	-			culation versio
Panel details Structural wood panel sheathin	a on both sides					
Panel height	9 011 2011 01400	h = 10 ft				
Panel length		b = 11 ft				
Total area of wall		A = h * b = 1	10 ft ²			
	Eg					
	▲					
	-01 CP1			Ch2 sdiy 629		
	8			8.		
			11'			
Panel construction						
Nominal stud size		2" x 4"				
Dressed stud size		1.5" x 3.5"				
Cross-sectional area of studs		As = 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	5"			
Cross-sectional area of end po	sts	Ae = 15.75 ir	1 ²			
Hole diameter		Dia = 1 in				
Net cross-sectional area of end	posts	Aen = 11.25 i	n²			
Nominal collector size		2 x 2" x 4"				
Dressed collector size		2 x 1.5" x 3.5	5"			
Service condition		Dry				
Temperature Vertical anchor stiffness		100 degF or				
venucal anchor stittness		ka = 80000 ll	חו/כ			

Tekla Tedds Fast + Epp	Project Yaroslavsky	y Residence			Job Ref. 8119			
323 Dean Street, Suite #3	Section				Sheet no./rev.			
Brooklyn, NY 11217	Wood Shea	ar Wall - Supp. Up	per Level Wall	9	2			
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date		
From NDS Supplement Table Species, grade and size classifi		-	or visually gra Larch, no.2 gra		=	" thick)		
Specific gravity	cation	G = 0.50	Laron, no.2 gra					
Tension parallel to grain		Ft = 575 lb/ir	1 ²					
Compression parallel to grain		Fc = 1350 lb/	/in²					
Modulus of elasticity		E = 160000) lb/in ²					
Minimum modulus of elasticity		Emin = 58000	0 lb/in ²					
Sheathing details								
Sheathing material		15/32'' woo	d panel 3-ply	plywood she	athing			
Fastener type		8d commor	n nails at 2"ce	nters				
From SDPWS Table 4.3A Non Nominal unit shear capacity		-		hear Walls - V	Wood-based Pa	nels		
Nominal unit shear capacity		0						
Apparent shear wall shear st	-	$G_a = 20 \text{ kips}$						
		Ga – 20 Kip:	5/111					
Combined unit shear capaciti Combined nominal unit shea		seismic design						
Combined norminal and Shee		$V_{sc} = 2 * V_{s}$	– 2560 lh/ft					
Combined nominal unit shea	r canacity for y		- 2300 10/11					
Combined norminal and Shee		$V_{wc} = 2 * V_{w}$	– 3580 lb/ft					
Combined apparent shear w	all shear stiffne			/in				
Loading details								
Self weight of panel		Swt = 12 lb/ft	2					
In plane seismic load acting at h	nead of panel	Eq = 9492.3	E ₉ = 9492.3 lbs					
Design spectral response accel	. par., short perio	ods SDS = 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			0.5Lf + 0.5(Lr o	r S or R)				
Load combination no.3		1.2D + E + 0	0.5Lf + 0.7S					
Load combination no.4		0.9D + W						
Load combination no.5		0.9D + E						
Adjustment factors								
Format conversion factor for ter	ision – Table N1							
Format conversion factor for co	moression - Tak	KFt = 2.70						
	mpression – Tal	KFc = 2.40						
Format conversion factor for mo	odulus of elastici							
		KFE = 1.76						
Resistance factor for tension -	Table N2	$\phi t = 0.80$						
Resistance factor for compressi	on – Table N2	$\phi_{\rm c} = 0.90$						
Resistance factor for modulus of		ble N2						
		$\phi_s = 0.85$						
Time effect factor – Table N3		$\lambda = 1.00$						

Fast + Epp	Project Yaroslavsky	y Residence	Job Ref. 8119						
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	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date			
Sheathing resistance factor		φ D = 0.80							
Size factor for tension – Table 4	A	C _{Ft} = 1.50							
Size factor for compression – Ta		CFc = 1.15							
Wet service factor for tension –		C _{Mt} = 1.00							
Wet service factor for compress	ion – Table 4A	C _{Mc} = 1.00							
Wet service factor for modulus of	of elasticity – Ta	ble 4A							
		CME = 1.00							
Temperature factor for tension -	- Table 2.3.3	Ctt = 1.00							
Temperature factor for compres	sion – Table 2.3	3.3							
		$C_{tc} = \textbf{1.00}$							
Temperature factor for modulus	of elasticity - T	able 2.3.3							
		CtE = 1.00							
Incising factor – cl.4.3.8		Ci = 1.00							
Buckling stiffness factor - cl.4.4	.2	CT = 1.00							
Adjusted modulus of elasticity		Emin' = Emin *	Кге * фs * Сме *	CtE * Ci * CT =	870000 psi				
Critical buckling design value		F _{cE} = 0.822 >	< Emin' / (h / d)2	= 608 psi					
Reference compression design	value	$F_{c}^{*} = F_{c} * K_{Fc}$	* фс * λ * Смс *	$C_{tc} * C_{Fc} * C_i =$	3353 psi				
For sawn lumber		c = 0.8							
Column stability factor - eqn	.3.7-1	C _P = (1 + (F		< c) – √([(1 + ((FcE / Fc*)) / (2 >	< c)] ² - (F _{cE}			
		$F_{c}^{*}) / c) = 0$	17						
From SDPWS Table 4.3.4 Max	imum Shear W	all Aspect Ratio							
Maximum shear wall aspect rati		3.5	•						
Shear wall length	0	b = 11 ft							
		$\mathbf{b} = 11$ is							
U U		h / b = 0.909							
Shear wall aspect ratio	h.	h / b = 0.909							
Shear wall aspect ratio Segmented shear wall capacit	-								
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under	seismic loadin	g Vs_max = Eq	= 9.492 kips	kips					
Shear wall aspect ratio Segmented shear wall capacit	seismic loadin	$g V_{s_max} = E_q$ $V_s = \phi_D * v_{sc}$	= 9.492 kips * b = 22.528 k	kips					
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under	seismic loadin	g V _{s_max} = Eq V _s = φ _D * V _s V _{s_max} / V _s =	= 9.492 kips s * b = 22.528 k 0.421	-	exceeds maxim	um shear f			
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo	seismic loadin ading	g V _{s_max} = Eq V _s = φ _D * V _s V _{s_max} / V _s =	= 9.492 kips s * b = 22.528 k 0.421	-	exceeds maxim	um shear f			
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo Chord capacity for chords 1 a	seismic loadin ading	g $V_{s_max} = E_q$ $V_s = \phi D^* V_{s_max} / V_s =$ PASS - Shea	= 9.492 kips * b = 22.528 k 0.421 ar capacity for	-	exceeds maxim	um shear f			
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo Chord capacity for chords 1 a Shear wall aspect ratio	seismic loadin ading	g V _{s_max} = Eq V _s = φ _D * V _s V _{s_max} / V _s =	= 9.492 kips * b = 22.528 k 0.421 ar capacity for	-	exceeds maxim	um shear f			
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo Chord capacity for chords 1 a Shear wall aspect ratio Load combination 5	seismic loadin ading nd 2	g $V_{s_max} = E_q$ $V_s = \phi D * V_{s_max}$ $V_{s_max} / V_s =$ PASS - Sheat h / b = 0.909	= 9.492 kips s * b = 22.528 k 0.421 ar capacity for	-	exceeds maxim	um shear f			
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo Chord capacity for chords 1 a Shear wall aspect ratio Load combination 5 Shear force for maximum tension	seismic loadin ading nd 2	g $V_{s_max} = E_q$ $V_s = \phi_D * V_{s_max}$ $V_{s_max} / V_s =$ PASS - Shea h / b = 0.909 $V = E_q = 9.4$	= 9.492 kips * b = 22.528 k 0.421 ar capacity for 02 kips	-	exceeds maxim	um shear fi			
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo Chord capacity for chords 1 a Shear wall aspect ratio Load combination 5 Shear force for maximum tensio Axial force for maximum tensio	seismic loadin ading nd 2	g $V_{s_max} = E_q$ $V_s = \phi_D * V_{s_max} / V_s =$ PASS - Shea h / b = 0.909 $V = E_q = 9.44$ P = 0 kips =	= 9.492 kips * b = 22.528 k 0.421 ar capacity for 92 kips = 0 kips	seismic load	exceeds maxim	um shear f			
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo Chord capacity for chords 1 a Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension	seismic loadin ading nd 2 n sion	g $V_{s_max} = E_q$ $V_s = \phi_D * V_{s_max} / V_s =$ PASS - Shea h / b = 0.909 $V = E_q = 9.4$ P = 0 kips = T = V * h / (b	= 9.492 kips * b = 22.528 k 0.421 ar capacity for 92 kips = 0 kips) - P = 8.629 ki	seismic load	exceeds maxim	um shear fi			
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo Chord capacity for chords 1 a Shear wall aspect ratio Load combination 5 Shear force for maximum tensic Axial force for maximum tensic Maximum tensile force in chord Maximum applied tensile stress	seismic loadin ading nd 2 n sion	g $V_{s_max} = E_q$ $V_s = \phi_D * V_{s_max}$ $V_{s_max} / V_s =$ PASS - Sheat h / b = 0.909 $V = E_q = 9.44$ P = 0 kips = $T = V * h / (b_{s_max})$	= 9.492 kips * b = 22.528 k 0.421 ar capacity for 02 kips = 0 kips) - P = 8.629 ki 767 lb/in ²	seismic load		um shear f			
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo Chord capacity for chords 1 a Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension	seismic loadin ading nd 2 n sion	$g V_{s_max} = E_q$ $V_s = \phi_D * V_{sq}$ $V_{s_max} / V_s =$ PASS - Sheat $h / b = 0.909$ $V = E_q = 9.44$ $P = 0 \text{ kips} =$ $T = V * h / (bt)$ $f_t = T / A_{en} =$ $F_t' = F_t * K_{Ft} * K_{Ft}$	= 9.492 kips • b = 22.528 k 0.421 ar capacity for 02 kips = 0 kips) - P = 8.629 ki 767 lb/in ² • φt * λ * CMt * C	seismic load		um shear f			
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo Chord capacity for chords 1 a Shear wall aspect ratio Load combination 5 Shear force for maximum tensic Axial force for maximum tensic Maximum tensile force in chord Maximum applied tensile stress	seismic loadin ading nd 2 n sion	$\begin{array}{ll} g & V_{s_max} = E_q \\ V_s = \phi_D * V_{sq} \\ V_{s_max} / V_s = \\ PASS - Shea \\ h / b = 0.909 \\ V = E_q = 9.44 \\ P = 0 \ kips = \\ T = V * h / (b \\ f_t = T / A_{en} = \\ F_t' = F_t * K_{Ft} * \\ f_t / F_t' = 0.412 \end{array}$	= 9.492 kips = * b = 22.528 k 0.421 ar capacity for 02 kips = 0 kips) - P = 8.629 ki 767 lb/in ² * φt * λ * CMt * C 2	seismic load o ps tt * CFt * Ci = 18	1 63 lb/in²				
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo Chord capacity for chords 1 a Shear wall aspect ratio Load combination 5 Shear force for maximum tensic Axial force for maximum tensic Maximum tensile force in chord Maximum applied tensile stress	seismic loadin ading nd 2 n sion	$\begin{array}{ll} g & V_{s_max} = E_q \\ V_s = \phi_D * V_{sq} \\ V_{s_max} / V_s = \\ PASS - Shea \\ h / b = 0.909 \\ V = E_q = 9.44 \\ P = 0 \ kips = \\ T = V * h / (b \\ f_t = T / A_{en} = \\ F_t' = F_t * K_{Ft} * \\ f_t / F_t' = 0.412 \end{array}$	= 9.492 kips = * b = 22.528 k 0.421 ar capacity for 02 kips = 0 kips) - P = 8.629 ki 767 lb/in ² * φt * λ * CMt * C 2	seismic load o ps tt * CFt * Ci = 18					
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo Chord capacity for chords 1 a Shear wall aspect ratio Load combination 5 Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 3	seismic loadin ading nd 2 n sion	g $V_{s_max} = E_q$ $V_s = \phi D * V_{s_q}$ $V_{s_max} / V_s =$ PASS - Sheat h / b = 0.909 $V = E_q = 9.44$ P = 0 kips = T = V * h / (bt) $f_t = T / A_{en} =$ $F_t' = F_t * K_{F_t} *$ $f_t / F_t' = 0.412$ PASS - Des	= 9.492 kips = b = 22.528 k 0.421 ar capacity for 22 kips = 0 kips) - P = 8.629 ki 767 lb/in ² $f \phi_{i} * \lambda * C_{Mt} * C_{2}$ gn tensile stree	seismic load o ps tt * CFt * Ci = 18	1 63 lb/in²				
Shear wall aspect ratio Segmented shear wall capacit Maximum shear force under Shear capacity for seismic lo Chord capacity for chords 1 a Shear wall aspect ratio Load combination 5 Shear force for maximum tensic Axial force for maximum tensic Maximum tensile force in chord Maximum applied tensile stress Design tensile stress	seismic loadin ading nd 2 n sion	$\begin{array}{llllllllllllllllllllllllllllllllllll$	= 9.492 kips = * b = 22.528 k 0.421 ar capacity for 02 kips = 0 kips) - P = 8.629 ki 767 lb/in ² * φt * λ * CMt * C 2 gn tensile stree 02 kips	seismic load o ps tt * CFt * Ci = 18 ess exceeds m	1 63 lb/in²	d tensile st			

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323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Wood Shea	ar Wall - Supp. Up	per Level Wall	9	Sheet no./rev. 4	Sheet no./rev.			
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date			
Maximum applied compressive	e stress	$f_c = C / A_e = S$	555 lb/in ²						
Design compressive stress		Fc' = Fc * KFc	* фс * λ * Смс *	Ctc * CFc * Ci *	CP = 584 lb/in ²				
		fc / Fc' = 0.95	fc / Fc' = 0.951						
	PASS -	Design compres	sive stress ex	ceeds maxim	um applied com	pressive stre			
Hold down force									
Chord 1		T1 = 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		ν _{δs} = V _{δs} / b = 862.94 lb/ft							
Anchor tension force		T _δ = max(0 kips,v _{δs} * h) = 8.629 kips							
Shear wall elastic deflection -	Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_{ac}) + h * T_{\delta} / (k_a * b) = 0.33$							
		in							
Deflection ampification factor		$C_{d\delta} = 4$							
Seismic importance factor		le = 1							
Amp. seis. deflection – ASCE7	′ Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{sws}$	wse / le = 1.355	in					
		δ_{sws} / Δ_{s_allow} =	= 0.564						
			PASS - She	ar wall deflec	tion is less than	deflection lin			

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323 Dean Street, Suite #3 Brooklyn, NY 11217	Section				Sheet no./rev.	
,,,		ar Wall - Supp. Up			1	
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date
WOOD SHEAR WALL DESIG	N (NDS)					
In accordance with NDS2018	load reduction	factor design and	d the segment	ed shear wal		culation version
Panel details						
Structural wood panel sheathin	ig on both sides					
Panel height		h = 10 ft				
Panel length Total area of wall		b = 5.833 ft	0 222 42			
lotal area of wall		A = h * b = 58	5.333 II ²			
	<u> </u>	E _q				
	Ī	s1				
	10'-					
		Ch1	CI			
	<u> </u>		┘┘───┤┘───╃┘┘			
		Kips	Kips			
		.63 kips	63 kips			
		œ	œ			
		L	40"			
		▲ 5'	10"			
Panel construction						
Nominal stud size		2" x 6"				
Dressed stud size		1.5" x 5.5"				
Cross-sectional area of studs		As = 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 po	sts	Ae = 16.5 in ²				
Hole diameter		Dia = 1 in				
Net cross-sectional area of end	d posts	A _{en} = 13.5 in ²	2			
Nominal collector size		2 x 2" x 6"				
Dressed collector size		2 x 1.5" x 5.5)"			
Service condition		Dry				
Temperature Vertical anchor stiffness		100 degF or ka = 80000 lb				

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323 Dean Street, Suite #3 Brooklyn, NY 11217	Section						
2.00xyn, 111 11217		Wall - Supp. Up	-		2		
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date	
From NDS Supplement Table Species, grade and size classifi		-	or visually gra Larch, no.2 gra		-	" thick)	
Specific gravity	callon	G = 0.50	Laich, no.2 gra				
Tension parallel to grain		Ft = 575 lb/ir) ²				
Compression parallel to grain		Fc = 1350 lb/	/in²				
Modulus of elasticity		E = 160000) lb/in ²				
Minimum modulus of elasticity		Emin = 58000	0 lb/in ²				
Sheathing details							
Sheathing material			d panel 3-ply		athing		
Fastener type		8d commor	n nails at 2"ce	nters			
From SDPWS Table 4.3A Nom		-		hear Walls -	Wood-based Pa	nels	
Nominal unit shear capacity		•					
Nominal unit shear capacity	-						
Apparent shear wall shear st		Ga = 20 kip:	S/IN				
Combined unit shear capaciti							
Combined nominal unit shea	r capacity for se	-					
		$V_{sc} = 2 * V_{s}$	= 2560 ID/II				
Combined nominal unit shea	r capacity for w	vwc = 2 * vw	- 2500 lb/ft				
Combined apparent shear w	all shear stiffne:			/in			
Loading details							
Self weight of panel		Swt = 12 lb/ft	2				
In plane seismic load acting at h	nead of panel	E _q = 5034 lbs					
Design spectral response accel	par., short perio	ds SDS = 0.944					
From IBC 2018 cl.1605.2							
Load combination no.1		•	r or S or R) + 0				
Load combination no.2			0.5Lf + 0.5(Lr o	r S or R)			
Load combination no.3		1.2D + E + 0	0.5Lf + 0.7S				
Load combination no.4 Load combination no.5		0.9D + W					
		0.9D + E					
Adjustment factors Format conversion factor for ter	sion - Table N4						
		KFt = 2.70					
Format conversion factor for con	mpression – Tabl						
		KFc = 2.40					
Format conversion factor for mo	dulus of elasticity	/ – Table N1					
		Kfe = 1.76					
Resistance factor for tension -		$\varphi_t = \textbf{0.80}$					
Resistance factor for compressi		φ _c = 0.90					
Resistance factor for modulus o	f elasticity – Tabl						
		φs = 0.85					

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Brooklyn, NY 11217	Wood Shea	Wood Shear Wall - Supp. Upper Level Wall 10 3							
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date			
Sheathing resistance factor		φD = 0.80							
Size factor for tension – Table 4	A	CFt = 1.30							
Size factor for compression – Ta	able 4A	CFc = 1.10							
Wet service factor for tension -	Table 4A	C _{Mt} = 1.00							
Wet service factor for compress	on - Table 4A	C _{Mc} = 1.00							
Wet service factor for modulus of	of elasticity – Ta	able 4A							
		CME = 1.00							
Temperature factor for tension -	- Table 2.3.3	Ctt = 1.00							
Temperature factor for compres	sion – Table 2.3	3.3							
		Ctc = 1.00							
Temperature factor for modulus	of elasticity – T	able 2.3.3							
		CtE = 1.00							
Incising factor – cl.4.3.8		Ci = 1.00							
Buckling stiffness factor - cl.4.4	.2	C⊤ = 1.00							
Adjusted modulus of elasticity		Emin' = Emin *	Кге * фs * Сме *	CtE * Ci * CT =	870000 psi				
Critical buckling design value		F _{cE} = 0.822 >	× E _{min} ' / (h / d)²	= 1502 psi					
Reference compression design	value	$F_{c}^{*} = F_{c} * K_{Fc}$	с*фс*λ*Смс*	Ctc * CFc * Ci =	3208 psi				
For sawn lumber		c = 0.8							
Column stability factor - eqn.	3.7-1	C _P = (1 + (F	FcE / Fc*)) / (2 :	< c) – √([(1 +	(FcE / Fc*)) / (2 >	< c)]² - (F c∈			
		$F_{c^{*}}) / c) = 0$.41						
From SDPWS Table 4.3.4 Max	imum Shoar W	all Aspect Pation	-						
Maximum shear wall aspect ratio		3.5	5						
Shear wall length	5	b = 5.833 ft							
Shear wall aspect ratio		h / b = 1.714	L						
Segmented shear wall capacit Maximum shear force under	-	ig Vs_max = Eq	= 5.034 kips						
Shear capacity for seismic lo		-	∞ * b = 11.947 ∣	kips					
	9	Vs_max / Vs =							
				seismic load	exceeds maxim	um shear f			
Chard appeality for shareds 4 -	nd 2								
Chord capacity for chords 1 a Shear wall aspect ratio		h / b = 1.714	l						
Load combination 5		11/0 = 1.714	,						
Shear force for maximum tensio	n		34 kine						
Axial force for maximum tension			V = E _q = 5.034 kips P = 0 kips = 0 kips						
Maximum tensile force in chord		•) - P = 8.630 ki	ns					
Maximum applied tensile stress		ft = T / Aen =		49					
Design tensile stress			639 ID/III≏ *φt *λ * C∧t * C		515 lh/in ²				
Design tensile stress		Ft = Ft K + t ft / Ft' = 0.39							
				ass evreeds n	naximum applie	d tensile st			
Load combination 3		1 400 - 662	IST CHOICE SUR			G CHOID SL			
Shear force for maximum compl	ession	V = E _q = 5.0 3	34 kips						
Axial force for maximum com			•	S _{DS} * S _{wt} * h) *	*s/2= 0.111 ki	ps			
Maximum compressive force in	-	,) + P = 8.741 ⊦		-,	- -			
		0 - v 117 (r	·/ · · · · · · · · · · · · · · ·						

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323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Wood Shea	ar Wall - Supp. Upp	per Level Wall	10	Sheet no./rev. 4				
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date			
Maximum applied compressive	e stress	fc = C / Ae = 5	30 lb/in ²						
Design compressive stress		$F_{c}' = F_{c} * K_{Fc}$	* фс * λ * Смс *	Ctc * CFc * Ci *	C _P = 1318 lb/in ²				
		fc / Fc' = 0.40	fc / Fc' = 0.402						
	PASS - I	Design compress	ive stress ex	ceeds maxim	um applied com	oressive stre			
Hold down force									
Chord 1		T1 = 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		vδs = Vδs / b = 862.98 lb/ft							
Anchor tension force		T _δ = max(0 kips,v _{δs} * h) = 8.630 kips							
Shear wall elastic deflection -	Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_{ac}) + h * T_{\delta} / (k_a * b) = 0.44$							
		in							
Deflection ampification factor		$C_{d\delta} = 4$							
Seismic importance factor		le = 1							
Amp. seis. deflection – ASCE7	7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{sws}$	wse / Ie = 1.782	in					
		δ sws / Δ s_allow =	0.742						
			PASS - She	ar wall deflec	tion is less than	deflection lin			

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323 Dean Street, Suite #3 Brooklyn, NY 11217	Section				Sheet no./rev	
		r Wall - Supp. Up			1	
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date
WOOD SHEAR WALL DESIG	N (NDS)					
In accordance with NDS2018	load reduction f	actor design an	d the segmen	ted shear wall		culation versior
Panel details						
Structural wood panel sheathin	ng on one side					
Panel height		h = 10 ft				
Panel length		b = 11 ft				
Total area of wall	F	A = h * b = 1	10 ft ²			
	s1					
	-					
		1		Ch2		
	482 kips			482 kips		
	22 k			32 k		
	4			4		
	4	·	11'	►		
Panel construction						
Nominal stud size		2" x 6"				
Dressed stud size		2 x 0 1.5" x 5.5"				
Cross-sectional area of studs		As = 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 po	ists	A _e = 16.5 in ²				
Hole diameter		Dia = 1 in				
Net cross-sectional area of end	d 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	5"			
Service condition		Dry	-			
Temperature		100 degF or	1000			
			1622			

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Brooklyn, NY 11217	Wood Shear	r Wall - Supp. Up	per Level Wall	2	2			
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date		
From NDS Supplement Table 4 Species, grade and size classific Specific gravity Tension parallel to grain Compression parallel to grain Modulus of elasticity Minimum modulus of elasticity Sheathing details Sheathing material Fastener type From SDPWS Table 4.3A Nomi Nominal unit shear capacity fo Nominal unit shear capacity fo Apparent shear wall shear stif Loading details	ation nal Unit Shear or seismic des or wind design	Douglas Fir-I G = 0.50 Ft = 575 lb/in Fc = 1350 lb/ E = 1600000 Emin = 58000 15/32'' woo 8d common Capacities for V ign vs = 980 lb/f	_arch, no.2 gra ² in ² Ib/in ² 0 Ib/in ² d panel 3-ply nails at 3''ce Vood-Frame S t /ft	de, 2" & wider plywood she nters	athing			
Self weight of panel In plane seismic load acting at he Design spectral response accel.		$S_{wt} = 12 \text{ lb/ft}^2$ $E_q = 4930 \text{ lb}^2$ ids $S_{DS} = 0.944$						
From IBC 2018 cl.1605.2								
Load combination no.1		1.2D + 1.6(L	or S or R) + 0	.5W				
Load combination no.2		1.2D + W + ().5Lf + 0.5(Lr o	r S or R)				
Load combination no.3		1.2D + E + 0	.5Lf + 0.7S					
Load combination no.4		0.9D + W	0.9D + W					
Load combination no.5		0.9D + E	0.9D + E					
Adjustment factors								
Format conversion factor for tens	sion – Table N1							
_		K _{Ft} = 2.70						
Format conversion factor for com	ipression – Tab							
Format conversion factor for mod	tulue of alasticity	KFc = 2.40						
		KFE = 1.76						
Resistance factor for tension – T	ahla N2	here = 1.76 here = 0.80						
		·						
Resistance factor for compression		φc = 0.90						
Resistance factor for modulus of	elasticity – Tab	φs = 0.85						
Time offect feator Table N2								
Time effect factor – Table N3		$\lambda = 1.00$						
Sheathing resistance factor		φD = 0.80						
Size factor for tension – Table 4/		C _{Ft} = 1.30						
Size factor for compression – Ta		CFc = 1.10						
Wet service factor for tension –		C _{Mt} = 1.00						
Wet service factor for compression		Смс = 1.00						
Wet service factor for modulus of	elasticity – I al	DIE 4A						

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Temperature factor for tension – T	able 2.3.3	C _{tt} = 1.00		I					
Temperature factor for compression									
		Ctc = 1.00							
Temperature factor for modulus of	elasticity – T								
		CtE = 1.00							
Incising factor – cl.4.3.8		Ci = 1.00 CT = 1.00							
Buckling stiffness factor – cl.4.4.2			K * + * Cu- *	· · · · · · · · · · ·	97000 poi				
Adjusted modulus of elasticity			КFE * фs * Сме *		670000 psi				
Critical buckling design value			× Emin' / (h / d)²	-					
Reference compression design va	lue		с*фс*λ*Смс*	$C_{tc} * C_{Fc} * C_i =$	= 3208 psi				
For sawn lumber		c = 0.8			/ _ / _ \\ /				
Column stability factor – eqn.3.	7-1		<i>,,</i> , , , , , , , , , , , , , , , , , ,	× c) – √([(1 +	(F _{cE} / F _c *)) / (2	× C)] ² - (FcE			
		F _c *) / c) = 0	.41						
From SDPWS Table 4.3.4 Maxim	um Shear W	all Aspect Ratio	s						
Maximum shear wall aspect ratio		3.5							
Shear wall length		b = 11 ft							
Shear wall aspect ratio		h / b = 0.90	h / b = 0.909						
Segmented shear wall capacity									
Maximum shear force under se	eismic loadin	Ig Vs_max = Eq	= 4.93 kips						
Shear capacity for seismic load	ding	$V_s = \phi D * V_s$	$V_{s} = \phi_{D} * v_{s} * b = 8.624 \text{ kips}$						
		$V_{s_max} / V_s =$	0.572						
		PASS - She	ar capacity for	seismic load	exceeds maxim	num shear f			
Chord capacity for chords 1 and	12								
Shear wall aspect ratio Load combination 5		h / b = 0.90)						
Shear force for maximum tension		V = E _q = 4.9	3 kips						
Axial force for maximum tensic	n	P = 0 kips :	P = 0 kips = 0 kips						
Maximum tensile force in chord		T = V * h / (t	T = V * h / (b) - P = 4.482 kips						
Maximum applied tensile stress			ft = T / Aen = 332 lb/in ²						
Design tensile stress			$F_t' = F_t * K_{Ft} * \varphi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i = 1615 \text{ lb/in}^2$						
		ft / Ft' = 0.20							
		PASS - Des	ign tensile stre	ess exceeds r	naximum applie	d tensile s			
Load combination 3		\ / = /-	o 1 [.]						
Shear force for maximum comprese Axial force for maximum comp			V = Eq = 4.93 kips P = (1.2 * Swt * h + 0.2 * Sps * Swt * h) * s / 2 = 0.111 kips						
Maximum compressive force in ch			b) + P = 4.593 k		372 - 0.111 K	,ho			
Maximum applied compressive str		C = V H / (I) f _c = C / A _e =		viha					
Design compressive stress					CP = 1318 lb/in ²				
2 coign compressive stress		$f_c / F_c' = 0.2^{\circ}$	-						
	PASS -	Design compres		ceeds maxim	um applied com	pressive st			
Hold down force									
Chord 1		T₁ = 4.482 k	ips						
		$T_2 = 4.482$ k	-						

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Seismic o	deflection
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Design shear force	V _{ðs} = E _q = 4.93 kips
Deflection limit	$\Delta_{s_allow} = 0.020 * h = 2.4 in$
Induced unit shear	v _{ðs} = V _{ðs} / b = 448.18 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * 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	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e = 1.448$ in
	$\delta_{sws} / \Delta_{s_allow} = 0.603$

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WOOD SHEAR WALL DES	GIGN (NDS)					
In accordance with NDS20	18 load reduction	factor design an	d the segment	ed shear wall r		
Panel details					Tedds calcu	Jiation versio
Structural wood panel shear	hing on one side					
Panel height		h = 12 ft				
Panel length		b = 29 ft				
Total area of wall		A = h * b = 3	48 ft ²			
s1						
12						
Ch	1				Ch2	
					sd	
.68 kips					1.68 kips	
, 					, (
 			29'			
Panel construction		0.11 0.11				
Nominal stud size		2" x 6" 1 5" x 5 5"				
Dressed stud size	le.	1.5" x 5.5" As = 8.25 in²				
Cross-sectional area of stud Stud spacing	12	As = 8.23 in ² s = 16 in				
Nominal end post size		s = 16 m 2 x 2" x 6"				
Dressed end post size		2 x 1.5" x 5.5	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	5"			
Service condition		Dry				
Temperature		100 degF or	less			
Vertical anchor stiffness		ka = 80000 lk	o/in			
From NDS Supplement Ta	ble 4A - Reference	e design values fo	or visually gra	ded dimension	lumber (2" - 4"	thick)
Species, grade and size cla	ssification	Douglas Fir-	Larch, no.2 gra	de, 2" & wider		
Specific gravity		G = 0.50				
Tension parallel to grain		Ft = 575 lb/in				

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Compression parallel to grain		Fc = 1350 lb.	/in ²					
Modulus of elasticity		E = 160000) lb/in ²					
Minimum modulus of elasticity		Emin = 58000	0 lb/in ²					
Sheathing details								
Sheathing material		15/32" woo	d panel 3-ply	plywood she	athing			
Fastener type		8d commor	n nails at 3"ce	enters				
From SDPWS Table 4.3A Nor	ninal Unit Shear	Capacities for \	Nood-Frame S	Shear Walls - V	Nood-based Pa	nels		
Nominal unit shear capacity		-						
Nominal unit shear capacity		•						
Apparent shear wall shear s	•	Ga = 15 kip:						
			e, II I					
Loading details		S _{wt} = 12 lb/ft	2					
Self weight of panel In plane seismic load acting at	head of nanel	$S_{wt} = 12 \text{ ID/II}$ Eq = 4060 lb						
Design spectral response acce	-		3					
	i. par., short pene							
From IBC 2018 cl.1605.2 Load combination no.1		1 2D ± 1 6/1	.r or S or R) + 0	5\\/				
Load combination no.2		-	0.5Lf + 0.5(Lr o					
Load combination no.3		1.2D + E + C	-					
Load combination no.4		0.9D + W						
Load combination no.5		0.9D + E						
Adjustment factors								
Format conversion factor for te	nsion – Table N1							
		KFt = 2.70						
Format conversion factor for co	ompression – Tab	ole N1						
		KFc = 2.40						
Format conversion factor for m	odulus of elasticit	•						
	T	KFE = 1.76						
Resistance factor for tension –			$\phi_t = 0.80$					
Resistance factor for compress		$\phi_c = 0.90$						
Resistance factor for modulus	or elasticity – Tac							
Time effect factor – Table N3		$φ_s = 0.85$ λ = 1.00						
Sheathing resistance factor Size factor for tension – Table	10	φ D = 0.80 CFt = 1.30						
Size factor for compression – Table		$C_{Fc} = 1.30$ $C_{Fc} = 1.10$						
Wet service factor for tension -		Сгс – 1.10 Смt = 1.00						
Wet service factor for compres		$C_{Mc} = 1.00$						
Wet service factor for modulus								
		Сме = 1.00						
Temperature factor for tension	– Table 2.3.3	$C_{tt} = \textbf{1.00}$						
Temperature factor for compre	ssion – Table 2.3	.3						

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		CtE = 1.00									
Incising factor – cl.4.3.8		Ci = 1.00									
Buckling stiffness factor - cl.4.4.	.2	C⊤ = 1.00									
Adjusted modulus of elasticity		Emin' = Emin *	Кге * фs * Сме `	* CtE * Ci * CT =	870000 psi						
Critical buckling design value		$F_{cE} = 0.822$	< Emin' / (h / d)²	= 1043 psi							
Reference compression design	value	$F_{c}^{*} = F_{c} * K_{Fc}$	с*фс*λ*Смс*	* Ctc * CFc * Ci =	= 3208 psi						
For sawn lumber		c = 0.8									
Column stability factor - eqn.	3.7-1	CP = (1 + (F	F _{cE} / F _c *)) / (2	× c) – √([(1 +	(F _{cE} / F _c *)) / (2 >	× c)] ² - (F _{cE}					
		Fc*) / c) = 0	.30								
From SDPWS Table 4.3.4 Maxi	imum Shear Wa	all Aspect Ratio	5								
Maximum shear wall aspect ratio		3.5									
Shear wall length		b = 29 ft									
Shear wall aspect ratio		h / b = 0.414									
Segmented shear wall capacit	у										
Maximum shear force under	seismic loading	g Vs_max = Eq	= 4.06 kips								
Shear capacity for seismic loa	ading	Vs = $\phi_D * v_s$	* b = 22.736 k	kips							
	-	Vs_max / Vs =	0.179								
		PASS - Shea	ar capacity for	r seismic load	exceeds maxim	num shear f					
Chord capacity for chords 1 a	nd 2										
Shear wall aspect ratio		h / b = 0.414									
Load combination 5											
Shear force for maximum tensio	n	V = Eq = 4.0	6 kips								
Axial force for maximum tens	ion	P = 0 kips =	P = 0 kips = 0 kips								
				•	T = V * h / (b) - P = 1.680 kips						
Maximum tensile force in chord		•) - P = 1.680 k	lips							
Maximum tensile force in chord Maximum applied tensile stress		•		ups							
		T = V * h / (b ft = T / A _{en} =	124 lb/in ²	tips Ctt * CFt * Ci = 1 0	615 lb/in²						
Maximum applied tensile stress		T = V * h / (b) ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.07	124 lb/in² * фt * λ * Смt * С 7	$C_{tt} * C_{Ft} * C_i = 10$							
Maximum applied tensile stress Design tensile stress		T = V * h / (b) ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.07	124 lb/in² * фt * λ * Смt * С 7	$C_{tt} * C_{Ft} * C_i = 10$	615 lb/in² naximum applie	d tensile st					
Maximum applied tensile stress Design tensile stress Load combination 3		T = V * h / (b ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.07 PASS - Des	124 lb/in² * φι * λ * C _{Mt} * C 7 ign tensile str	$C_{tt} * C_{Ft} * C_i = 10$		d tensile st					
Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum compr	ression	T = V * h / (b) ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.07 PASS - Des V = Eq = 4.0	124 lb/in ² * φι * λ * C _M ι * C 7 ign tensile str 6 kips	Ctt * CFt * Ci = 1 ess exceeds r	naximum applie						
Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum compr Axial force for maximum com	ression	T = V * h / (b) ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.07 PASS - Des $V = E_q = 4.0$ P = (1.2 * S	124 lb/in ² * φt * λ * Cmt * C 7 ign tensile str 6 kips wt * h + 0.2 * 5	Ctt * CFt * Ci = 1 ress exceeds r Sds * Swt * h)							
Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum compr Axial force for maximum com Maximum compressive force in o	ression apression chord	T = V * h / (b) ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.07 PASS - Des V = Eq = 4.0 P = (1.2 * S) C = V * h / (b)	124 lb/in ² * φι * λ * C _M ι * C ign tensile str 6 kips wι * h + 0.2 * 5 b) + P = 1.813	Ctt * CFt * Ci = 1 ress exceeds r Sds * Swt * h)	naximum applie						
Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum compr Axial force for maximum com Maximum compressive force in o Maximum applied compressive st	ression apression chord	T = V * h / (b) ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.07 PASS - Des $V = E_q = 4.0$ P = (1.2 * S) C = V * h / (b) fc = C / Ae =	 124 lb/in² φt * λ * CMt * C 7 ign tensile str 6 kips iwt * h + 0.2 * 3) + P = 1.813 110 lb/in² 	Ctt * CFt * Ci = 1 ress exceeds r Sds * Swt * h) kips	naximum applie * s / 2 = 0.133 k i						
Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum compr Axial force for maximum com Maximum compressive force in o	ression apression chord	T = V * h / (b) $f_{t} = T / A_{en} =$ $F_{t}' = F_{t} * K_{Ft}$ $f_{t} / F_{t}' = 0.07$ PASS - Des $V = E_{q} = 4.0$ P = (1.2 * S) C = V * h / (b) $f_{c} = C / A_{e} =$ $F_{c}' = F_{c} * K_{Fc}$	124 lb/in ² * φt * λ * CMt * C 7 ign tensile str 6 kips wt * h + 0.2 * 3 b) + P = 1.813 110 lb/in ² * φc * λ * CMc *	Ctt * CFt * Ci = 1 ress exceeds r Sds * Swt * h) kips	naximum applie						
Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum compr Axial force for maximum com Maximum compressive force in o Maximum applied compressive st	ression pression chord stress	T = V * h / (b) $f_{t} = T / A_{en} =$ $F_{t}' = F_{t} * K_{Ft}$ $f_{t} / F_{t}' = 0.07$ PASS - Des $V = E_{q} = 4.0$ P = (1.2 * S) C = V * h / (b) $f_{c} = C / A_{e} =$ $F_{c}' = F_{c} * K_{Fc}$ $f_{c} / F_{c}' = 0.11$	124 lb/in ² * φt * λ * CMt * C 7 ign tensile str 6 kips iwt * h + 0.2 * 3 b) + P = 1.813 1 10 lb/in ² * φc * λ * CMc * 4	Ctt * CFt * Ci = 1 ess exceeds r SDs * Swt * h) kips * Ctc * CFc * Ci *	naximum applie * s / 2 = 0.133 ki CP = 961 lb/in ²	ips					
Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum compr Axial force for maximum com Maximum compressive force in of Maximum applied compressive st Design compressive stress	ression pression chord stress	T = V * h / (b) $f_{t} = T / A_{en} =$ $F_{t}' = F_{t} * K_{Ft}$ $f_{t} / F_{t}' = 0.07$ PASS - Des $V = E_{q} = 4.0$ P = (1.2 * S) C = V * h / (b) $f_{c} = C / A_{e} =$ $F_{c}' = F_{c} * K_{Fc}$ $f_{c} / F_{c}' = 0.11$	124 lb/in ² * φt * λ * CMt * C 7 ign tensile str 6 kips iwt * h + 0.2 * 3 b) + P = 1.813 1 10 lb/in ² * φc * λ * CMc * 4	Ctt * CFt * Ci = 1 ess exceeds r SDs * Swt * h) kips * Ctc * CFc * Ci *	naximum applie * s / 2 = 0.133 k i	ips					
Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum compr Axial force for maximum com Maximum compressive force in o Maximum applied compressive st Design compressive stress Hold down force	ression pression chord stress	$T = V * h / (b)$ $f_{t} = T / A_{en} =$ $F_{t}' = F_{t} * K_{Ft}$ $f_{t} / F_{t}' = 0.07$ $PASS - Des$ $V = E_{q} = 4.0$ $P = (1.2 * S)$ $C = V * h / (b)$ $f_{c} = C / A_{e} =$ $F_{c}' = F_{c} * K_{Fc}$ $f_{c} / F_{c}' = 0.11$ Design compres	124 lb/in ² * $\phi_t * \lambda * C_{Mt} * C_7$ ign tensile str 6 kips $\phi_t * h + 0.2 * S_7$ 7 110 lb/in ² * $\phi_c * \lambda * C_{Mc} * 4$ sive stress ex	Ctt * CFt * Ci = 1 ess exceeds r SDs * Swt * h) kips * Ctc * CFc * Ci *	naximum applie * s / 2 = 0.133 ki CP = 961 lb/in ²	ips					
Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum compr Axial force for maximum comp Maximum compressive force in of Maximum applied compressive st Design compressive stress Hold down force Chord 1	ression pression chord stress	T = V * h / (k ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.07' PASS - Des $V = E_q = 4.0'P = (1.2 * S)$ C = V * h / (k fc = C / Ae = Fc' = Fc * KFc fc / Fc' = 0.11 Design compres T1 = 1.68 kip	124 lb/in ² * $\phi_t * \lambda * C_{Mt} * C_7$ ign tensile str 6 kips Swt * h + 0.2 * 3 b) + P = 1.813 110 lb/in ² * $\phi_c * \lambda * C_{Mc} * 4$ sive stress ex	Ctt * CFt * Ci = 1 ess exceeds r SDs * Swt * h) kips * Ctc * CFc * Ci *	naximum applie * s / 2 = 0.133 ki CP = 961 lb/in ²	ips					
Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum compr Axial force for maximum com Maximum compressive force in o Maximum applied compressive st Design compressive stress Hold down force Chord 1 Chord 2	ression pression chord stress	$T = V * h / (b)$ $f_{t} = T / A_{en} =$ $F_{t}' = F_{t} * K_{Ft}$ $f_{t} / F_{t}' = 0.07$ $PASS - Des$ $V = E_{q} = 4.0$ $P = (1.2 * S)$ $C = V * h / (b)$ $f_{c} = C / A_{e} =$ $F_{c}' = F_{c} * K_{Fc}$ $f_{c} / F_{c}' = 0.11$ Design compres	124 lb/in ² * $\phi_t * \lambda * C_{Mt} * C_7$ ign tensile str 6 kips Swt * h + 0.2 * 3 b) + P = 1.813 110 lb/in ² * $\phi_c * \lambda * C_{Mc} * 4$ sive stress ex	Ctt * CFt * Ci = 1 ess exceeds r SDs * Swt * h) kips * Ctc * CFc * Ci *	naximum applie * s / 2 = 0.133 ki CP = 961 lb/in ²	ips					
Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum compr Axial force for maximum compr Maximum compressive force in of Maximum applied compressive st Design compressive stress Hold down force Chord 1 Chord 2 Seismic deflection	ression pression chord stress	$T = V * h / (b_{ft} = T / A_{en} = F_t' = F_t * K_{Ft}$ $f_t / F_t' = 0.07$ PASS - Des $V = E_q = 4.0$ $P = (1.2 * S)$ $C = V * h / (b_{fc} = C / A_e = F_c' = F_c * K_{Fc}$ $f_c / F_c' = 0.11$ Design compres $T_1 = 1.68 \text{ kip}$ $T_2 = 1.68 \text{ kip}$	124 lb/in ² * $\phi_t * \lambda * C_{Mt} * C_7$ ign tensile str 6 kips iwt * h + 0.2 * 3 b) + P = 1.813 110 lb/in ² * $\phi_c * \lambda * C_{Mc} * 4$ sive stress ex is	Ctt * CFt * Ci = 1 ess exceeds r SDs * Swt * h) kips * Ctc * CFc * Ci *	naximum applie * s / 2 = 0.133 ki CP = 961 lb/in ²	ips					
Maximum applied tensile stress Design tensile stress Load combination 3 Shear force for maximum compr Axial force for maximum com Maximum compressive force in o Maximum applied compressive st Design compressive stress Hold down force Chord 1 Chord 2	ression pression chord stress	T = V * h / (k ft = T / Aen = Ft' = Ft * KFt ft / Ft' = 0.07' PASS - Des $V = E_q = 4.0'P = (1.2 * S)$ C = V * h / (k fc = C / Ae = Fc' = Fc * KFc fc / Fc' = 0.11 Design compres T1 = 1.68 kip	124 lb/in ² * $\phi_t * \lambda * C_{Mt} * C_7$ ign tensile str 6 kips iwt * h + 0.2 * 3 b) + P = 1.813 110 lb/in ² * $\phi_c * \lambda * C_{Mc} * 4$ sive stress ex is	Ctt * CFt * Ci = 1 ess exceeds r SDs * Swt * h) kips * Ctc * CFc * Ci *	naximum applie * s / 2 = 0.133 ki CP = 961 lb/in ²	ips					

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Induced unit shear	$v_{\delta s} = V_{\delta s} / b = 140 \text{ lb/ft}$
Anchor tension force	T _δ = max(0 kips,v _{δs} * 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	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	δ_{sws} = Cd $_{\delta}$ * δ_{swse} / Ie = 0.493 in
	$\delta_{sws} / \Delta_{s_allow} = 0.171$

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WOOD SHEAR WALL DESIGN (NDS)

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

Panel details

Structural wood panel sheathing on both sides

Panel height Panel length

Total area of wall



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	As = 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	ka = 84000 lb/in

Tedds calculation version 1.2.04

Tekla Tedds Fast + Epp	Project Yaroslavsky	Residence		8119			
323 Dean Street, Suite #3	Section		Sheet no./rev.				
Brooklyn, NY 11217	Wood Shear	Wall - Supp. Up	per Level Wall	13	2		
	Wood Shear Wall - Supp. Upper Level Wall 132Calc. by BJWDate 2/23/2021Chk'd byDateApper Apper Appe	App'd by	Date				
Species, grade and size classifie Specific gravity Tension parallel to grain Compression parallel to grain Modulus of elasticity Minimum modulus of elasticity Sheathing details Sheathing material Fastener type From SDPWS Table 4.3A Nom Nominal unit shear capacity for Nominal unit shear capacity for Apparent shear wall shear st	inal Unit Shear (for seismic design for wind design iffness	Douglas Fir- G = 0.50 Ft = 575 lb/ir Fc = 1350 lb/ E = 1600000 Emin = 58000 15/32'' woo 10d commo Capacities for V gn Vs = 1540 lb Vw = 2155 lb	Larch, no.2 gra ¹² ¹ /in ²) Ib/in ² 0 Ib/in ² d panel 3-ply on nails at 2"c Nood-Frame S /ft	de, 2'' & wider plywood she enters	athing		
Combined unit shear capacitie							
Combined nominal unit shea	r capacity for se	-					
			= 3080 lb/ft				
Combined nominal unit shea	r capacity for wi	-					
Combined apparent shear w	all shear stiffnes			/in			
Loading details Self weight of panel		Swt = 12 lb/ft ²	2				
In plane seismic load acting at h	ead of panel	$E_{q} = 6190 \text{ lbs}$					
Design spectral response accel.							
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		-	0.5Lf + 0.5(Lr 0				
Load combination no.3		1.2D + E + 0	-				
Load combination no.4		0.9D + W					
Load combination no.5		0.9D + E					
Adjustment factors							
Format conversion factor for ten	sion – Table N1						
		KFt = 2.70					
Format conversion factor for cor	npression – Tabl	e N1					
		KFc = 2.40					
Format conversion factor for mo	dulus of elasticity						
		Kfe = 1.76					
Resistance factor for tension –		$\varphi t = 0.80$					
Resistance factor for compression		$\varphi_{\rm c}=\boldsymbol{0.90}$					
Resistance factor for modulus of	f elasticity – Tabl	e N2					

Fast + Epp	Project Yaroslavsky	Residence			Job Ref. 8119				
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Brooklyn, NY 11217		Wall - Supp. Up	per Level Wall	13	3				
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date			
Sheathing resistance factor		φ D = 0.80							
Size factor for tension – Table 4		CFt = 1.30							
Size factor for compression – Ta		CFc = 1.10							
Wet service factor for tension –		C _{Mt} = 1.00							
Wet service factor for compress		C _{Mc} = 1.00							
Wet service factor for modulus of		Сме = 1.00							
Temperature factor for tension -	Table 2.2.2	$C_{ME} = 1.00$ $C_{tt} = 1.00$							
Temperature factor for compres									
		$C_{tc} = 1.00$							
Temperature factor for modulus	of elasticity – Ta	ble 2.3.3							
		CtE = 1.00							
Incising factor – cl.4.3.8		Ci = 1.00							
Buckling stiffness factor - cl.4.4	.2	C⊤ = 1.00							
Adjusted modulus of elasticity		Emin' = Emin * KFE *							
Critical buckling design value		$F_{cE} = 0.822 \times E_{min}' / (h / d)^2 = 1502 \text{ psi}$							
Reference compression design	value	$F_{c}^* = F_c * K_{Fc}$	* фс * λ * Смс *	$C_{tc} * C_{Fc} * C_i =$	3208 psi				
For sawn lumber		c = 0.8							
Column stability factor - eqn.	3.7-1	C _P = (1 + (F		(c) – √([(1 + ((FcE / Fc*)) / (2 >	< c)]² - (F cE			
, , ,		F_{c}^{*}) / c) = 0			· · · · ·)] (
From SDPWS Table 4.3.4 Max	imum Shoor Wo	, ,							
Maximum shear wall aspect ratio		3.5	5						
Shear wall length	0	5.5 b = 4.583 ft							
Shear wall aspect ratio		h / b = 2.182							
·		117 6 - 2.102							
Segmented shear wall capacit Maximum shear force under	-	Vs_max = Eq	= 6.19 kips						
Shear capacity for seismic lo	-		-) 125 * h / h ₋)	= 11.037 kips				
Uncar capacity for scisific io	aaniy	•		.120 11/US)	- 11.037 NIPS				
		$V_{s_max} / V_s =$		solemic load	exceeds maxim	um choor f			
		F A33 - 31188	ιι ταματιτή ιθι	SCISITIIC IUdU		un snedi i			
Chord capacity for chords 1 a	nd 2								
Shear wall aspect ratio		h / b = 2.182							
Load combination 5	-) kin a						
Shear force for maximum tensio		$V = E_q = 6.19 \text{ kips}$							
Axial force for maximum tension		P = 0 kips = 0 kips							
		I = V * h / (b) - P = 13.506	kips					
Maximum tensile force in chord		ft = T / A _{en} = 667 lb/in ²							
Maximum applied tensile stress				* 0 * 0 * -					
		$F_t = F_t * K_{Ft}$	[*] φt * λ * Cмt * C	tt * CFt * Ci = 16	615 lb/in ²				
Maximum applied tensile stress		Ft' = Ft * KFt * ft / Ft' = 0.41	^έ φt * λ * Cмt * C 3			d topolla at			
Maximum applied tensile stress		Ft' = Ft * KFt * ft / Ft' = 0.41	^έ φt * λ * Cмt * C 3		15 lb/in² naximum applie	d tensile st			
Maximum applied tensile stress Design tensile stress Load combination 3	ression	Ft' = Ft * KFt * ft / Ft' = 0.41	ኛ φι*λ*C _{Mt} *C 3 gn tensile stre			d tensile st			
Maximum applied tensile stress Design tensile stress		Ft' = Ft * KFt * ft / Ft' = 0.41 PASS - Des V = Eq = 6.1	* φι * λ * C _{Mt} * C 3 gn tensile stre 9 kips	ess exceeds m					

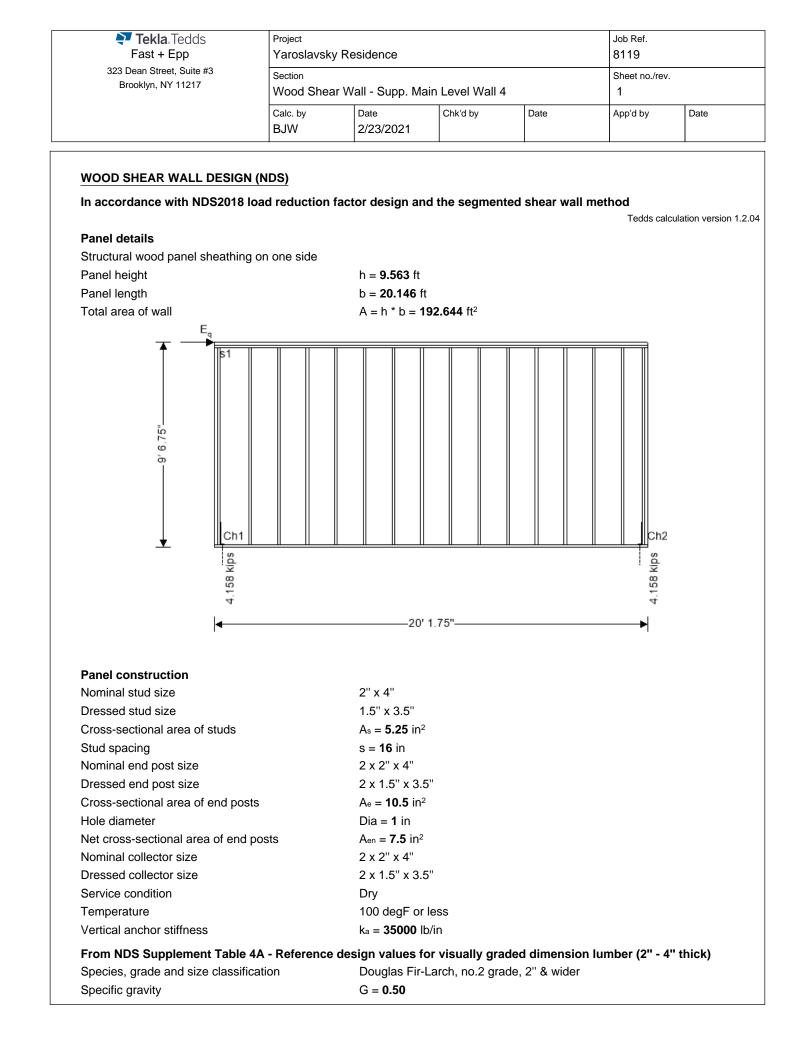
Tekla Tedds Fast + Epp	Project Yaroslavsk	y Residence			Job Ref. 8119				
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Wood She	ar Wall - Supp. Up	per Level Wall	13	Sheet no./rev. 4				
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date			
Maximum applied compressive	stress	$f_c = C / A_e =$	550 lb/in ²						
Design compressive stress		$F_{c}' = F_{c} * K_{Fc}$	* фс * λ * Смс *	Ctc * CFc * Ci *	CP = 1318 lb/in ²				
		fc / Fc' = 0.417							
	PASS -	Design compres	sive stress ex	ceeds maxim	um applied com	pressive stre			
Hold down force									
Chord 1		T ₁ = 13.506	T1 = 13.506 kips						
Chord 2		T ₂ = 13.506	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 _{8s} = V _{8s} / b = 1350.56 lb/ft							
Anchor tension force		T _δ = max(0 kips,v _{δs} * 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		le = 1							
Amp. seis. deflection – ASCE7	' Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{sws}$	swse / Ie = 2.816	in					
		δ sws / Δ s_allow \approx	= 1.173						
			FAIL - S	Shear wall def	flection exceeds	deflection lin			

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323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Wood Shea	ar Wall - Supp. Ma	ain Level Wall 3	3	Sheet no./rev.	
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date
WOOD SHEAR WALL DESIGN	N (NDS)			I	I	
In accordance with NDS2018	load reduction	factor design an	d the segmen	ted shear wall		ulation versio
Panel details						
Structural wood panel sheathing	g on both sides					
Panel height		h = 9.563 ft				
Panel length		b = 13.646 ft				
Total area of wall		A = h * b = 1				
	↓ ↓↓↓↓	□+। ↓↓↓↓↓↓↓↓	└ _┦ +S ┶╈╈╈╈╈╈╈	↓↓↓↓↓↓		
-	Eq					
9, 6.75	Ch1 sdjy 58			Ch2 sdy sg	2	
	13.38			13.38		
		13'	7.75"	►		
Panel construction						
Nominal stud size		2" x 4"				
Dressed stud size		1.5" x 3.5"				
Cross-sectional area of studs		As = 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	5"			
Cross-sectional area of end pos	sts	A _e = 15.75 ir	1 ²			
Hole diameter		Dia = 1 in				
Net cross-sectional area of end	posts	A _{en} = 11.25 i	n²			
Nominal collector size		2 x 2" x 4"	-11			
Dressed collector size		2 x 1.5" x 3.5) ''			
Service condition		Dry				
Temperature		100 degF or				
Vertical anchor stiffness		ka = 80000 lk	n/in			

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323 Dean Street, Suite #3	Section				Sheet no./rev	
Brooklyn, NY 11217	Wood Shea	r Wall - Supp. Ma	ain Level Wall 3	3	2	
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date
Specific gravity		G = 0.50				
Tension parallel to grain		Ft = 575 lb/ir	1 ²			
Compression parallel to grain		Fc = 1350 lb/				
Modulus of elasticity		E = 160000				
Minimum modulus of elasticity		Emin = 58000	U ID/IN ²			
Sheathing details		15/2011	d nonal O nh			
Sheathing material			d panel 3-ply nails at 2''ce		atning	
Fastener type						
From SDPWS Table 4.3A Nomi		-		Shear Walls -	Wood-based Pa	nels
Nominal unit shear capacity for		0				
Nominal unit shear capacity for	-					
Apparent shear wall shear sti		Ga = 20 kips	s/IN			
Combined unit shear capacitie						
Combined nominal unit shear	capacity for s	-				
		$v_{sc} = 2 * v_s$	= 2560 lb/ft			
Combined nominal unit shear	capacity for v	-				
.		$V_{wc} = 2 * V_{w}$				
Combined apparent shear wa	Il shear stiffne	$G_{ac} = G_{a1} + G_{a1}$	G _{a2} = 40 kips	/in		
Loading details						
Dead load acting on top of panel		D = 200 lb/ft				
Floor live load acting on top of pa		$L_f = 400 \text{ lb/ft}$				
Snow load acting on top of panel Self weight of panel		S = 200 lb/ft Swt = 12 lb/ft				
In plane seismic load acting at he	ead of panel	$E_q = 19100$				
Design spectral response accel.	-					
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).5Lf + 0.5(Lr o			
Load combination no.3		1.2D + E + 0	.5Lf + 0.7S			
Load combination no.4		0.9D + W				
Load combination no.5		0.9D + E				
Adjustment factors						
Format conversion factor for tens	sion – Table N1					
Format conversion factor for corr	nression - Tak	KFt = 2.70				
	1916331011 - 1 dl	KFc = 2.40				
Format conversion factor for mod	dulus of elasticit					
		KFE = 1.76				
Resistance factor for tension – T	able N2	$\varphi t = \textbf{0.80}$				
Resistance factor for compression	n – Table N2	$\varphi_{\rm c}=\boldsymbol{0.90}$				
Resistance factor for modulus of	elasticity - Tat	ole N2				
		φs = 0.85				

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Brooklyn, NY 11217	Wood Shear	Wall - Supp. Ma	ain Level Wall 3		3				
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date			
Time effect factor – Table N3		λ = 1.00							
Sheathing resistance factor		φD = 0.80							
Size factor for tension – Table 4	4	CFt = 1.50							
Size factor for compression – Ta		CFc = 1.15							
Wet service factor for tension -		C _{Mt} = 1.00							
Wet service factor for compressi		C _{Mc} = 1.00							
Wet service factor for modulus o	r elasticity – Tab								
Temperature factor for tension –	Table 2.2.2	Сме = 1.00 Сtt = 1.00							
Temperature factor for compress									
		Ctc = 1.00							
Temperature factor for modulus	of elasticity – Ta	ble 2.3.3							
		CtE = 1.00							
Incising factor – cl.4.3.8		Ci = 1.00							
Buckling stiffness factor - cl.4.4.	2	C⊤ = 1.00							
Adjusted modulus of elasticity		Emin' = Emin *	Кге * фs * Сме *	CtE * Ci * CT =	870000 psi				
Critical buckling design value		F _{cE} = 0.822 >	< E _{min} ' / (h / d)²	= 665 psi					
Reference compression design v	alue	$F_{c}^{*} = F_{c} * K_{Fc}$	e* φc*λ*CMc*	$C_{tc} * C_{Fc} * C_i =$	3353 psi				
For sawn lumber		c = 0.8							
Column stability factor - eqn.	3.7-1	C _P = (1 + (F	FcE / Fc*)) / (2 >	< c) – √([(1 + ((FcE / Fc*)) / (2 ×	c)]² - (F _{cE}			
		Fc*) / C) = 0 .	.19						
From SDPWS Table 4.3.4 Maxi	mum Shear Wa	II Aspect Ratios	6						
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 capacit									
Maximum shear force under	-		•						
Shear capacity for seismic loa	ading		e * b = 27.947	kips					
		$V_{s_max} / V_s = 0$							
		PASS - Shea	ar capacity for	seismic load	exceeds maxim	um shear f			
Chord capacity for chords 1 and	nd 2								
Shear wall aspect ratio Load combination 5		h / b = 0.701							
Shear force for maximum tension	ı	V = Eq = 19. 1	I kips						
Axial force for maximum tens	ion	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		$F_t = F_t * K_{Ft} *$	* φt * λ * Cмt * C	tt * CFt * Ci = 18	863 lb/in ²				
		ft / Ft' = 0.63							
		PASS - Desi	ign tensile stre	ess exceeds m	naximum applied	d tensile st			
Load combination 3 Shear force for maximum compr									
	aaaian	V = Eq = 19. 1	l kins						

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	Calc. by BJW	Date 2/23/2021							
Axial force for maximum co	mpression	P = (1.2 * (D) + S _{wt} * h) + (0.2 * Sps * (C	0 + S _{wt} * h) + 0.5	5 * Lf + 0.7 *			
		* s / 2 = 0.5 1	8 kips						
Maximum compressive force in	n chord	C = V * h / (b)) + P = 13.903	kips					
Maximum applied compressive	e stress	$f_c = C / A_e = 8$	83 lb/in ²						
Design compressive stress		$F_{c}{}^{'}=F_{c}{}^{*}K_{Fc}{}^{*}\phi_{c}{}^{*}^{*}C_{Mc}{}^{*}C_{tc}{}^{*}C_{Fc}{}^{*}C_{i}{}^{*}C_{P}=\textbf{636} \text{ Ib/in}^{2}$							
		fc / Fc' = 1.38)						
	FAIL - Des	sign compressive	stress is less	than maxim	um applied com	pressive str			
Hold down force									
Chord 1		T1 = 13.385 k	ips						
Chord 2		T2 = 13.385 k	ips						
Seismic deflection									
Design shear force		V _{ðs} = E _q = 19.1 kips							
Deflection limit		Δ s allow= 0.020 * h = 2.295 in							
Induced unit shear		$v_{\delta s} = V_{\delta s} / b =$	$v_{\delta s} = V_{\delta s} / b = 1399.7 \text{ lb/ft}$						
Anchor tension force		T _δ = max(0 ki	ps,v₀s * h) = 1 3	. 385 kips					
Shear wall elastic deflection -	Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s}$	* h ³ / (3 * E * A	∿e * b) + Vδs * h	n / (Gac) + h * Τδ /	(ka * b) = 0.4			
Deflection ampification factor		$C_{d\delta} = 4$							
Seismic importance factor		le = 1							
Amp. seis. deflection – ASCE	7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{sws}$	wse / Ie = 1.921	in					
Amp. sels. defiection – Abou									
		δ_{sws} / Δ_{s_allow} =	0.837						

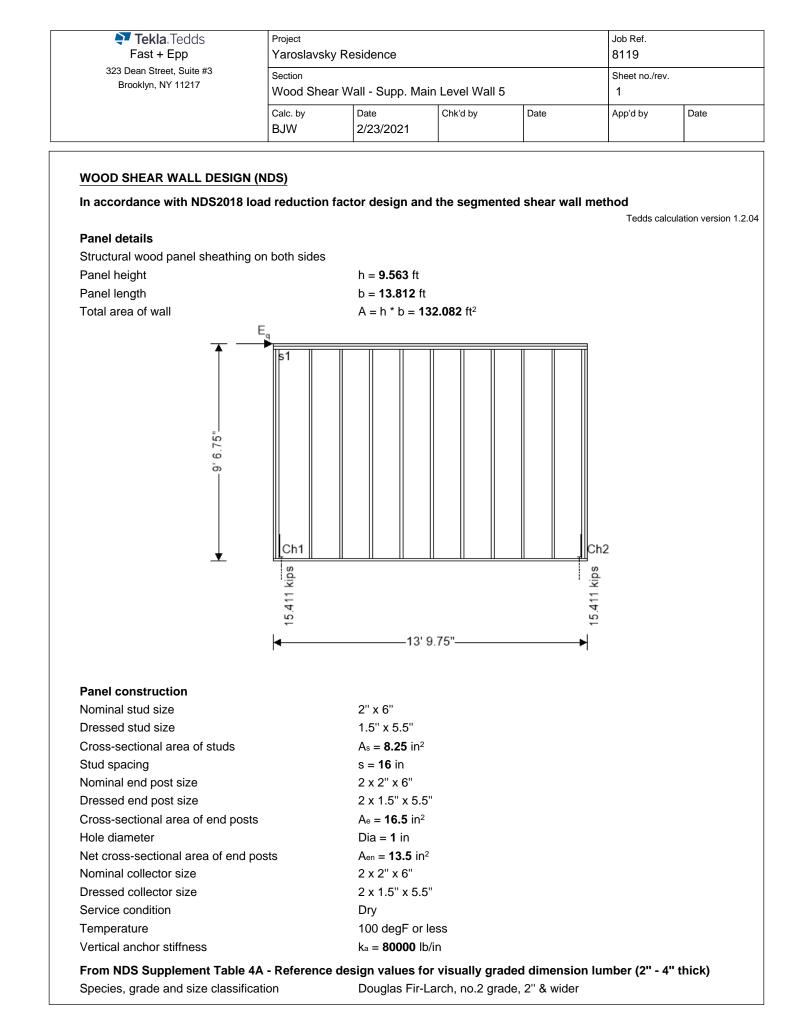


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Brooklyn, NY 11217	Wood Shea	ar Wall - Supp. Ma	ain Level Wall 4	1	2				
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	2 App'd by	Date			
Tension parallel to grain		Ft = 575 lb/it	n²						
Compression parallel to grain		Fc = 1350 lb	/in²						
Modulus of elasticity		E = 160000	0 lb/in ²						
Minimum modulus of elasticity		Emin = 58000	00 lb/in ²						
Sheathing details									
Sheathing material		15/32'' woo	od panel 3-ply	plywood she	athing				
Fastener type		8d commoi	n nails at 3"ce	enters					
From SDPWS Table 4.3A Nom	ninal Unit Shear	Capacities for	Wood-Frame S	Shear Walls - \	Nood-based Pa	nels			
Nominal unit shear capacity		-							
Nominal unit shear capacity		-							
Apparent shear wall shear st	-	Ga = 15 kip							
	-	····P							
Loading details Self weight of panel		S _{wt} = 12 lb/ft	-2						
In plane seismic load acting at head of panel		E _q = 8760 lb							
Design spectral response accel	-								
From IBC 2018 cl.1605.2	,								
Load combination no.1		1 2D + 1 6/I	_r or S or R) + 0	5W/					
Load combination no.2			0.5Lf + 0.5(Lr o						
Load combination no.3		1.2D + E + (
Load combination no.4		0.9D + W							
Load combination no.5		0.9D + E							
Adjustment factors									
Format conversion factor for ter	sion – Table N1	l							
		K _{Ft} = 2.70							
Format conversion factor for con	mpression – Tal	ole N1							
		KFc = 2.40							
Format conversion factor for mo	odulus of elastici	-							
		KFE = 1.76							
Resistance factor for tension –		$\phi_t = 0.80$							
Resistance factor for compressi		$\phi_{\rm c}=0.90$							
Resistance factor for modulus o	r elasticity – Tal								
T: (() () T) NO		φs = 0.85							
Time effect factor – Table N3			$\lambda = 1.00$						
Sheathing resistance factor		φ _D = 0.80							
Size factor for tension – Table 4		$C_{Ft} = 1.50$							
Size factor for compression – Ta		CFc = 1.15							
Wet service factor for tension –		Смt = 1.00 Смс = 1.00							
Wet service factor for compress Wet service factor for modulus of									
werservice factor for mouulus (ЮIE 4А Сме = 1.00							
Temperature factor for tension -	- Table 2.3.3	$C_{tt} = 1.00$							
Temperature factor for compres	sion - rable z.c	5.3							

Fast + Epp	Project Yaroslavsk							
323 Dean Street, Suite #3	Section	-			Sheet no./rev.			
Brooklyn, NY 11217	Wood She	ar Wall - Supp. Ma	ain Level Wall 4	Ļ	* CT = 870000 psi si si * Ci = 3353 psi [[(1 + (FcE / Fc*)) / (2 si c load exceeds maxim			
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date		
Temperature factor for modulus	of elasticity - 1	Table 2.3.3						
		CtE = 1.00						
Incising factor – cl.4.3.8		Ci = 1.00						
Buckling stiffness factor – cl.4.4	1.2	CT = 1.00						
Adjusted modulus of elasticity			-		870000 psi			
Critical buckling design value			< Emin' / (h / d) ²					
Reference compression design	value		:* φc * λ * C _{Mc} *	Ctc * CFc * Ci =	3353 psi			
For sawn lumber	074	c = 0.8						
Column stability factor – eqn	column stability factor – eqn.3.7-1		<i>,,,</i> (× C) – ∿([(1 +	(FcE / Fc [*])) / (2 >	< C)]² - (Fc		
		$F_{c^{*}}) / c) = 0.$.19					
From SDPWS Table 4.3.4 Max		-	6					
Maximum shear wall aspect rat	io	3.5						
Shear wall length		b = 20.146 ft						
Shear wall aspect ratio		h / b = 0.475						
Segmented shear wall capaci	-	.v. –	I '					
	Maximum shear force under seismic loading		= 8.76 kips					
Shear capacity for seismic lo	Shear capacity for seismic loading		* b = 15.794 k	lips				
		$V_{s_max} / V_s =$						
		PASS - Shea	ar capacity for	seismic load	exceeds maxim	ium snear		
Chord capacity for chords 1 a	and 2							
Shear wall aspect ratio		h / b = 0.475						
Shear force for maximum tension	an	V = Eq = 8.7 0	6 kins					
Axial force for maximum ten		P = 0 kips =	•					
Maximum tensile force in chord		•) - P = 4.158 ki	DS				
Maximum applied tensile stress		ft = T / Aen =						
Design tensile stress		F_{t} = F_{t} * K_{Ft} * ϕ_{t} * λ * C_{Mt} * C_{tt} * C_{Ft} * C_{i} = 1863 Ib/in ²						
		ft / Ft' = 0.298	3					
		PASS - Desi	ign tensile stre	ess exceeds r	naximum applie	d tensile s		
Load combination 3								
Shear force for maximum comp		$V = E_q = 8.70$	•					
Axial force for maximum cor	•			,	s / 2 = 0.106 ki	ps		
Maximum compressive force in			C = V * h / (b) + P = 4.264 kips					
Maximum applied compressive Design compressive stress	511622	fc = C / Ae = 406 lb/in ² Fc' = Fc * KFc * φc * λ * CMc * Ctc * CFc * Ci * CP = 636 lb/in ²						
Design compressive siless		fc / Fc' = 0.63	-		0r - 000 10/111 ⁻			
	PASS -	Design compress		ceeds maxim	um applied com	pressive s		
Hold down force		- •						
Chord 1		T₁ = 4.158 ki	ps					
Chord 2		T ₂ = 4.158 ki	-					
Seismic deflection								
		V _{ðs} = Eq = 8						

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	,	Date 2/23/2021	Chk'd by	Date	App'd by	Date

Deflection limit	Δ_{s_allow} = 0.020 * h = 2.295 in
Induced unit shear	v _{ðs} = V _{ðs} / b = 434.83 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * h) = 4.158 kips
Shear wall elastic deflection - Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_\delta / (k_a * b) = 0.343$ in
Deflection ampification factor	$C_{d\delta} = 4$
Seismic importance factor	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e = 1.37$ in
	$\delta_{sws} / \Delta_{s_allow} = 0.597$
	PASS - Shear wall deflection is less than deflection limit



Fast + Epp	Project Yaroslavsky	Residence			Job Ref. 8119	
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		Wall - Supp. Ma			2	Data
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date
Specific gravity		G = 0.50				
Tension parallel to grain		Ft = 575 lb/ir				
Compression parallel to grain		Fc = 1350 lb/				
Modulus of elasticity Minimum modulus of elasticity		E = 1600000 Emin = 58000				
-						
Sheathing details Sheathing material		15/22" woo	d panel 3-ply	nhuwood obo	othing	
Fastener type			n nails at 2"ce		aunng	
From SDPWS Table 4.3A Nomina Nominal unit shear capacity for		-		hear Walls -	Wood-based Pa	nels
Nominal unit shear capacity for		•				
Apparent shear wall shear stiffn	•	Ga = 20 kips				
Combined unit shear capacities						
Combined unit shear capacities	anacity for se	eismic design				
	apacity for or	$V_{sc} = 2 * V_{s}$	= 2560 lb/ft			
Combined nominal unit shear ca	apacity for w					
		$V_{wc} = 2 * V_w$	= 3580 lb/ft			
Combined apparent shear wall	shear stiffne			/in		
Loading details						
Self weight of panel		Swt = 12 lb/ft	2			
In plane seismic load acting at head	d of panel	Eq = 22260 I	bs			
Design spectral response accel. pa	r., short perio	ds 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			0.5Lf + 0.5(Lr 0	r S or R)		
Load combination no.3		1.2D + E + 0	0.5Lf + 0.7S			
Load combination no.4 Load combination no.5		0.9D + W 0.9D + E				
		0.90 + 2				
Adjustment factors	n Tabla N1					
Format conversion factor for tensio		KFt = 2.70				
Format conversion factor for compr	ession – Tabl					
		KFc = 2.40				
Format conversion factor for modul	us of elasticity	/ – Table N1				
		Kfe = 1.76				
Resistance factor for tension - Tab	le N2	$\phi t = \boldsymbol{0.80}$				
Resistance factor for compression		$\varphi_{\rm c}=\boldsymbol{0.90}$				
Resistance factor for modulus of ela	asticity – Tab					
		φs = 0.85				
Time effect factor – Table N3 Sheathing resistance factor		$\lambda = 1.00$ $\phi D = 0.80$				

Fast + Epp	y Residence			Job Ref. 8119						
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		ar Wall - Supp. M	ain Level Wall 5		3					
	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date				
Size factor for compression – Tabl	le 4A	CFc = 1.10								
Wet service factor for tension - Ta	able 4A	C _{Mt} = 1.00								
Wet service factor for compression	n – Table 4A	C _{Mc} = 1.00								
Wet service factor for modulus of e	elasticity – Ta	able 4A								
		Сме = 1.00								
Temperature factor for tension – T Temperature factor for compression		Ctt = 1.00 3.3								
		Ctc = 1.00								
Temperature factor for modulus of	elasticity - T	able 2.3.3								
		CtE = 1.00								
Incising factor – cl.4.3.8		Ci = 1.00								
Buckling stiffness factor - cl.4.4.2		C⊤ = 1.00								
Adjusted modulus of elasticity		Emin' = Emin	* Кге * фѕ * Сме *	$C_{tE} * C_i * C_T =$	870000 psi					
Critical buckling design value		F _{cE} = 0.822	\times Emin' / (h / d)²	= 1643 psi						
Reference compression design val	lue	$F_{c}^{*} = F_{c} * K_{f}$	=с*фс*λ*Смс*	Ctc * CFc * Ci =	3208 psi					
For sawn lumber		c = 0.8								
Column stability factor - eqn.3.	7-1	C _P = (1 + (FcE / Fc*)) / (2 >	< c) – √([(1 +	(FcE / Fc*)) / (2 :	× с)] ² - (F _{сЕ}				
		$F_{c^{*}}) / c) = 0$).44							
From SDPWS Table 4.3.4 Maxim	um Chaor W	all Aspect Datis	-							
Maximum shear wall aspect ratio		3.5	15							
Shear wall length		b = 13.812	ft							
Shear wall aspect ratio			h / b = 0.692							
			-							
Segmented shear wall capacity Maximum shear force under se	eismic loadir	ng Vs_max = Ec	= 22.26 kips							
Shear capacity for seismic load	lina	$V_{s} = \phi_{D} * v_{sc} * b = 28.288 \text{ kips}$								
		V _{s_max} / V _s = 0.787								
				seismic load	exceeds maxim	um shear f				
Chord capacity for chords 1 and	12									
Shear wall aspect ratio	· -	h / b = 0.69	2							
Shear force for maximum tension		V = Eq = 22	.26 kips							
Axial force for maximum tensio	n		P = 0 kips = 0 kips							
Maximum tensile force in chord			•	kips						
Maximum applied tensile stress			T = V * h / (b) - P = 15.411 kips ft = T / Aen = 1142 lb/in ²							
Design tensile stress			* φt * λ * CMt * C	tt * CFt * Ci = 16	615 lb/in ²					
U		ft / Ft' = 0.7 0	-							
				ess exceeds n	naximum applie	d tensile st				
Load combination 3										
Shear force for maximum compres		$V = E_q = 22$	-							
Axial force for maximum compr	ression	P = (1.2 * 3	Swt * h + 0.2 * \$	SDS * Swt * h) *	* s / 2 = 0.106 k	ips				
Maximum compressive force in chord		C = V * h / (b) + P = 15.517 kips								
•	Maximum applied compressive stress		fc = C / Ae = 940 lb/in ²							
•	ess				CP = 1418 lb/in ²					

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	Calc. by BJW	Date 2/23/2021	Chk'd by	Date	App'd by	Date	
		fc / Fc' = 0.66	3				
	PASS - [Design compres	sive stress ex	ceeds maxim	um applied com	pressive stre	
Hold down force							
Chord 1	T ₁ = 15.411 kips						
Chord 2		T ₂ = 15.411 kips					
Seismic deflection							
Design shear force		V _{∂s} = E _q = 22.26 kips					
Deflection limit		$\Delta_{s_{allow}} = 0.020 * h = 2.295$ in					
Induced unit shear		vðs = Vðs / b = 1611.58 lb/ft					
Anchor tension force		T _δ = max(0 kips,v _{δs} * h) = 15.411 kips					
Shear wall elastic deflection -	Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s}$	₅ * h³ / (3 * E * /	Ae * b) + vձs * h	n / (Gac) + h * Τδ / ((ka * b) = 0.5 5	
Deflection ampification factor		$C_{d\delta} = 4$					
Seismic importance factor		le = 1					
Amp. seis. deflection – ASCE	7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e = 2.198$ in					
		δ_{sws} / Δ_{s_allow}	= 0.958				
			PASS - She	ar wall deflec	tion is less than	deflection li	

Tekla Tedds Fast + Epp	Project Yaroslavsky	y Residence			Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section		Sheet no./rev	<i>'</i> .		
		ar Wall - Supp. Ma			1	
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date
WOOD SHEAR WALL DESIG						
In accordance with NDS2018		factor design an	d the segmen	ted shear wal	l method	
		-	-			culation version
Panel details Structural wood panel sheathin	na on one side					
Panel height	ig on one side	h = 9.563 ft				
Panel length		b = 7.813 ft				
Total area of wall		A = h * b = 7	4.707 ft ²			
	E					
		IST				
	6.75"-					
	ັດ 					
	—	Ch1		Ch2		
		sd		sd		
		86 kips		886 kips		
		œ		ω _.		
		0		9		
		◄ 7' 9	9.75"———			
Panel construction						
Nominal stud size		2" x 6"				
Dressed stud size		1.5" x 5.5"				
Cross-sectional area of studs		As = 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 po	sts	A _e = 16.5 in ²				
Hole diameter		Dia = 1 in				
Net cross-sectional area of end	d posts	A _{en} = 13.5 in	2			
Nominal collector size		2 x 2" x 6"	- 11			
Dressed collector size		2 x 1.5" x 5.5	D''			
Service condition		Dry	1			
Temperature Vertical anchor stiffness		100 degF or ka = 60000 lk				

Tekla Tedds Fast + Epp	Project Yaroslavsky	Yaroslavsky Residence			Job Ref. 8119			
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Brooklyn, NY 11217	Wall - Supp. Ma	in Level Wall 7	7	2				
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date		
From NDS Supplement Table 4 Species, grade and size classific Specific gravity Tension parallel to grain Compression parallel to grain Modulus of elasticity Minimum modulus of elasticity Sheathing details Sheathing material Fastener type From SDPWS Table 4.3A Nom Nominal unit shear capacity f Nominal unit shear capacity f Apparent shear wall shear sti	i nal Unit Shear or seismic desi or wind design	Douglas Fir-l G = 0.50 Ft = 575 lb/in Fc = 1350 lb/ E = 1600000 Emin = 58000 15/32'' woo 8d common Capacities for V ign Vs = 1280 lb	Larch, no.2 gra ² in ² 1b/in ² 0 lb/in ² d panel 3-ply n nails at 2''ce Vood-Frame S /ft	ide, 2'' & wider plywood she enters	athing			
Loading details Self weight of panel In plane seismic load acting at head of panel Design spectral response accel. par., short periods		E _q = 5626 lbs	S _{wt} = 12 lb/ft ² E _q = 5626 lbs s S _{DS} = 0.944					
From IBC 2018 cl.1605.2								
Load combination no.1		1.2D + 1.6(L	1.2D + 1.6(Lr or S or R) + 0.5W					
Load combination no.2		1.2D + W + (1.2D + W + 0.5Lf + 0.5(Lr or S or R)					
Load combination no.3		1.2D + E + 0	1.2D + E + 0.5Lf + 0.7S					
Load combination no.4		0.9D + W	0.9D + W					
Load combination no.5		0.9D + E	0.9D + E					
Adjustment factors								
Format conversion factor for ten	sion – Table N1							
		KFt = 2.70						
Format conversion factor for con	npression – Tab	le N1						
		KFc = 2.40						
Format conversion factor for mo	dulus of elasticity							
		_	Kfe = 1.76					
Resistance factor for tension – 1			$\phi_t = 0.80$					
Resistance factor for compression		$\phi_c = 0.90$						
Resistance factor for modulus of	elasticity – Tab							
		$\phi_{s} = 0.85$						
Time effect factor – Table N3		$\lambda = 1.00$						
Sheathing resistance factor	tance factor		$\phi D = 0.80$					
Size factor for tension – Table 4	4	CFt = 1.30						
Size factor for compression – Ta	ble 4A	CFc = 1.10						
Wet service factor for tension -	Table 4A	C _{Mt} = 1.00						
Wet service factor for compressi	on – Table 4A	Смс = 1.00						
Wet service factor for modulus o	f alasticity _ Tak	ole 4A						

Fast + Epp	y Residence	lesidence							
323 Dean Street, Suite #3 Section			Sheet						
Brooklyn, NY 11217		ar Wall - Supp. Ma	Vall - Supp. Main Level Wall 7 3						
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date			
Temperature factor for tension -	- Table 2.3.3	C _{tt} = 1.00			I	I			
Temperature factor for compres	sion – Table 2.3	3.3							
		Ctc = 1.00							
Temperature factor for modulus	of elasticity - T	able 2.3.3							
		CtE = 1.00							
Incising factor – cl.4.3.8	0	Ci = 1.00							
Buckling stiffness factor – cl.4.4	.2	C⊤ = 1.00			070000 ·				
Adjusted modulus of elasticity			Кге * фs * Сме *		870000 psi				
Critical buckling design value			× E _{min} ' / (h / d)²	-					
Reference compression design	value		с*фс*λ*Смс*	$C_{tc} * C_{Fc} * C_i =$	3208 psi				
For sawn lumber		c = 0.8		,					
Column stability factor – eqn	.3.7-1	C _P = (1 + (I	= _{cE} / F _c *)) / (2	× c) – √([(1 +	(FcE / Fc*)) / (2 >	< c)]² - (F cE			
		F _c *) / c) = 0.44							
From SDPWS Table 4.3.4 Max	imum Shear Wa	all Aspect Ratio	S						
Maximum shear wall aspect rati		3.5							
Shear wall length		b = 7.813 ft	b = 7.813 ft						
Shear wall aspect ratio		h / b = 1.22 4	L I						
Segmented shear wall capacit	v								
Maximum shear force under	-	g Vs_max = Eq	= 5.626 kips						
Shear capacity for seismic lo		-	* b = 8 kips						
	g	Vs_max / Vs = 0.703							
				seismic load	exceeds maxim	um shear			
Chord capacity for chords 1 a	nd 2								
Shear wall aspect ratio		h / b = 1.22 4	h / b = 1.224						
Load combination 5									
Shear force for maximum tensic	'n	$V = E_q = 5.6$	V = E _q = 5.626 kips						
Axial force for maximum tens	sion	P = 0 kips :	P = 0 kips = 0 kips						
Maximum tensile force in chord		T = V * h / (t	T = V * h / (b) - P = 6.886 kips						
Maximum applied tensile stress		ft = T / Aen =	ft = T / Aen = 510 lb/in ²						
Design tensile stress		$F_t' = F_t * K_{Ft}$	$Ft' = Ft * K_{Ft} * \varphi t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i = 1615 \text{ lb/in}^2$						
		ft / Ft' = 0.316							
		PASS - Des	ign tensile str	ess exceeds n	naximum applie	d tensile s			
Load combination 3									
Shear force for maximum compression			$V = E_q = 5.626$ kips						
Axial force for maximum compression		$P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2 = 0.106 \text{ kips}$							
Maximum compressive force in chord			C = V * h / (b) + P = 6.992 kips						
Maximum applied compressive stress Design compressive stress		$f_c = C / A_e = 424 \text{ lb/in}^2$							
			-	Ctc * CFc * Ci *	C _P = 1418 lb/in ²				
		fc / Fc' = 0.2 Design compres		coode movimi	im applied com	nrassiva			
Held dawn (m. 1	г <i>н</i> ээ - I	Design compres	51VE 311E22 EX		ani applieu com	picssive S			
Hold down force		T 0.000 '	ine						
			-						
Chord 1 Chord 2		T1 = 6.886 k T2 = 6.886 k	-						

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Seismic deflection

Design shear force	V _{ðs} = E _q = 5.626 kips
Deflection limit	Δ_{s_allow} = 0.020 * h = 2.295 in
Induced unit shear	v _{õs} = V _{õs} / b = 720.13 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * 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	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e = 2.037$ in
	$\delta_{sws} / \Delta_{s_allow} = 0.888$

Fast + Epp	ProjectJob Ref.Yaroslavsky Residence8119						
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Shear Wall - Supp. Main Level Wall 8 1					neet no./rev.	
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date	
WOOD SHEAR WALL DESIG	N (NDS)						
In accordance with NDS2018	load reduction	factor design an	d the segmen	ed shear wall		culation versi	
Panel details							
Structural wood panel sheathin	g on one side						
Panel height		h = 9.563 ft					
Panel length		b = 9.125 ft					
Total area of wall	_	A = h * b = 8	7.258 ft ²				
	E						
	f f	s1					
	2						
	6.75"						
	-б 						
	L I	Ch1		Ch2			
	<u> </u>	<u>v</u>	<u> </u>	v			
		887 kips		887 kips			
		88		88.			
		Q		9			
	•	9'	1.5"	►			
Panel construction							
Nominal stud size		2" x 4"					
Dressed stud size		1.5" x 3.5"					
Cross-sectional area of studs		As = 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 pos	sts	A _e = 15.75 in	2				
Hole diameter		Dia = 1 in					
Net cross-sectional area of end	l posts	A _{en} = 11.25 i	n²				
Nominal collector size		2 x 2" x 4"					
Dressed collector size		2 x 1.5" x 3.5	5"				
Service condition		Dry					
Temperature		100 degF or	less				
Vertical anchor stiffness		ka = 60000 lk	r.				

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Brooklyn, NY 11217	Vall - Supp. Ma	/all - Supp. Main Level Wall 8			2			
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date		
Specific gravity		G = 0.50						
Tension parallel to grain		Ft = 575 lb/ir	2					
Compression parallel to grain		Fc = 1350 lb/	'in²					
Modulus of elasticity		E = 1600000	lb/in ²					
Minimum modulus of elasticity		Emin = 58000	0 lb/in ²					
Sheathing details								
Sheathing material		15/32" woo	d panel 3-ply	plywood she	athing			
Fastener type		8d commor	nails at 2"ce	enters				
From SDPWS Table 4.3A Non	ninal Unit Shear Ca	apacities for V	Vood-Frame S	Shear Walls - \	Nood-based Pa	nels		
Nominal unit shear capacity		-						
Nominal unit shear capacity	-	v _w = 1790 lb						
Apparent shear wall shear s	•	Ga = 20 kips	s/in					
Loading details		Swt = 12 lb/ft ²	2					
Self weight of panel In plane seismic load acting at	head of nanel	$E_q = 6572 \text{ lb}$						
Design spectral response acce	-	•						
	i. par., short periods	000 - 0.044						
From IBC 2018 cl.1605.2 Load combination no.1			or S or B) + 0	5\\/				
Load combination no.2		1.2D + 1.6(Lr or S or R) + 0.5W 1.2D + W + 0.5Lr + 0.5(Lr 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 tel	nsion – Table N1							
		KFt = 2.70						
Format conversion factor for co	mpression – Table	N1						
		KFc = 2.40						
Format conversion factor for me	odulus of elasticity -	- Table N1						
		K _{FE} = 1.76						
Resistance factor for tension -	Table N2	$\phi_t = 0.80$						
Resistance factor for compress	ion – Table N2	$\phi_{\rm c} = 0.90$						
Resistance factor for modulus of	of elasticity - Table	N2						
		$\phi_s = 0.85$						
Time effect factor – Table N3		$\lambda = 1.00$						
Sheathing resistance factor		$\phi D = 0.80$						
Size factor for tension – Table 4A		CFt = 1.50						
•		C _{Fc} = 1.15						
Wet service factor for tension – Table 4A		C _{Mt} = 1.00						
Wet service factor for compress		CMc = 1.00						
Wet service factor for modulus	or elasticity – Table	е 4А Сме = 1.00						
Temperature factor for tension		$C_{tt} = 1.00$						

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Brooklyn, NY 11217	Section Wood Shea	ar Wall - Supp. Ma	Wall - Supp. Main Level Wall 8							
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date				
		C _{tc} = 1.00								
Temperature factor for modulus	of elasticity - 7	Table 2.3.3								
		CtE = 1.00								
Incising factor – cl.4.3.8		Ci = 1.00								
Buckling stiffness factor - cl.4.4.	2	C⊤ = 1.00								
Adjusted modulus of elasticity		Emin' = Emin *	$Kfe * \varphi_{s} * Cme *$	* CtE * Ci * CT =	870000 psi					
Critical buckling design value		F _{cE} = 0.822	× Emin' / (h / d)²	= 665 psi						
Reference compression design v	value	$F_{c}^{*} = F_{c} * K_{F}$	с*фс*λ*Смс*	[*] C _{tc} * C _{Fc} * C _i =	= 3353 psi					
For sawn lumber		c = 0.8			-					
Column stability factor - eqn.	3.7-1	C _P = (1 + (Ⅰ	F _{cE} / F _c *)) / (2	× c) – √([(1 +	(F _{cE} / F _c *)) / (2 >	≺ c)]² - (F _{cE}				
		F _c *) / c) = 0	<i>//</i> (
From SDPWS Table 4.3.4 Maxi		-	S							
Maximum shear wall aspect ratio)	3.5								
Shear wall length			b = 9.125 ft							
Shear wall aspect ratio		$\Pi / D = 1.040$	h / b = 1.048							
Segmented shear wall capacity	-									
Maximum shear force under s	seismic loadir	-	= 6.572 kips							
Shear capacity for seismic loa	ading	$V_s = \phi_D * v_s$	* b = 9.344 ki	ps						
			0.703							
		PASS - Shea	ar capacity for	seismic load	exceeds maxim	ium shear f				
Chord capacity for chords 1 ar	nd 2									
Shear wall aspect ratio		h / b = 1.048	3							
Load combination 5										
Shear force for maximum tension	า		$V = E_q = 6.572$ kips							
Axial force for maximum tens	ion	•	P = 0 kips = 0 kips							
Maximum tensile force in chord		T = V * h / (b	T = V * h / (b) - P = 6.887 kips							
Maximum applied tensile stress			$f_t = T / A_{en} = 612 \text{ lb/in}^2$							
Design tensile stress			$F_{t}' = F_{t} * K_{Ft} * \phi_{t} * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_{i} = 1863 \text{ lb/in}^{2}$							
			ft / Ft' = 0.329 PASS - Design tensile stress exceeds maximum applied tensile str							
		PASS - Des	ign tensile str	ess exceeds r	naximum applie	d tensile st				
Load combination 3			70 1.:							
Shear force for maximum compression			$V = E_q = 6.572$ kips							
Axial force for maximum com	-		$P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2 = 0.106 \text{ kips}$							
Maximum compressive force in o		C = V * h / (b) + P = 6.993 kips								
Maximum applied compressive s	$f_c = C / A_e = 444 \text{ lb/in}^2$									
Design compressive stress	Fc' = Fc * KFc * φc * λ * CMc * Ctc * CFc * Ci * CP = 636 lb/in ² fc / Fc' = 0.699									
	PASS -	Design compres	-	ceeds maxim	um applied com	pressive st				
Hold down force										
Chord 1		T, _ 6 997 L	ine							
Chord 2		T ₁ = 6.887 kips T ₂ = 6.887 kips								
		12 - 0.007 K	,ho							
Seismic deflection		–								
Design shear force		$V_{\delta s} = E_q = 6$	6 .572 kips							

Tekla Tedds Fast + Epp						Epp Yaroslavsky Residence 8119			
323 Dean Street, Suite #3 Brooklyn, NY 11217	Maria I Ohaan Mall. Ourse Maria I such Mall O				Sheet no./rev. 4				
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date			

Deflection limit	$\Delta_{s_{allow}} = 0.020 * h = 2.295 in$
Induced unit shear	v _{ðs} = V _{ðs} / b = 720.22 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * h) = 6.887 kips
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 * v_{\delta s} * h^3 / (3 * E * A_e * b) + v_{\delta s} * h / (G_a) + h * T_\delta / (k_a * b) = 0.487$ in
Deflection ampification factor	$C_{d\delta} = 4$
Seismic importance factor	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e = 1.946$ in
	$\delta_{sws} / \Delta_{s_allow} = 0.848$
	DACC. Cheen well deflection is loss than deflection limit

Tekla Tedds Fast + Epp		Project Yaroslavsky Residence				Job Ref. 8119	
323 Dean Street, Suite #3	Section				Sheet no./rev		
Brooklyn, NY 11217	Wood Shea	Wood Shear Wall - Supp. Main Level Wall 9					
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date	
WOOD SHEAR WALL DESIG	N (NDS)						
In accordance with NDS2018	load reduction	factor design an	d the segmen	ted shear wall			
Devel details					Tedds cal	culation versio	
Panel details Structural wood panel sheathin	a on one side						
Panel height	g on one side	h = 9.563 ft					
Panel length		b = 9.25 ft					
Total area of wall		A = h * b = 8	8 453 ft ²				
	Eg						
	· ★ ──▶						
		s1					
	5"						
	6.75						
	6						
		Ch1		Ch2			
	▼ Ш						
		887 kips					
		887					
		9.9		9.9			
		9	' 3"				
	1	-		- 1			
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	s = 10 m 3 x 2" x 4"						
Dressed end post size	3 x 1.5" x 3.5"						
Cross-sectional area of end pos							
Hole diameter		Dia = 1 in					
Net cross-sectional area of end	posts	Aen = 11.25 i	n²				
Nominal collector size		2 x 2" x 4"					
Dressed collector size		2 x 1.5" x 3.5	5"				
		Dry	-				
Service condition	100 degF or less						
Service condition Temperature		-	less				

Tekla Tedds Project Fast + Epp Yaroslavs		Residence			Job Ref. 8119			
323 Dean Street, Suite #3	Section							
Brooklyn, NY 11217	Wood Shear Wall - Supp. Main Level Wall 9 2							
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date		
From NDS Supplement Table 4 Species, grade and size classificat Specific gravity Tension parallel to grain Compression parallel to grain Modulus of elasticity Minimum modulus of elasticity Sheathing details Sheathing material Fastener type From SDPWS Table 4.3A Nomi Nominal unit shear capacity for Nominal unit shear capacity for Apparent shear wall shear stif Loading details	nal Unit Shear or seismic des or wind design	Douglas Fir-l G = 0.50 Ft = 575 lb/in Fc = 1350 lb/ E = 1600000 Emin = 58000 15/32'' wood 8d common Capacities for V ign vs = 1280 lb/	_arch, no.2 gra ² in ² Ib/in ² 0 Ib/in ² d panel 3-ply nails at 2''ce Vood-Frame S /ft	de, 2'' & wider plywood she nters	athing			
Loading details Self weight of panel In plane seismic load acting at head of panel Design spectral response accel. par., short periods		E _q = 6662 lbs	$S_{wt} = 12 \text{ lb/ft}^2$ $E_q = 6662 \text{ lbs}$ $S_{DS} = 0.944$					
From IBC 2018 cl.1605.2								
Load combination no.1		1.2D + 1.6(L	or S or R) + 0	.5W				
Load combination no.2		1.2D + W + 0).5Lf + 0.5(Lr o	r 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	0.9D + E					
Adjustment factors								
Format conversion factor for tens	ion – Table N1							
_		KFt = 2.70						
Format conversion factor for com	pression – Tab							
Format constantions for the for	hulun of start 1	$K_{Fc} = 2.40$						
Format conversion factor for mod	iulus of elasticity							
Popietopoo fostar for taraira	obla NO	KFE = 1.76						
Resistance factor for tension – T			$\phi_t = 0.80$					
Resistance factor for compression – Table N2		·	$\phi_{\rm c} = 0.90$					
Resistance factor for modulus of	elasticity – Tab							
		$\phi_s = 0.85$						
Time effect factor – Table N3			$\lambda = 1.00$					
Sheathing resistance factor		φ D = 0.80						
		CFt = 1.50						
Size factor for compression – Table 4A		CFc = 1.15	C _{Fc} = 1.15					
Wet service factor for tension – 1		C _{Mt} = 1.00						
Wet service factor for compression		Смс = 1.00						
Wet service factor for modulus of	elasticity - Tal	ole 4A						

Tekla.Tedds Project Fast + Epp Yaroslavsky 323 Dean Street, Suite #3 Section		y Residence	v Residence			Job Ref. 8119			
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Brooklyn, NY 11217	Wood Shear Wall - Supp. Main Level Wall 9 3								
	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date			
Temperature factor for tension –	Table 2.3.3	Ctt = 1.00		-		• •			
Temperature factor for compress	ion – Table 2.	3.3							
		$C_{tc} = 1.00$							
Temperature factor for modulus of	of elasticity - 7								
		Ct∈ = 1.00							
Incising factor – cl.4.3.8	`	Ci = 1.00							
Buckling stiffness factor – cl.4.4.2	2	CT = 1.00			070000				
Adjusted modulus of elasticity			* Кге * фs * Сме *		870000 psi				
Critical buckling design value			\times Emin' / (h / d) ²	-					
Reference compression design v	alue		с*фс*λ*Смс*	$C_{tc} * C_{Fc} * C_i =$: 3353 psi				
For sawn lumber		c = 0.8		,					
Column stability factor – eqn.3	3.7-1	C _P = (1 + (FcE / Fc*)) / (2	× c) – √([(1 +	(FcE / Fc*)) / (2 :	× c)]² - (Fce			
		Fc*) / C) = (Fc*) / C) = 0.19						
From SDPWS Table 4.3.4 Maxin	num Shear W	all Aspect Ratio	S						
Maximum shear wall aspect ratio		3.5	-						
Shear wall length		b = 9.25 ft	b = 9.25 ft						
Shear wall aspect ratio	h / b = 1.03 4	h / b = 1.034							
Segmented shear wall capacity	,								
Maximum shear force under s		ng Vs_max = Eq	= 6.662 kips						
Shear capacity for seismic loading		$V_s = \phi D * V_s$	s * b = 9.472 ki	ps					
		•	Vs_max / Vs = 0.703						
				seismic load	exceeds maxim	um shear i			
Chord capacity for chords 1 an	d 2								
Shear wall aspect ratio Load combination 5		h / b = 1.03	4						
Shear force for maximum tension		V = Eq = 6.6	62 kips						
Axial force for maximum tension		P = 0 kips	= 0 kips						
Maximum tensile force in chord		T = V * h / (T = V * h / (b) - P = 6.887 kips						
Maximum applied tensile stress		ft = T / A _{en} =	ft = T / A _{en} = 612 lb/in ²						
Design tensile stress		$F_t = F_t * K_{Ft}$	$F_t' = F_t * K_{Ft} * \varphi_t * \lambda * C_{Mt} * C_{tt} * C_{Ft} * C_i = \textbf{1863} \text{ lb/in}^2$						
		ft / Ft' = 0.32	ft / Ft' = 0.329						
		PASS - Des	sign tensile str	ess exceeds n	naximum applie	d tensile s			
Load combination 3									
Shear force for maximum compression			$V = E_q = 6.662$ kips						
Axial force for maximum compression			$P = (1.2 * S_{wt} * h + 0.2 * S_{DS} * S_{wt} * h) * s / 2 = 0.106 \text{ kips}$						
Maximum compressive force in chord			C = V * h / (b) + P = 6.993 kips						
Maximum applied compressive s	tress	$f_c = C / A_e =$							
Design compressive stress			с*фс*λ*Смс*	Ctc * CFc * Ci *	CP = 636 lb/in ²				
		$f_c / F_c' = 0.6$		coode movim	im applied ear	proceive			
	PA22 -	Design compres	Sive Stress ex	ceeus maximi	ani appileo com	pressive s			
Hold down force									
Chord 1		T1 = 6.887 k	din a						

Tekla Tedds	Project	esidence	Job Ref.			
Fast + Epp	Yaroslavsky Re		8119			
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	Calc. by BJW	Date 2/22/2021	Chk'd by	Date	App'd by	Date

Seismic deflection

Design shear force	V _{ðs} = E _q = 6.662 kips
Deflection limit	$\Delta_{s_allow} = 0.020 * h = 2.295$ in
Induced unit shear	v _{õs} = V _{õs} / b = 720.22 lb/ft
Anchor tension force	T _δ = max(0 kips,v _{δs} * 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	le = 1
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} * \delta_{swse} / I_e = 1.939$ in
	$\delta_{sws} / \Delta_{s_allow} = 0.845$

3.2 | STEEL MOMENT FRAME DESIGN

Fast + Epp

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

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
Live load	LL =	276.667	plf	0.28	klf
Snow load	SL =	207.500	plf	0.21	klf

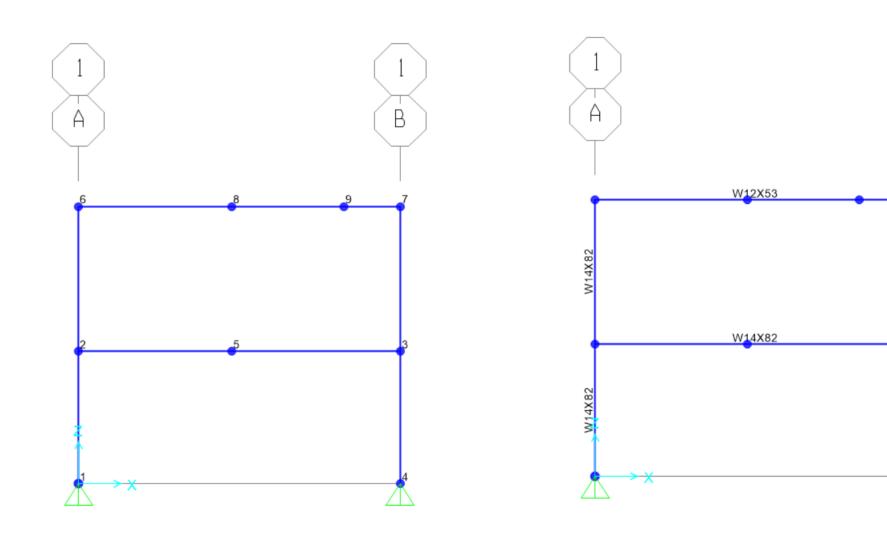
(Includes 200 plf for door)

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

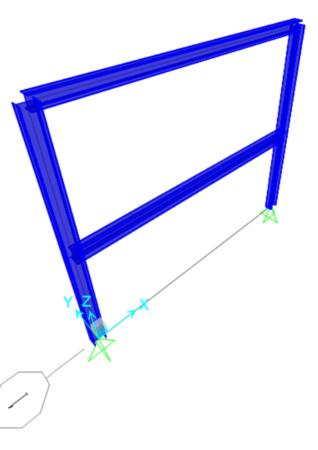


JOINT LABELS

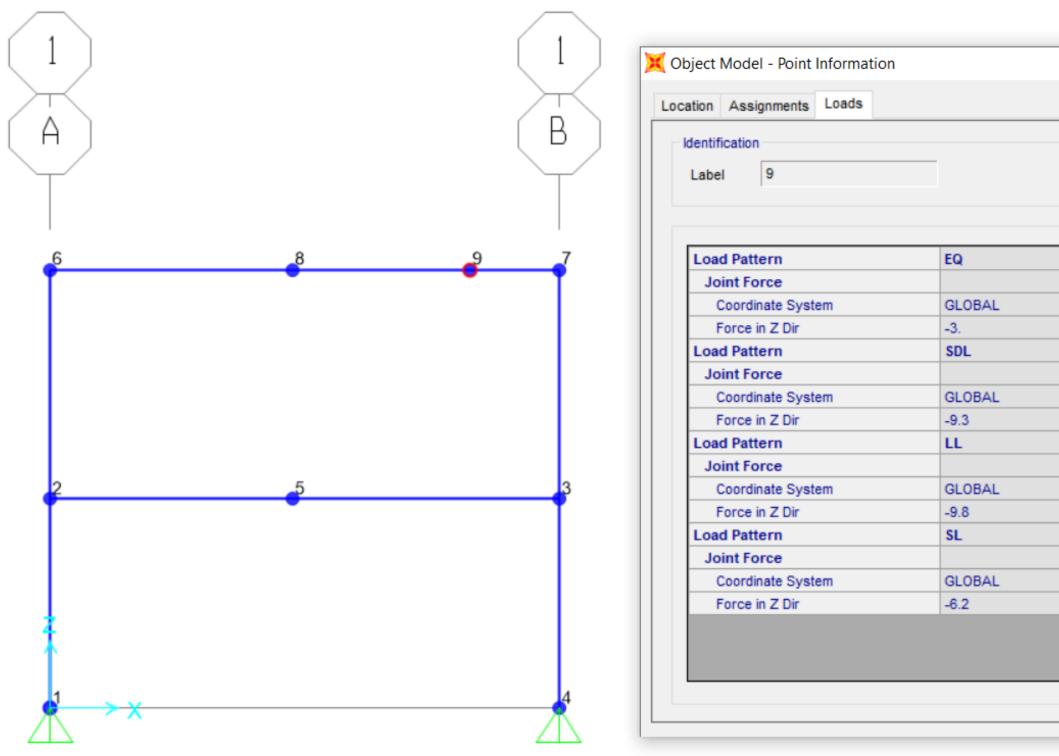
FRAME SECTION ASSIGNMENTS

В

N14X8

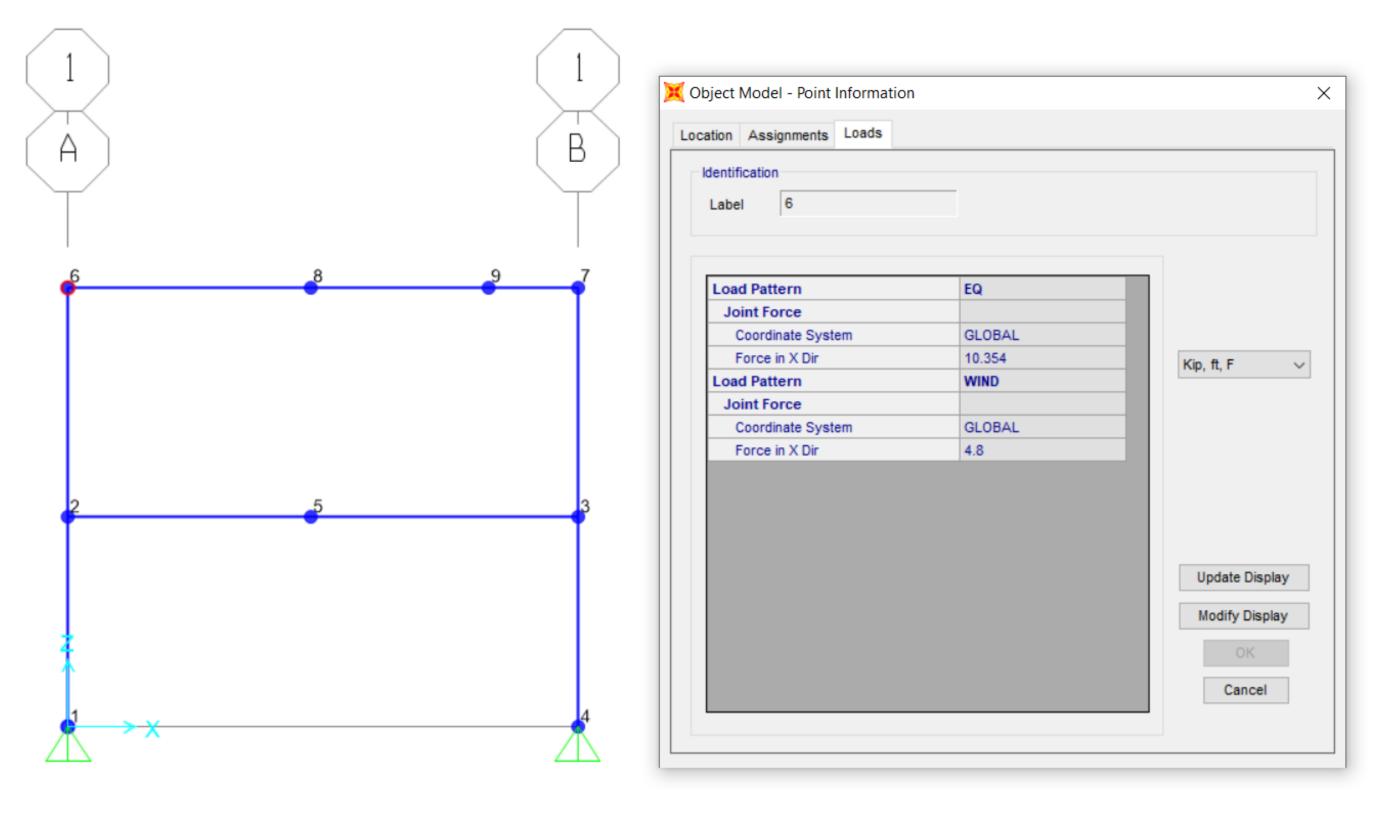


<u>3D MODEL VIEW</u>

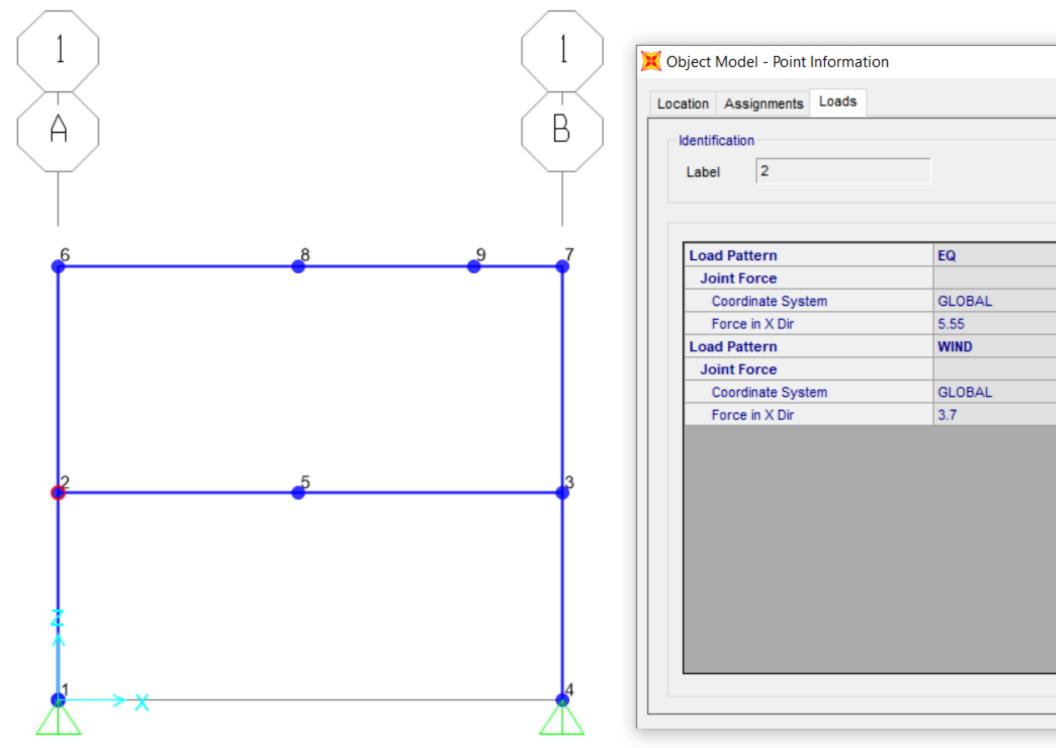


JOINT 9 LOADING - POINT LOAD FROM STEEL TRANSFER BEAM

	×
	Kip, ft, F 🗸 🗸
_	
	Update Display
	Modify Display
	ОК
	Cancel

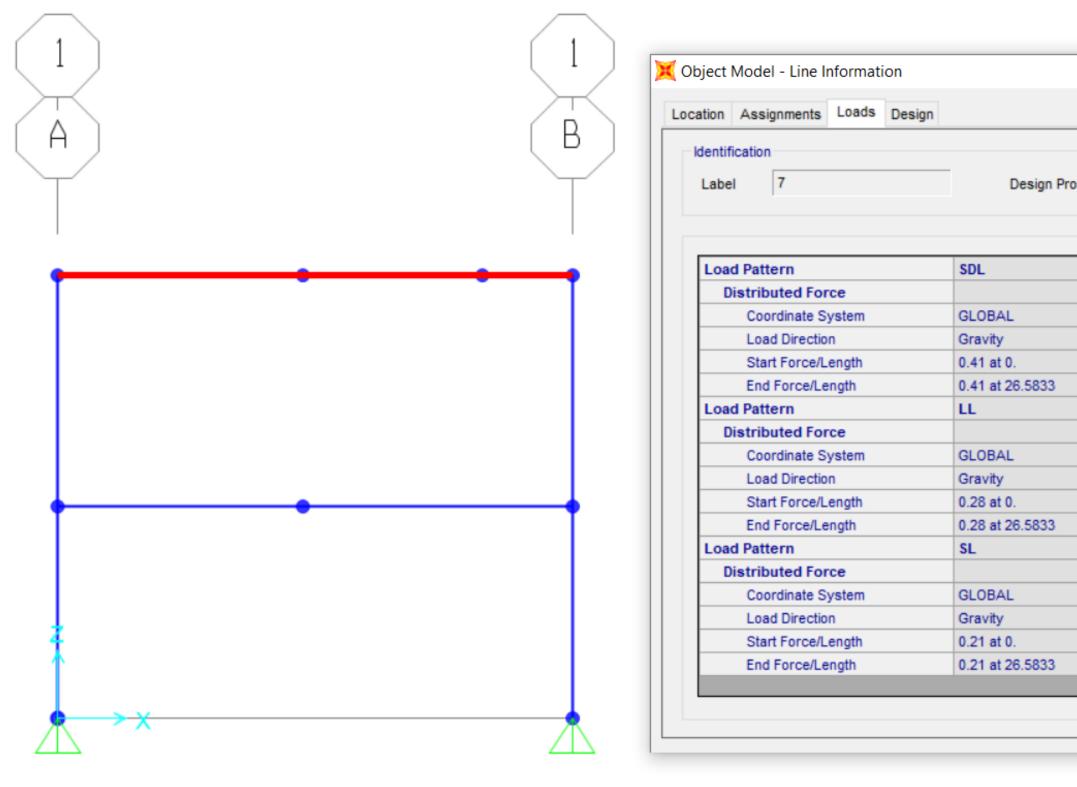


JOINT 6 LOADING



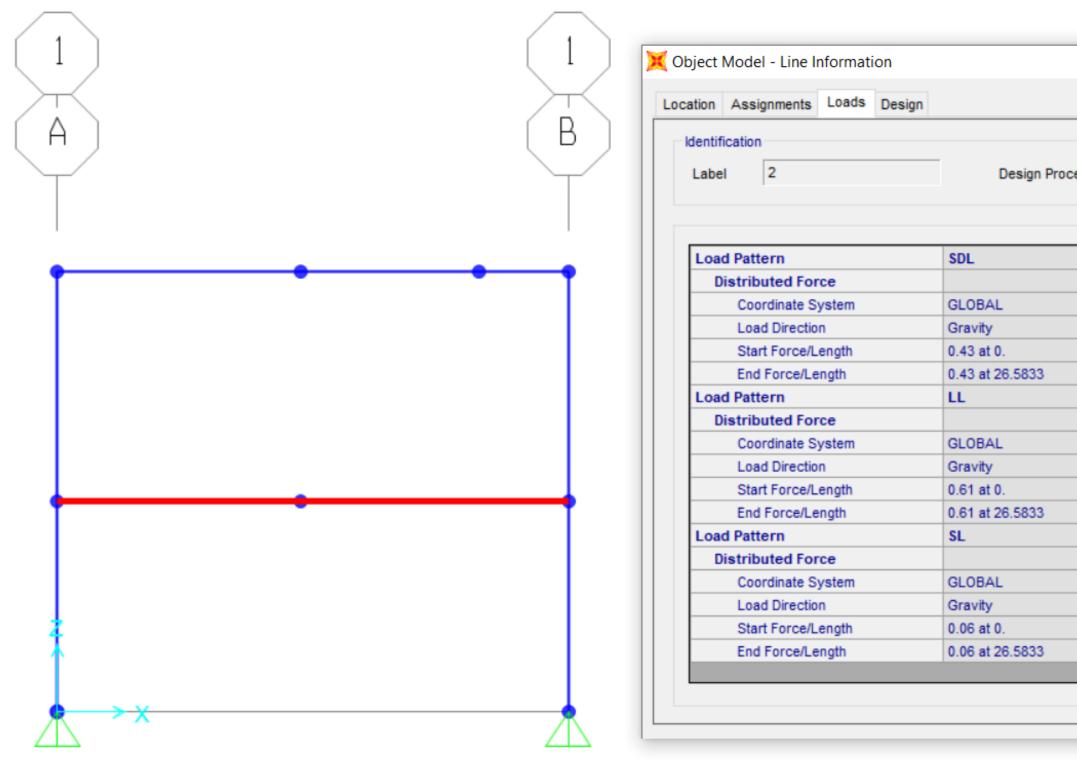
JOINT 2 LOADING

×
Kip, ft, F 🗸
Update Display Modify Display OK Cancel



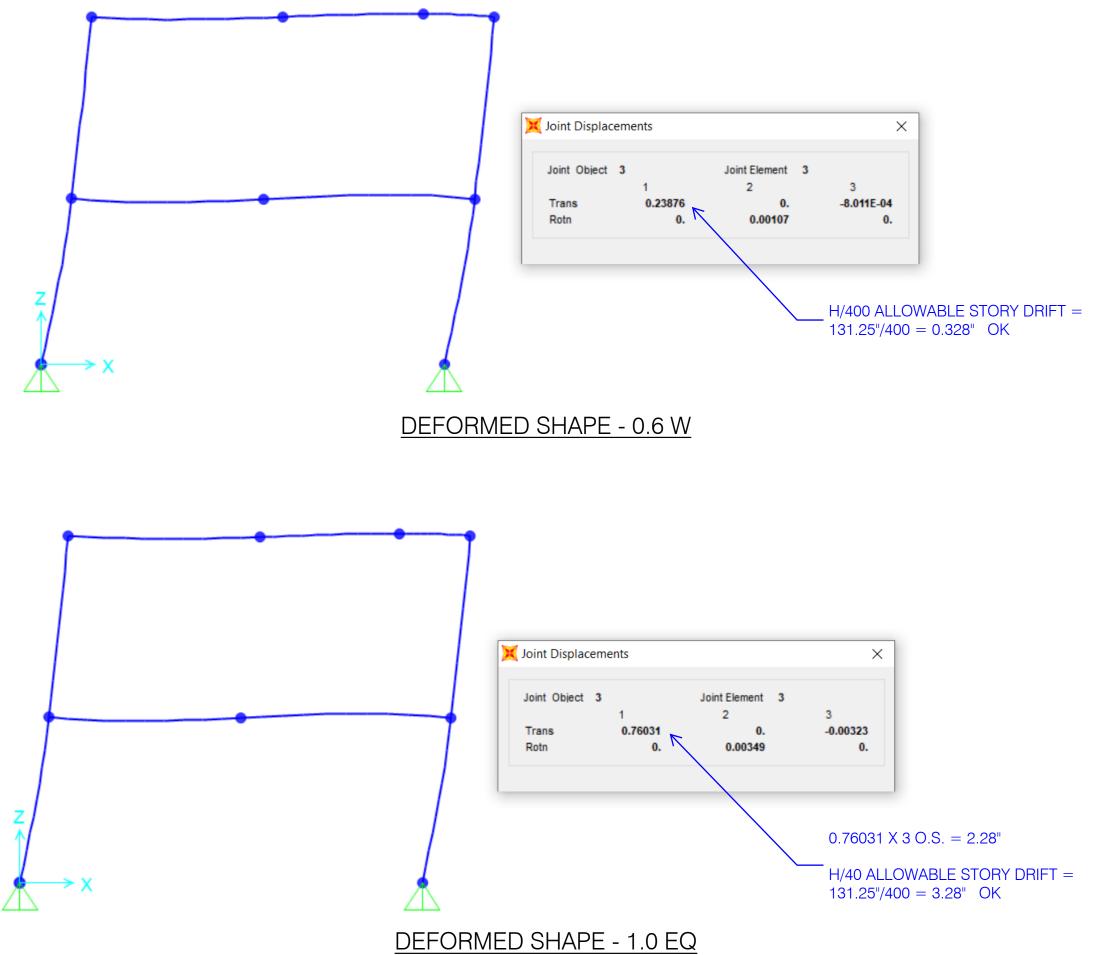
UPPER FRAME ELEMENT LOADING

	×
rocedure	Steel Frame 🗸
	Kip, ft, F 🗸
_	Update Display Modify Display
	OK Cancel

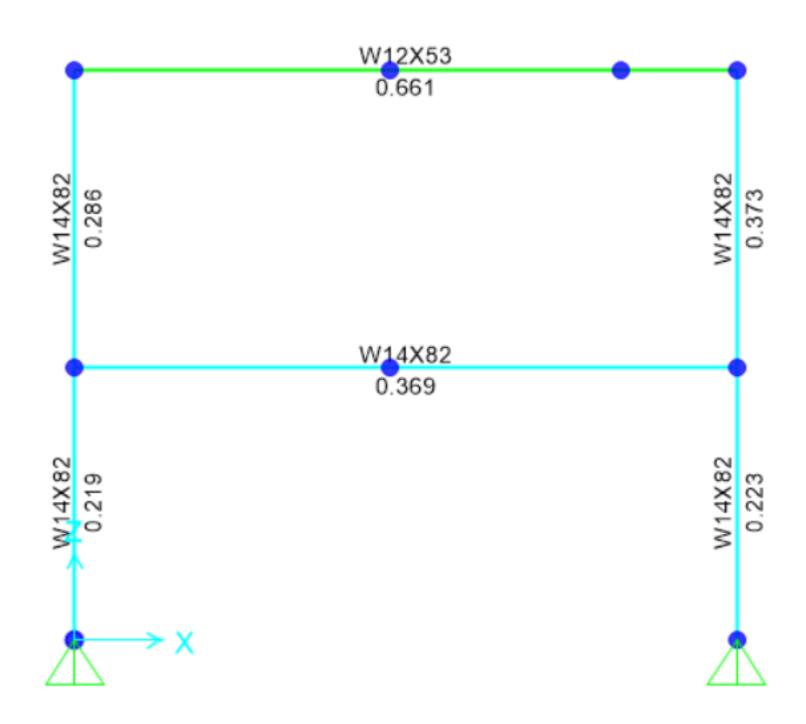


LOWER FRAME ELEMENT LOADING

	×
cedure	Steel Frame V
	Kip, ft, F 🗸
	Update Display Modify Display
	Cancel



STRENGTH DESIGN CHECK (PER AISC 360)





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Design: Fastening point:	Moment Frame Base Plate	Date:	3/3/2021

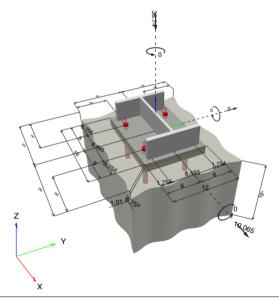
Specifier's comments:

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 :	$I_x \times I_y \times t = 16.000$ 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))

 $^{\rm R}$ - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]



Input data and results must be checked for conformity with the existing conditions and for plausibility! PROFIS Engineering (c) 2003-2021 Hilti AG, FL-9494 Schaan Hilti is a registered Trademark of Hilti AG, Schaan



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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, (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

Cas	e 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;	yes	14
			$M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;$		

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 co	ompressive strain:	0.	.04 [‰]				

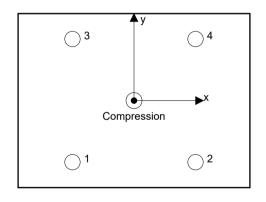
max. concrete compressive strain.0.04 [m]max. concrete compressive stress:191 [psi]resulting tension force in (x/y)=(0.000/0.000):0.000 [kip]resulting compression force in (x/y)=(0.000/0.000):36.740 [kip]

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

3 Tension load

	Load N _{ua} [kip]	Capacity ଦ 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

Input data and results must be checked for conformity with the existing conditions and for plausibility! PROFIS Engineering (c) 2003-2021 Hilti AG, FL-9494 Schaan Hilti is a registered Trademark of Hilti AG, Schaan





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4 Shear load

	Load V _{ua} [kip]	Capacity ف 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)
φV _{stee}	el ≥ 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]	ф _{steel}	ϕ_{eb}	∮ V _{sa,eq} [kip]	V _{ua} [kip]
21.089	0.650	0.800	10.966	2.516



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4.2 Steel failure (with lever arm)

V^{M}_{s}	$=\frac{\alpha_{M}\cdot M_{s}}{L_{b}}$	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
ϕV^M_s	$\geq V_{ua}$	ACI 318-11 Table D.4.1.1

Variables

α_{M}	f _{u,min} [psi]	N _{ua} [kip]	∮ 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}	ϕV_s^M [kip]	V _{ua} [kip]			
4.687	0.650	3.047	2.516			



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4.3 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_{b} \right]$	ACI 318-11 Eq. (D-41)
$\phi V_{cpg} \ge V_{ua}$	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)	
$A_{\rm Nc0} = 9 h_{\rm ef}^2$	ACI 318-11 Eq. (D-5)
$\Psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}}\right) \le 1.0$	ACI 318-11 Eq. (D-8)
$\Psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}} \right) \le 1.0$	ACI 318-11 Eq. (D-10)
$\psi_{\text{ cp,N}} = \text{MAX}\left(\frac{c_{a,\text{min}}}{c_{\underline{ac}}}, \frac{1.5h_{\text{ef}}}{c_{ac}}\right) \leq 1.0$	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]	$\Psi_{\text{ec1,N}}$	$\Psi_{ec2,N}$	$\psi_{\text{ed},\text{N}}$	$\Psi_{\text{cp},\text{N}}$	N _b [kip]
701.87	324.00	1.000	1.000	1.000	1.000	24.942
Results						
V _{cpg} [kip]	ϕ_{concrete}	$\phi_{seismic}$	$\phi_{nonductile}$	φ V _{cpg} [kip]	V _{ua} [kip]	
108.061	0.700	1.000	1.000	75.642	10.065	-



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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	Moment Frame Base Plate	Date:	3/3/2021
Fastening point:			

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 ω₀.

Fastening meets the design criteria!

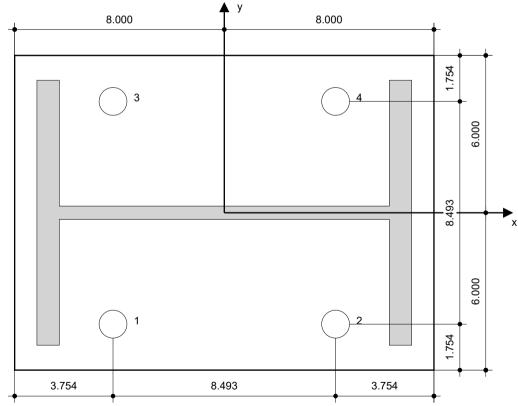


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Company: Address:		Page: Specifier:	7	
Phone I Fax:		E-Mail:		
Design: Fastening point:	Moment Frame Base Plate	Date:	3/3/2021	
6 Installation da	ata			
		Anchor type and diameter: Hex Head A	STM F 1554 GR.	
		36 1		
Profile: W shape (AIS	C), W14X82; (L x W x T x FT) = 14.300 in. x 10.100 in. x	Item number: not available		
0.510 in. x 0.855 in.				
Hole diameter in the fi	ixture: d _f = 1.062 in.	Installation torque: -		
Plate thickness (input): 0.750 in.	Hole diameter in the base material: - in.		

Recommended plate thickness: not calculated

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	У	C _{-x}	C+x	c_y	с _{+у}
1	-4.246	-4.246	-	-	-	-
2	4.246	-4.246	-	-	-	-
3	-4.246	4.246	-	-	-	-
4	4.246	4.246	-	-	-	-

Input data and results must be checked for conformity with the existing conditions and for plausibility! PROFIS Engineering (c) 2003-2021 Hilti AG, FL-9494 Schaan Hilti is a registered Trademark of Hilti AG, Schaan



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Company:		Page:	8
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	Moment Frame Base Plate	Date:	3/3/2021
Fastening point:			

7 Remarks; Your Cooperation Duties

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the
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 case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data
 or programs, arising from a culpable breach of duty by you.

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	ОК	Case 1	304	-	-	-	-
В	3.36	0.384	8.75	11.38	ОК	Case 1	369	-	-	-	-
С	3.36	0.384	8.75	12.58	ОК	Case 1	334	-	-	-	-
1	4	0.288	13.89	16.88	ОК	Case 3	296	-	-	-	-
2	4	0.288	13.89	16.38	ОК	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	ОК	Case 2-6	291	-	-	-	-
В	9.2	0.288	31.94	25.40	NG	Case 2-6	453	B to C, 15/32 8d @ 6"	540	-	-
С	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	ОК	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	ОК	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	ОК	Case 2-6	150	-	-	-	-
В	15.8	0.848	18.63	26.19	ОК	Case 2-6	754	-	-	-	-
С	11.79	0.848	13.90	16.08	ОК	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	ОК	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	ОК	Case 2-6	84	-	-	-	-
6	0.25	0.848	0.29	4.15	ОК	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	
	ALLOWABLE TEN	SION LOADS (lbs)
MODEL NO.	ASD	LRFD
HDU2-SDS2.5	3075	4147
HDU4-SDS2.5	4565	6157
HDU5-SDS2.5	5645	7614
HDU8-SDS2.5	6765	9124

SST FL	OOR TO FLOOR STRA	APS
MODEL NO.	ALLOWABLE TEN	SION LOADS (lbs)
WIODEL NO.	ASD	LRFD
CMSTC16	4690	6326
CMST14	ASD LRFD 4690 63 6475 87	8733
CMST12	9215	12429

	SST HANGERS	
MODEL NO.	ALLOWABLE TEN	SION LOADS (lbs)
WIODEL NO.	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

Tekla Tedds	Project Yaroslavsky	/ Residence			Job Ref. 8119	
Fast + Epp 323 Dean Street, Suite #3	Section				Sheet no./rev.	
Brooklyn, NY 11217	Typical Wal	I Footing (F1)			1	
	Calc. by BW	Date 3/3/2021	Chk'd by	Date	App'd by	Date
Foundation analysis & design <u>FOOTING ANALYSIS</u> Length of foundation	(ACI318) in ac	cordance with <i>A</i> L _x = 1 ft	\Cl318-14		Tedds calc	ulation versio
Width of foundation Foundation area		$L_y = 1.5 \text{ ft}$ $A = L_x \times L_y =$	- 1 5 ft 2			
Depth of foundation		h = 10 in	- 1.5 1			
Depth of soil over foundation		h _{soil} = 18 in				
Density of concrete		$\gamma_{\rm conc} = 150.0$	lb/ft ³			
Wall no.1 details	2.313 ksf		2.313 k	51		
Wall no.1 details Width of wall position in y-axis		l _{y1} = 8 in y1 = 9 in				
Soil properties Gross allowable bearing pressur Density of soil Angle of internal friction Design base friction angle Coefficient of base friction	e	qallow_Gross = γsoil = 125.0 φb = 30.0 de δbb = 19.3 d tan(δbb) = 0 .	lb/ft ³ g əg			
Foundation loads						
Self weight		Fswt = h * γcc	nc = 125 psf			
		11 /00				

Fast + Epp	Project Yaroslavsk	y Residence			Job Ref. 8119	
Fast + ⊏pp 323 Dean Street, Suite #3	Section				Sheet no./rev.	
Brooklyn, NY 11217	Typical Wa	II Footing (F1)			2	
	Calc. by BW	Date 3/3/2021	Chk'd by	Date	App'd by	Date
Wall no.1 loads per linear foot						
Dead load in z		F _{Dz1} = 1.5 ki	ps			
Live load in z		F _{Lz1} = 1.5 ki	ps			
Footing analysis for soil and st	ability					
Load combinations per ASCE 7 1.0D (0.525) 1.0D + 1.0L (0.925)	7-16					
Combination 2 results: 1.0D + 7	1 01					
Forces on foundation per linea						
Force in z-axis		F _{dz} = γ _D * A	* (Fswt + Fsoil) + [,]	γd * Fdz1 + γl * F	- Lz1 = 3.5 kips	
Moments on foundation per lin	ear foot	10 11	, , , , , , ,	, ,- ,		
Moment in y-axis, about y is 0		M _{dy} = γ _D * (A kip_ft	X * (Fswt + Fsoil) *	L _y / 2) + γ _D * (F	^E Dz1 * y 1) + γ∟ * (F	Lz1 * y1) = 2.0
Uplift verification						
Vertical force		Fdz = 3.469	kips			
				PASS - Fou	undation is not	subject to ι
Stability against sliding						
Resistance due to base friction		$F_{RFriction} = m$	ax(F _{dz} , 0 kN) * t	an(δьь) = 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 = 0.00)0 in		
Strip base pressures						
		$q_1 = F_{dz} * (1)$	- 6 * e _{dy} / L _y) / (L _y * 1 ft) = 2.31	2 ksf	
		•	+ 6 * e _{dy} / L _y) /	,	12 ksf	
Minimum base pressure			1,q2) = 2.312 ks			
Maximum base pressure		$q_{max} = max($	q1,q2) = 2.312 k	sf		
Allowable bearing capacity						
Allowable bearing capacity			_Gross = 2.5 ksf			
		qmax / qallow =			ovoodo daal	baca pro-
		PA22 -	Allowable bea	aring capacity	exceeds desigr	i base pres
FOOTING DESIGN (ACI318)						
In accordance with ACI318-14						
Material details						
Compressive strength of concrete	Э	f'c = 4000 ps	si			
Yield strength of reinforcement		fy = 60000 p	si			
Compression-controlled strain lin	nit (21.2.2)	εty = 0.0020	D			
Cover to reinforcement		Cnom = 3 in				
Concrete type		Normal weig	ght			
Concrete modification factor		$\lambda = 1.00$				
Wall type		Concrete				

Fast + Epp	Project Yaroslavsk	y Residence			Job Ref. 8119	
323 Dean Street, Suite #3	Section				Sheet no./rev.	
Brooklyn, NY 11217	Typical Wa	Ill Footing (F1)			3	
	Calc. by BW	Date 3/3/2021	Chk'd by	Date	App'd by	Date
Analysis and design of concre	te 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 linea	r foot					
Ultimate force in z-axis		$F_{uz} = \gamma D * A$	* (Fswt + Fsoil) + γ	γd * Fdz1 + γl *	F _{Lz1} = 4.8 kips	
Moments on foundation per lin	ear foot					
Ultimate moment in y-axis, about		M _{uy} = γ _D * (A kip_ft	.* (Fswt + Fsoil) *	Ly / 2) + γ _D * (I	^Ξ Dz1 * y 1) + γL * (Fι	z1 * y1) = 3.6
Eccentricity of base reaction						
Eccentricity of base reaction in y-	axis	e _{uy} = M _{uy} / F	uz - Ly / 2 = 0.00	0 in		
Strip base pressures						
		qu1 = Fuz * (1	- 6 * e _{uy} / L _y) /	(L _y * 1 ft) = 3.1	75 ksf	
		$q_{u2} = F_{uz} * (1)$	+ 6 * e _{uy} / L _y) /	(L _y * 1 ft) = 3.	1 75 ksf	
Minimum ultimate base pressure		$q_{umin} = min(c)$	Ju1, Qu2) = 3.175	ksf		
Maximum ultimate base pressure)	q _{umax} = max((qu1,qu2) = 3.175	i ksf		
		Shear diag	ram (kips)			
		2.1				
0.0				(
		-2.1				
	0.2	Moment diag	ram (kip_ft)			
0	0.2			(
v						
		0.8				
Moment design, y direction, po Ultimate bending moment	sitive momer		12 kin 4			
		Mu.y.max = 0.2	243 kip_ft : 8.0 in c/c botto	m		
	ovided	Asy.bot.prov = 0		411 		
Tension reinforcement provided			18 * L _x * h = 0.2	2 16 in ²		
Tension reinforcement provided Area of tension reinforcement pro		$A_{s \min} = 0.00$				
Tension reinforcement provided		As.min = 0.00		of reinforcem	ent provided exc	eeds minin
Tension reinforcement provided Area of tension reinforcement pro	7.6.1.1)				ent provided exc	eeds minin
Tension reinforcement provided Area of tension reinforcement pro Minimum area of reinforcement (7.6.1.1) ent (7.7.2.3)		PASS - Area o * h, 18 in) = 18	in		
Tension reinforcement provided Area of tension reinforcement pro Minimum area of reinforcement (7.6.1.1) ent (7.7.2.3)	s _{max} = min(3 ASS - Maximum	PASS - Area o * h, 18 in) = 18	in inforcement s		
Tension reinforcement provided Area of tension reinforcement pro Minimum area of reinforcement (Maximum spacing of reinforcement	7.6.1.1) ent (7.7.2.3)	s _{max} = min(3 ASS - Maximum d = h - c _{nom} -	PASS - Area (* h, 18 in) = 18 permissible rei	in inforcement s 8 in	pacing exceeds	

Fast + Epp	Project Yaroslavsk	y Residence			Job Ref. 8119		
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Typical Wa	ll Footing (F1)			Sheet no./rev 4		
	Calc. by BW	Date 3/3/2021	Chk'd by	Date	App'd by	Date	
Depth to neutral axis		c = a / β1 =	0.804 in				
Strain in tensile reinforcement		$\epsilon_t = 0.003$ *	d / c - 0.003 = (0.02194			
Minimum tensile strain(7.3.3.1)		εmin = 0.004	= 0.00400				

PASS - Tensile strain exceeds minimum required

Mn = Asy.bot.prov * fy * (d - a / 2) = **14.753** kip_ft

 $\phi_{f} = min(max(0.65 + 0.25 * (\epsilon_{t} - \epsilon_{ty}) / (0.005 - \epsilon_{ty}), 0.65), 0.9) = 0.900$

 $\phi M_n = \phi_f * M_n = \textbf{13.278} \text{ kip}_ft$

 $M_{u.y.max} \ / \ \varphi M_n = \textbf{0.018}$

PASS - Design moment capacity exceeds ultimate moment load

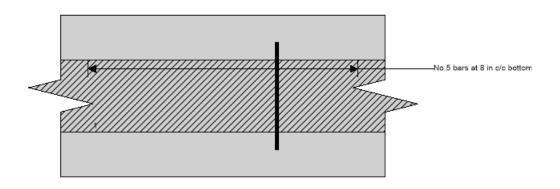
One-way shear design, y direction

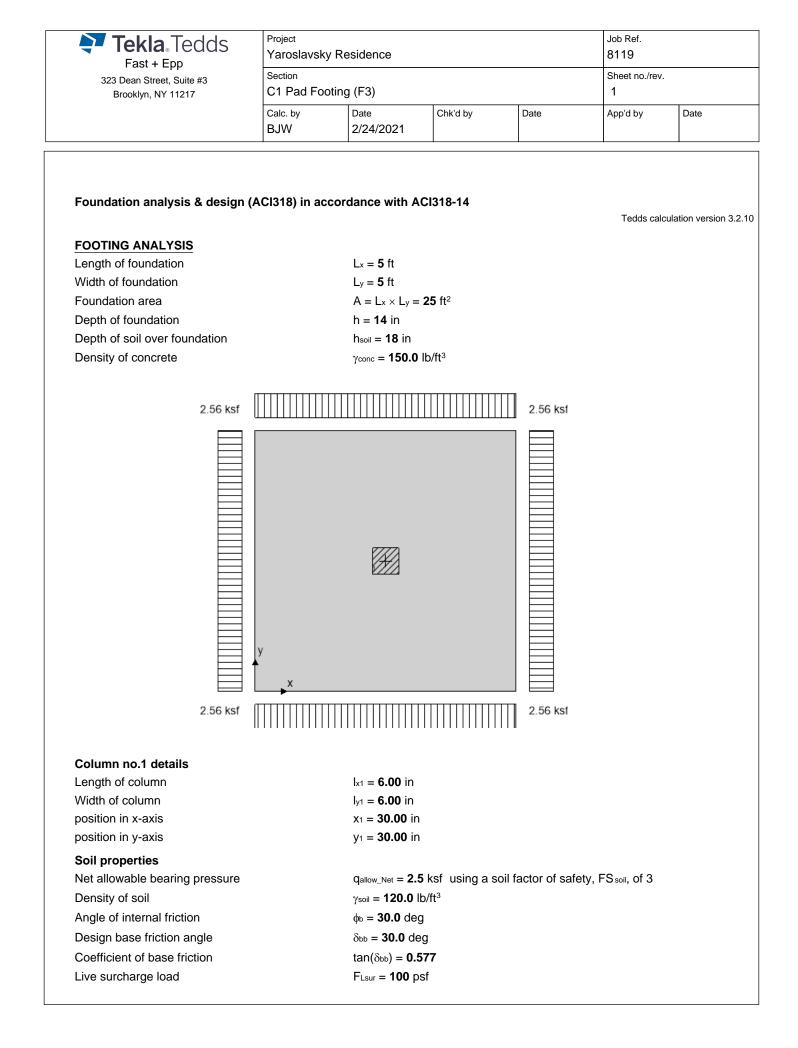
Nominal moment capacity

Design moment capacity

Flexural strength reduction factor

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.

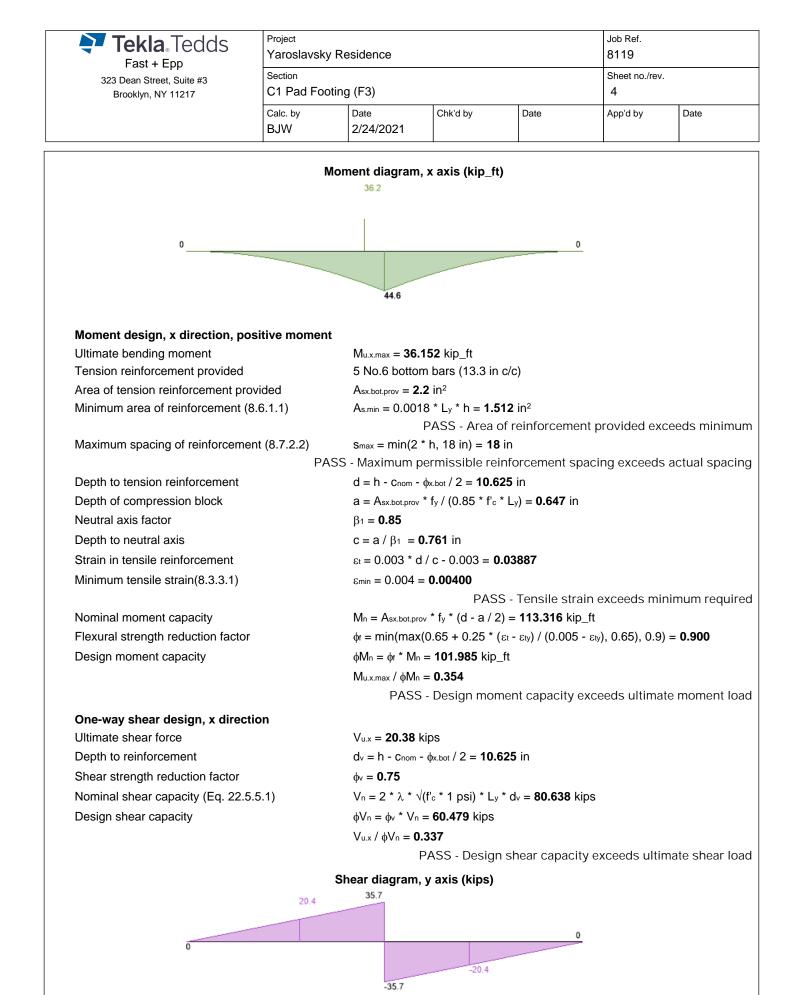


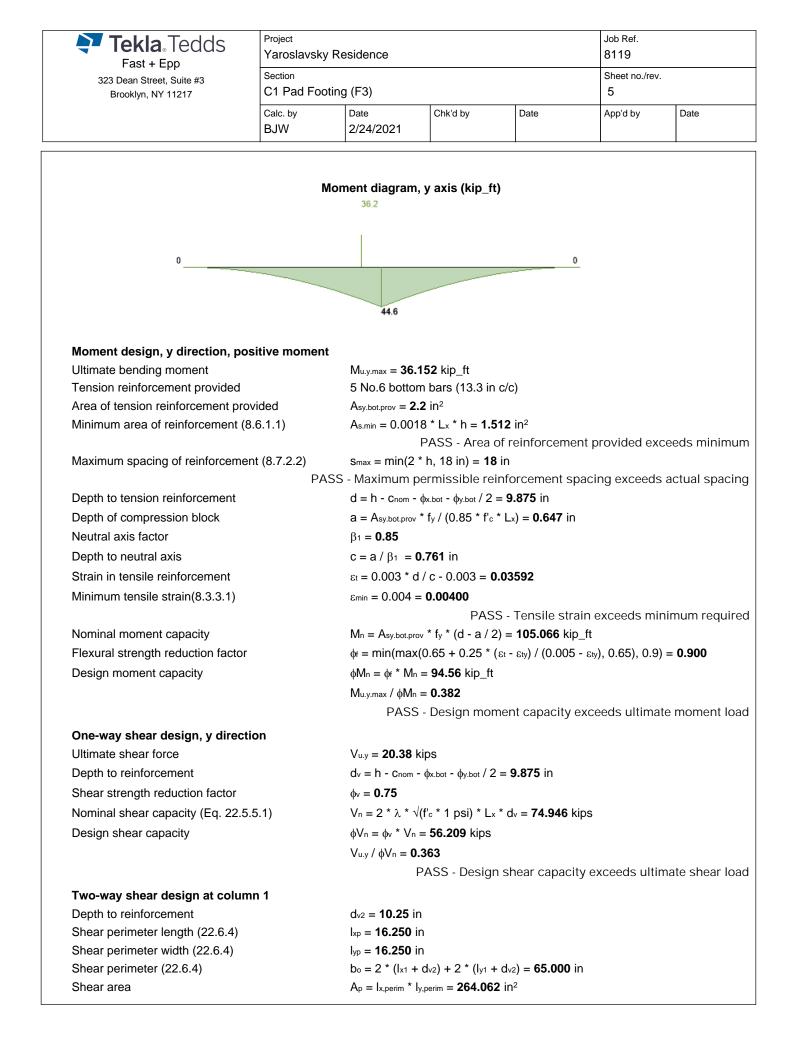


Tekla Tedds	Project Yaroslavsł	ky Residence	8119			
Fast + Epp 323 Dean Street, Suite #3	Section				Sheet no./rev.	
Brooklyn, NY 11217	C1 Pad Fo	ooting (F3)		1	Sheet no./rev. 2 App'd by $T_{D} * F_{Dz1} + \gamma_{L} * F_{Lz1} + \gamma_{L} * F_{Lz1} + \gamma_{L} * F_{Lz1} + \gamma_{L} +$	
	Brooklyn, NY 11217 C1 Pad Footi Calc. by BJW weight weight umn no.1 loads d load in z load in z w load in z ting analysis for soil and stability d combinations per ASCE 7-16 D (0.330)	Date 2/24/2021	Chk'd by	Date	App'd by	Date
Self weight		Fswt = h * γcor	c = 175 psf			
Soil weight		$F_{soil} = h_{soil} * \gamma$	_{soil} = 180 psf			
Column no.1 loads						
Dead load in z		F _{Dz1} = 12.6 k	ips			
Live load in z		F _{Lz1} = 26.5 k	ips			
Snow load in z		Fsz1 = 27.7 k	ips			
Footing analysis for soil and s	tability					
1.0D (0.330) 1.0D + 1.0L (0.775)						
		S + 0 45W				
	0.75 + 0.75	3 + 0.45W				
						4 L WO * Eo. 4 -
		64.0 kips		γL A ILSUFΤ)	YD I DZI T YL I LZ	ιτγ 5 Ι 5 2Ι -
Momente en foundation						
		$M_{dy} = \gamma p * (\Delta$	* (Fourt + Fooil) *	L _ν / 2) ± γι * Δ	* Flour * Ly / 2 + v	ף * (Ep-1 * v1)
) + γs * (Fsz1 *)			
Moment in v-axis, about v is 0						ר * (FD-71 * V1)
) + γs * (Fsz1 * y		•	b (1 b21 j1)
Unlift verification				,		
Vertical force		F _{dz} = 64 kips				
				PASS - Fo	undation is not s	subject to up
Bearing resistance						5
Eccentricity of base reaction						
Eccentricity of base reaction in x-	axis	$e_{dx} = M_{dx} / F_{dx}$	_{iz} - L _x / 2 = 0 in			
Eccentricity of base reaction in y-		$e_{dy} = M_{dy} / F_{dy}$	$L_z - L_y / 2 = 0$ 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	8 * e _{dy} / L _y) / (L _x	* Ly) = 2.56 ksf	
					* * L _y) = 2.56 ksf	
					× * Ly) = 2.56 ksf	
Minimum base pressure			,q ₂ ,q ₃ ,q ₄) = 2.5			
Maximum base pressure		$q_{max} = max(c)$	1,q2,q3,q4) = 2.	56 ksf		
Allowable bearing capacity		a	$hot + ((h + h_{coil}))$	* γsoil) / FSsoil =	2.607 ksf	
Allowable bearing capacity Allowable bearing capacity				1		
		$q_{max} / q_{allow} =$	0.982			

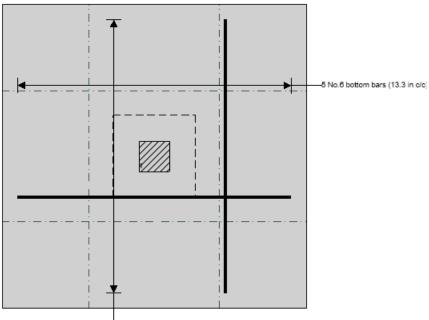
In accordance with ACI318-14

Fast + Epp	Project Yaroslavsk	ky Residence			Job Ref. 8119	
323 Dean Street, Suite #3	Section	(Sheet no./rev	
Brooklyn, NY 11217	C1 Pad Fo	oting (F3)			3	
	Calc. by BJW	Date 2/24/2021	Chk'd by	Date	App'd by	Date
Material details						
Compressive strength of concrete	e	f'c = 4000 ps	i			
Yield strength of reinforcement		f _y = 60000 ps	si			
Compression-controlled strain lin	nit (21.2.2)	εty = 0.00200)			
Cover to reinforcement		$C_{nom} = 3 in$				
Concrete type		Normal weig	ht			
Concrete modification factor		$\lambda = 1.00$				
Column type		Concrete				
Analysis and design of concre	te 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 +	I.6L + 0.5S					
Forces on foundation						
Ultimate force in z-axis		F _{uz} = γ _D * A * 86.0 kips	(Fswt + Fsoil) + [/]	γ∟ * A * FLsur + γ	′D * FDz1 + γL * FL	z1 + γs * Fsz1
Moments on foundation						
Ultimate moment in x-axis, about	x is 0				* FLsur * Lx / 2 + 7	γd * (Fdz1 * x
				(1) = 215.0 kip_		
Ultimate moment in y-axis, about	y is 0				* FLsur * Ly / 2 + y	yd * (Fdz1 * y
		γl * (Flz1 * y1) + γs * (Fsz1 *)	(1) = 215.0 kip_	ft	
Eccentricity of base reaction						
Eccentricity of base reaction in x-	axis	$e_{ux} = M_{ux} / F_{u}$	$_{\rm uz}$ - L _x / 2 = 0 in			
Eccentricity of base reaction in y-	axis	$e_{uy} = M_{uy} / F_u$	_{iz} - L _y / 2 = 0 in			
Pad base pressures						
		•			* L _y) = 3.441 ks	
		•			× * Ly) = 3.441 ks	
					x * Ly) = 3.441 ks	
Minimum ultime - t - t -					_{-x} * L _y) = 3.441 k	st
Minimum ultimate base pressure Maximum ultimate base pressure			u1,qu2,qu3,qu4) = qu1,qu2,qu3,qu4)			
Maximum unimate base pressure	:			= 3.441 KSI		
		Shear diagram	, x axis (kips)			
	20.4					
					0	
0					-	
			-20.4	-		
		-35.7				

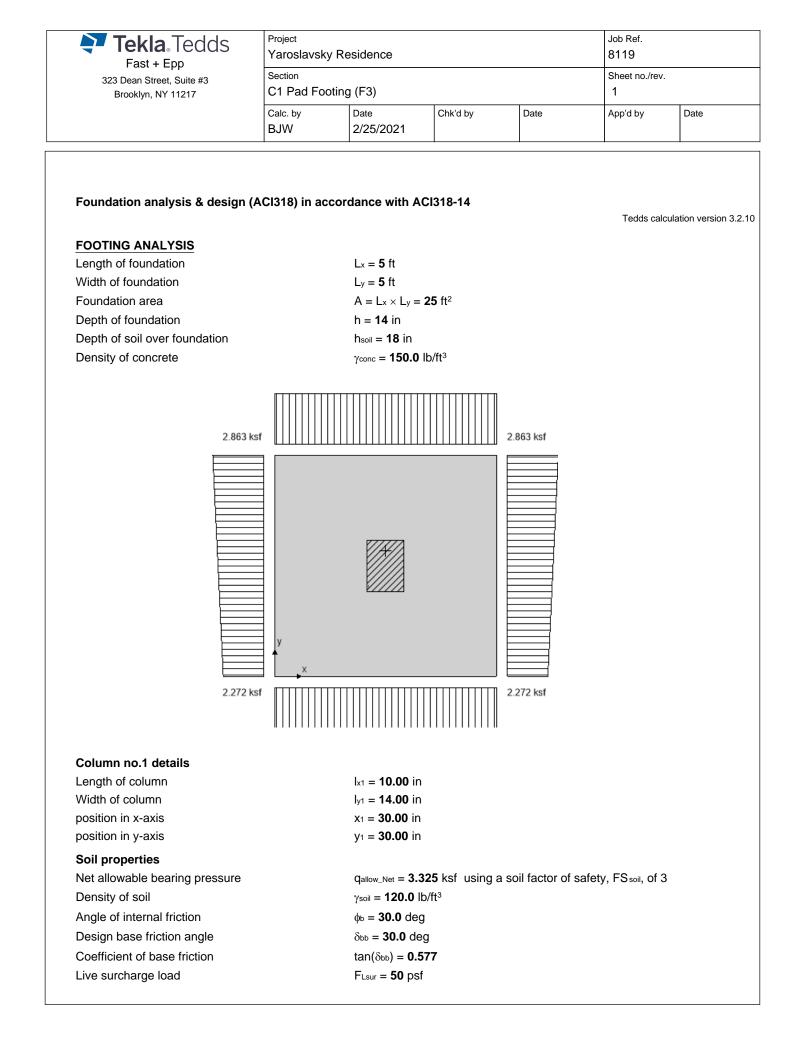




Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	^{Project} Yaroslavsky R	esidence	Job Ref. 8119					
	Section C1 Pad Footing (F3)			Sheet no./rev. 6				
	Calc. by BJW	Date 2/24/2021	Chk'd by	Date	App'd by	Date		
Surcharge loaded area		Asur = Ap - Ix1	* l _{y1} = 228.062	! in ²				
Ultimate bearing pressure at center of shear area		q _{up.avg} = 3.441 ksf						
Ultimate shear load		Fup = γD * FDz1 + γL * FLz1 + γS * FSz1 + γD * Ap * Fswt + γD * Asur * Fsoil + γL *						
		* FLsur - qup.avg * Ap = 66.041 kips						
Ultimate shear stress from vertical load		v _{ug} = max(F _{up} / (b _o * d _{v2}),0 psi) = 99.123 psi						
Column geometry factor (Table 22.6.5.2)		$\beta = I_{y1} / I_{x1} = 1.00$						
Column location factor (22.6.5.3)		αs =40						
Concrete shear strength (22.6.5.2)		v _{cpa} = (2 + 4 / β) * λ * √(f' _c * 1 psi) = 379.473 psi						
		v _{cpb} = (α _s * d _{v2} / b _o + 2) * λ * √(f' _c * 1 psi) = 525.425 psi						
		v _{cpc} = 4 * λ * √(f' _c * 1 psi) = 252.982 psi						
	v _{cp} = min(v _{cpa} ,v _{cpb} ,v _{cpc}) = 252.982 psi							
Shear strength reduction factor		$\phi_{\rm V}=0.75$						
Nominal shear stress capacity (E	q. 22.6.1.2)	Vn = Vcp = 25	2.982 psi					
Design shear stress capacity (8.5.1.1(d))		$\phi v_n = \phi_v * v_n = $ 189.737 psi						
		$V_{ug} / \phi V_n = 0.5$	522					



5 No.6 bottom bars (13.3 in c/c)



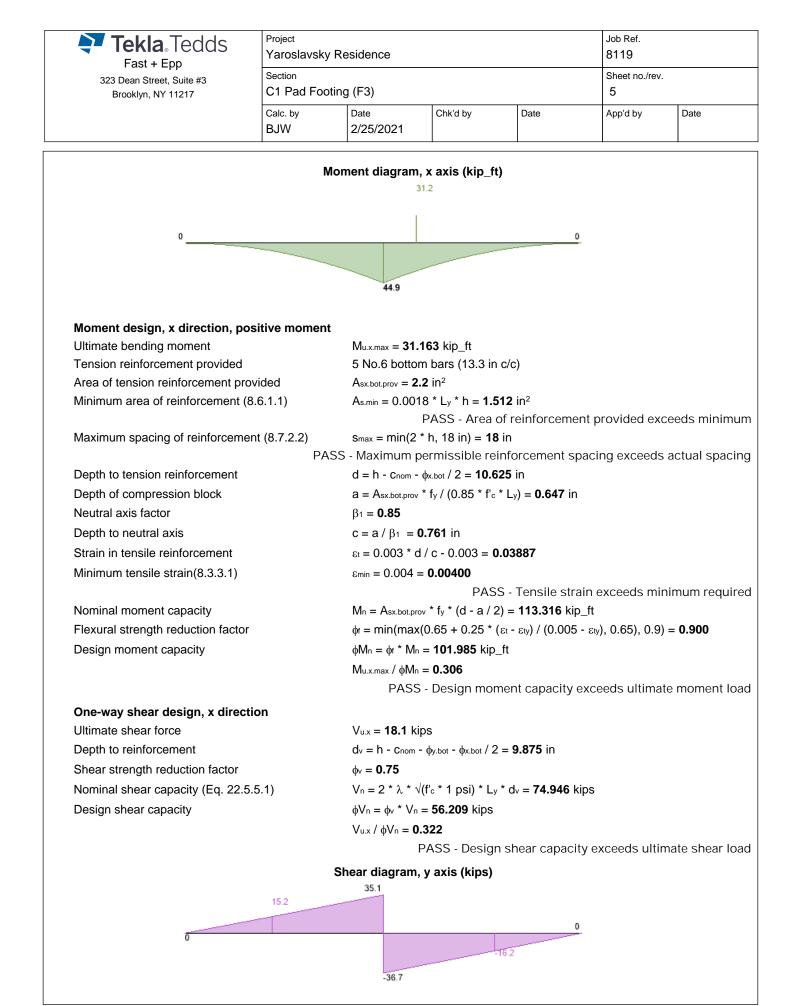
Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Project Yaroslavsky	y Residence			Job Ref. 8119				
	Section	Sheet no./rev.	Sheet no./rev.						
	C1 Pad Foo	oting (F3)	2						
	Calc. by BJW	Date 2/25/2021	Chk'd by	Date	App'd by	Date			
Self weight		Fswt = h * γcon	c = 175 psf						
Soil weight		$F_{soil} = h_{soil} * \gamma$	$F_{soil} = h_{soil} * \gamma_{soil} = 180 \text{ psf}$						
Column no.1 loads									
Dead load in z		F _{Dz1} = 30.0 k	ips						
Live load in z		F _{Lz1} = 20.0 kips							
Snow load in z	Fsz1 = 7.6 kip	Fsz1 = 7.6 kips							
Dead load in y	F _{Dy1} = 0.2 kip	F _{Dy1} = 0.2 kips							
Live load in y		F _{Ly1} = 1.2 kip	S						
Wind load in y	Fwy1 = 4.3 ki	os							
Seismic load in y	F _{Ey1} = 8.0 kip	S							
Footing analysis for soil and s	tability								
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 (
1.0D + 0.75L + 0.75S + 0.45W (0	-								
1.0D + 0.75L + 0.75R + 0.45W (0	,								
(1.0 + 0.10 * S _{DS})D + 0.75L + 0.7	5S + 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 * Sps)D +	0.75L + 0.75S +	0.525E						
Forces on foundation		_							
Force in y-axis		•	F _{Dy1} + γ _L * F _{Ly1} + γ _E * F _{Ey1} = 5.3 kips						
Force in z-axis		Fdz = γD * A * (Fswt + Fsoil) + γL * A * FLsur + γD * FDz1 + γL * FLz1 + γS * FSz1 = 64.2 kips							
Moments on foundation									
Moment in x-axis, about x is 0		$M_{dx} = \gamma D * (A$	* (Fswt + Fsoil) *	Lx / 2) + γ _L * A	* FLsur * Lx / 2 + γι	o * (FDz1 * X			
		γL * (FLz1 * X1) + γs * (Fsz1 * X1) = 160.5 kip_ft							
Moment in y-axis, about y is 0		M _{dy} = γ _D * (A * (F _{swt} + F _{soil}) * L _y / 2) + γ _L * A * F _{Lsur} * L _y / 2 + γ _D * (F _{Dz1} *							
		$y_1 + F_{Dy1} * h) + \gamma_L * (F_{Lz1} * y_1 + F_{Ly1} * h) + \gamma_S * (F_{Sz1} * y_1) + \gamma_E * (F_{Ey1} * h) = 1$ kip_ft							
Uplift verification									
•									
Vertical force		Fdz = 64.182	kips						

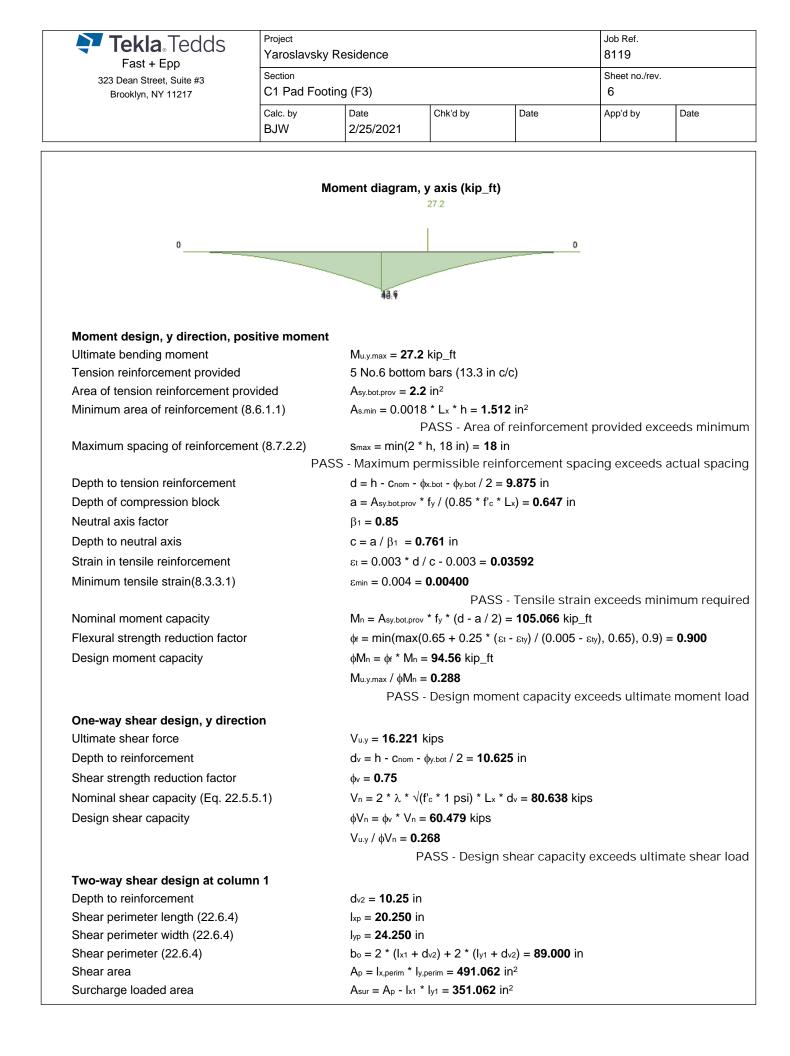
Fast + Epp					Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section C1 Pad Footing	ı (F3)				
	Calc. by BJW	Date 2/25/2021	Chk'd by	Date	App'd by	Date

Overturning moment	Motyl = γd * (Fdy1 * h) + γl * (Fly1 * h) + γε * (Fey1 * h) = 6.16 kip_ft
Resisting moment	$M_{RyL} = -1 * (\gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_L * A * F_{Lsur} * L_y / 2) + \gamma_D * (F_{Dz1} * C_{sur} + C_{s$
	$(y_1 - L_y)) + \gamma_L * (F_{Lz1} * (y_1 - L_y)) + \gamma_S * (F_{Sz1} * (y_1 - L_y)) = -160.46 \text{ kip_ft}$
Factor of safety	abs(M _{RyL} / M _{OTyL}) = 26.057
	PASS - Overturning moment safety factor exceeds the minimum of 1.0
Stability against sliding	
Resistance due to base friction	F _{RFriction} = max(F _{dz} , 0 kN) * tan(δ _{bb}) = 37.056 kips
Stability against sliding in y direction	
Total sliding resistance	Fry = Frefriction = 37.056 kips
Factor of safety	abs(F _{Ry} / F _{dy}) = 7.02
	PASS - Sliding factor of safety exceeds the minimum of 1.0
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	
	q1 = F _{dz} * (1 - 6 * e _{dx} / L _x - 6 * e _{dy} / L _y) / (L _x * L _y) = 2.272 ksf
	$q_2 = F_{dz} * (1 - 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y) = 2.863 \text{ ksf}$
	$q_3 = F_{dz} * (1 + 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y) = 2.272 \text{ ksf}$
	q4 = Fdz * (1 + 6 * edx / Lx + 6 * edy / Ly) / (Lx * Ly) = 2.863 ksf
Minimum base pressure	$q_{min} = min(q_{1},q_{2},q_{3},q_{4}) = 2.272 \text{ ksf}$
Maximum base pressure	$q_{max} = max(q_1,q_2,q_3,q_4) = 2.863$ ksf
Allowable bearing capacity	
Allowable bearing capacity	qallow = qallow_Net + ((h + hsoil) * γsoil) / FSsoil = 3.432 ksf
	q _{max} / q _{allow} = 0.834
	PASS - Allowable bearing capacity exceeds design base pressure
FOOTING DESIGN (ACI318)	
In accordance with ACI318-14	
Material details	
Compressive strength of concrete	ť c = 4000 psi
Yield strength of reinforcement	f _y = 60000 psi
Compression-controlled strain limit (21.2.2)	εty = 0.00200
Cover to reinforcement	Cnom = 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)

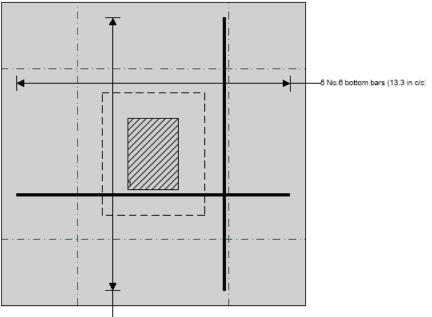
Tekla Tedds	Project Yaroslavsky	Residence			Job Ref. 8119	
Fast + Epp 323 Dean Street, Suite #3	Section				Sheet no./rev.	
Brooklyn, NY 11217	C1 Pad Foo	ting (F3)			4	
	Calc. by BJW	Date 2/25/2021	Chk'd by	Date	App'd by	Date
1.2D + 1.6L + 0.5S (0.355)						
1.2D + 1.6L + 0.5R (0.336)						
1.2D + 1.0L + 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	3)					
1.2D + 1.0L + 0.5S + 1.0W (0.292)					
1.2D + 1.0L + 0.5R + 1.0W (0.273)					
(1.2 + 0.2 * SDS)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		$F_{uy} = \gamma D * F_{c}$	οy1 + γL * FLy1 = 2	.2 kips		
Ultimate force in z-axis		F _{uz} = γ _D * A 84.4 kips	* (Fswt + Fsoil) + γ	/L * A * FLsur + γ	'D * Fdz1 + γl * Flz'	1 + γs * Fsz1 :
Moments on foundation						
Ultimate moment in x-axis, about a	k is 0	Mux = γD * (A	• * (Fswt + Fsoil) *	Lx / 2) + γL * A	* F _{Lsur} * L _x / 2 + γι	o * (FDz1 * X1
		γl * (Flz1 * x	1) + γs * (Fsz1 * x	a) = 211.1 kip_	ft	
Ultimate moment in y-axis, about	/ is 0	Muy = γD * (A	• (Fswt + Fsoil) *	Ly / 2) + γ _L * A	* FLsur * Ly / 2 + γι	o * (Fdz1 *
		y1+F _{Dy1} * h)	+ γL * (FLz1 * y1+	·F _{Ly1} * h) + γs *	(Fsz1 * y1) = 213.6	6 kip_ft
Eccentricity of base reaction						
Eccentricity of base reaction in x-a		$e_{ux} = M_{ux} / F$	^{fuz} - L _x / 2 = 0 in			
Eccentricity of base reaction in y-a	ixis	$e_{uy} = M_{uy} / F$	uz - Ly / 2 = 0.35	7 in		
Pad base pressures						
		• •			* L _y) = 3.257 ksf	
					× * L _y) = 3.499 ksf	
					× * L _y) = 3.257 ksf	
					_x * Ly) = 3.499 ks	f
Minimum ultimate base pressure		-	qu1,qu2,qu3,qu4) =			
Maximum ultimate base pressure			(qu1,qu2,qu3,qu4) =	= 3.499 ksf		
		Shear diagram 35.9	ı, x axis (kips)			
	18.1					
					0	
					0	
0						





Tekla Tedds Fast + Epp					Job Ref. 8119	
323 Dean Street, Suite #3	Section C1 Pad Footing	ı (F3)			Sheet no./rev. 7	
	Calc. by BJW	Date 2/25/2021	Chk'd by	Date	App'd by	Date

Ultimate bearing pressure at center of shear area	q _{up.avg} = 3.475 ksf
Ultimate shear load	$F_{up} = \gamma_D * F_{Dz1} + \gamma_L * F_{Lz1} + \gamma_S * F_{Sz1} + \gamma_D * A_p * F_{swt} + \gamma_D * A_{sur} * F_{soil} + \gamma_L * A_{sur}$
	* FLsur - qup.avg * Ap = 61.386 kips
Ultimate shear stress from vertical load	v _{ug} = max(F _{up} / (b ₀ * d _{v2}),0 psi) = 67.291 psi
Column geometry factor (Table 22.6.5.2)	$\beta = I_{y1} / I_{x1} = 1.40$
Column location factor (22.6.5.3)	αs =40
Concrete shear strength (22.6.5.2)	v _{cpa} = (2 + 4 / β) * λ * √(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 * λ * √(f' _c * 1 psi) = 252.982 psi
	vcp = min(vcpa, Vcpb, Vcpc) = 252.982 psi
Shear strength reduction factor	$\phi_{\rm V} = 0.75$
Nominal shear stress capacity (Eq. 22.6.1.2)	vn = v _{cp} = 252.982 psi
Design shear stress capacity (8.5.1.1(d))	φvn = φv * vn = 189.737 psi
	v _{ug} / φvn = 0.355
	PASS - Design shear stress capacity exceeds ultimate shear stress load



5 No.6 bottom bars (13.3 in c/c)

Fast + Epp

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 =
Wall length	L =

$f_{T} = 200.000 \text{ ft}^{2}$ = 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

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

5.83 ft

L =

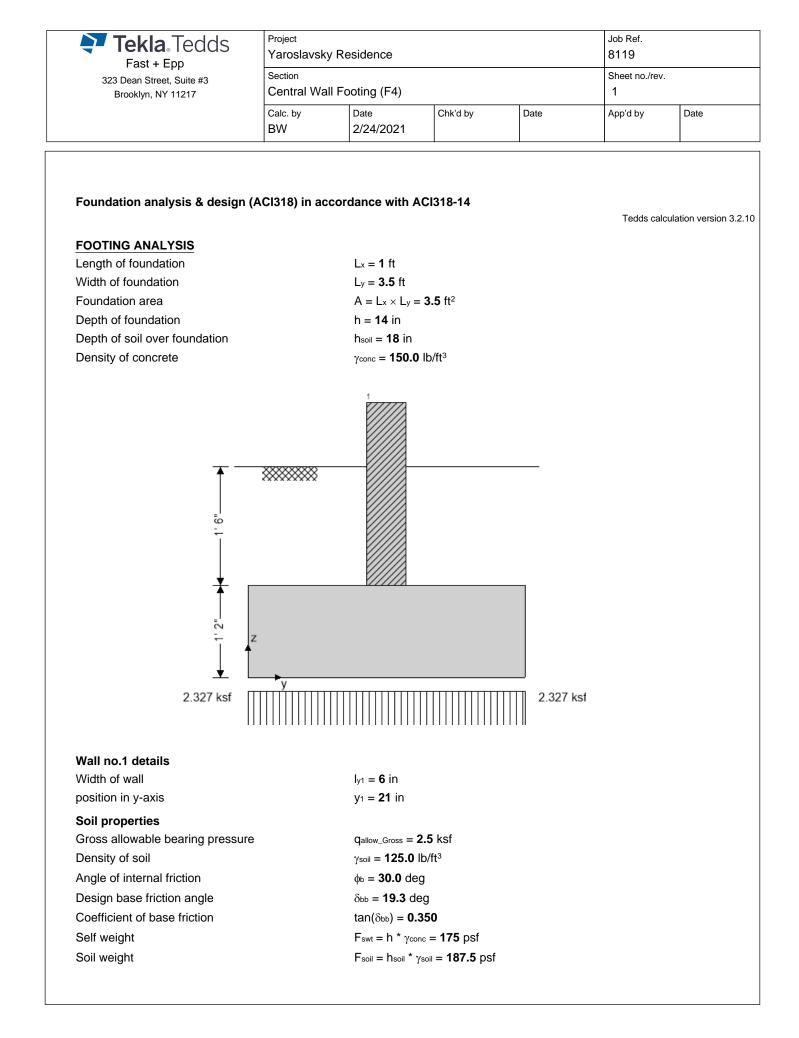
POINT LOADS (FROM TEDDS OUTPUT):

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:

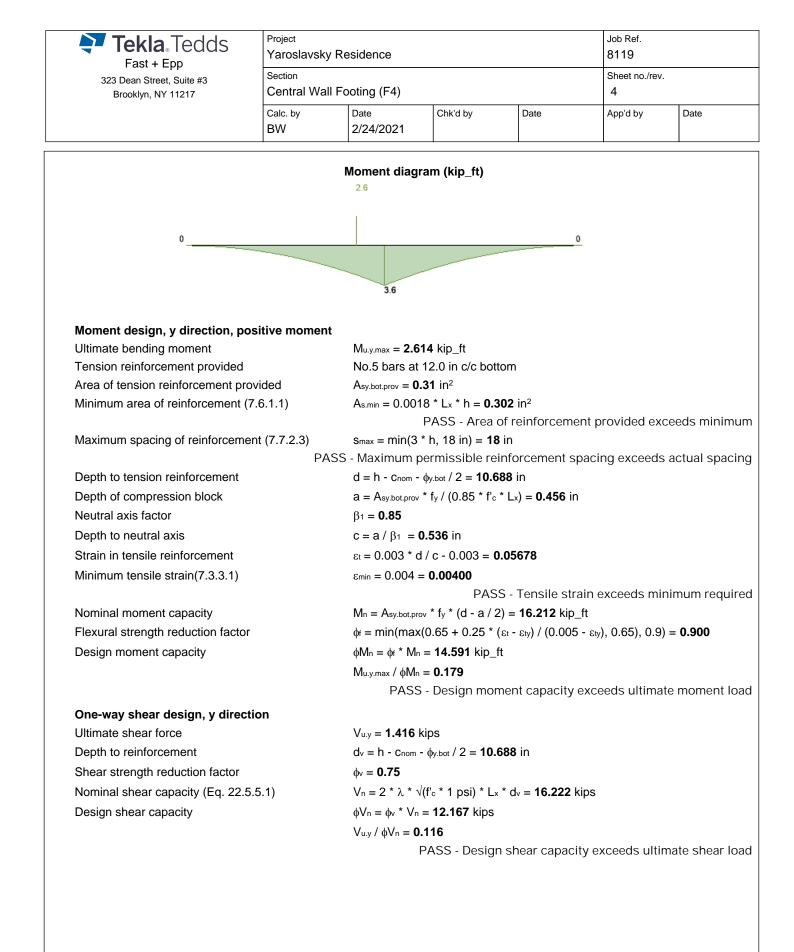
Dead load	DL =	0	plf	0	klf
Superimposed dead load	SDL =	2022.857	plf	2.023	klf
Live load	LL =	1422.857	plf	1.423	klf
Snow load	SL =	805.714	plf	0.806	klf
Seismic load	EQ =	1371.429	plf	1.371	klf

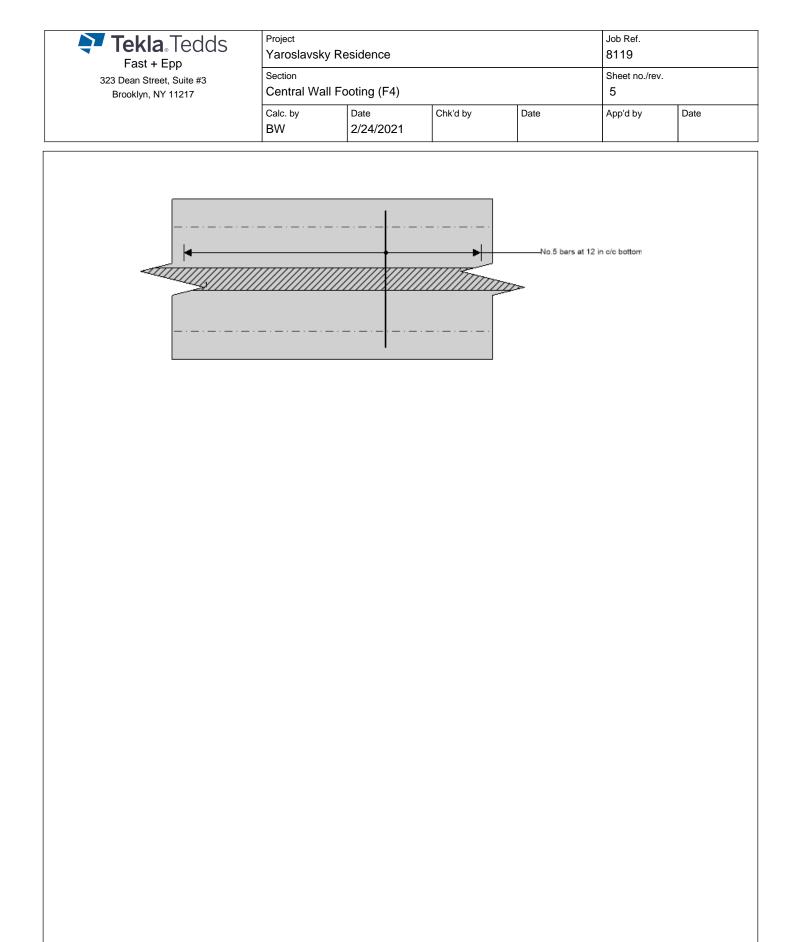
	Line Load Total						
SDL	3.05	klf					
LL	2.79	klf					
SL	0.81	klf					
EQ	1.37	klf					

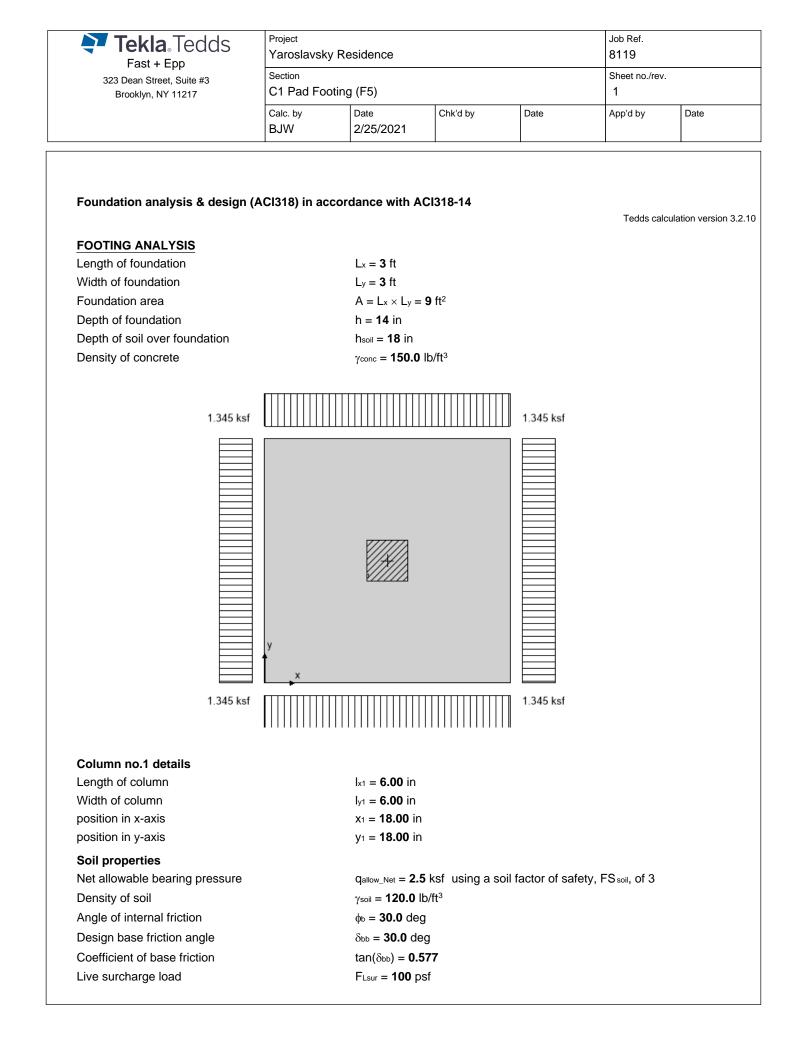


Tekla Tedds	Project Yaroslavsky	y Residence			Job Ref. 8119	
Fast + Epp 323 Dean Street, Suite #3	Section				Sheet no./rev.	
Brooklyn, NY 11217	Central Wa	ll Footing (F4)	2	2		
	Calc. by BW	Date 2/24/2021	Chk'd by	Date	App'd by	Date
Wall no.1 loads per linear foot						
Dead load in z		FDz1 = 3.1 kip	S			
Live load in z		F _{Lz1} = 2.8 kip				
Snow load in z		Fsz1 = 0.8 kip	S			
Seismic load in z		F _{Ez1} = 1.4 kip	S			
Footing analysis for soil and st	ability					
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 * SDS)D + 0.7E (0.668	-					
1.0D + 0.75L + 0.75Lr + 0.45W (0 1.0D + 0.75L + 0.75S + 0.45W (0	-					
1.0D + 0.75L + 0.75R + 0.45W (0	-					
(1.0 + 0.10 * S _D s)D + 0.75L + 0.75	-	931)				
0.6D + 0.6W (0.296)	0010.0202 (0					
(0.6 - 0.14 * S _{DS})D + 0.7E (0.341)						
Combination 14 results: (1.0.						
Combination 14 results: (1.0 +)	0.10 * Sps)D +	0.75L + 0.75S +	0.525E			
Forces on foundation per linea		0.75L + 0.75S + 0	0.525E			
				γd * Fdz1 + γl *	Flz1 + γs * Fsz1 + γ	e * Fez1 = 8.1
Forces on foundation per linea	r foot	F _{dz} = γ _D * A *		γd * Fdz1 + γl *	Flz1 + γs * Fsz1 + γ	e * Fez1 = 8.1
Forces on foundation per linea Force in z-axis	r foot	F _{dz} = γ _D * A * kips	(Fswt + Fsoil) + γ	· · ·	Flz1 + γs * Fsz1 + γ Fdz1 * y1) + γl * (Fl	
Forces on foundation per linea Force in z-axis Moments on foundation per lin	r foot	F _{dz} = γ _D * A * kips M _{dy} = γ _D * (A	(Fswt + Fsoil) + γ	L _y / 2) + γ _D * (F		
Forces on foundation per linea Force in z-axis Moments on foundation per lin Moment in y-axis, about y is 0	r foot	F _{dz} = γ _D * A * kips M _{dy} = γ _D * (A	(Fswt + Fsoil) + γ * (Fswt + Fsoil) *	L _y / 2) + γ _D * (F		
Forces on foundation per linea Force in z-axis Moments on foundation per lin	r foot	F _{dz} = γ _D * A * kips M _{dy} = γ _D * (A	(Fswt + Fsoil) + γ * (Fswt + Fsoil) * 'E * (FEz1 * Y1) =	L _y / 2) + γ _D * (F		
Forces on foundation per linea Force in z-axis Moments on foundation per lin Moment in y-axis, about y is 0 Uplift verification	r foot	$F_{dz} = \gamma_D * A *$ kips $M_{dy} = \gamma_D * (A (F_{Sz1} * y_1) + \gamma_1)$	(Fswt + Fsoil) + γ * (Fswt + Fsoil) * 'E * (FEz1 * Y1) =	L _y / 2) + γ _D * (f : 14.3 kip_ft		z1 * y 1) + γs *
Forces on foundation per linea Force in z-axis Moments on foundation per lin Moment in y-axis, about y is 0 Uplift verification	r foot	$F_{dz} = \gamma_D * A *$ kips $M_{dy} = \gamma_D * (A (F_{Sz1} * y_1) + \gamma_1)$	(Fswt + Fsoil) + γ * (Fswt + Fsoil) * 'E * (FEz1 * Y1) =	L _y / 2) + γ _D * (f : 14.3 kip_ft	^E dz1 * y1) + γι * (Fι	z1 * y 1) + γs *
Forces on foundation per linea Force in z-axis Moments on foundation per lin Moment in y-axis, about y is 0 Uplift verification Vertical force	r foot	$F_{dz} = \gamma_D * A *$ kips $M_{dy} = \gamma_D * (A (F_{Sz1} * y_1) + \gamma)$ $F_{dz} = 8.146$ k	(Fswt + Fsoil) + γ * (Fswt + Fsoil) * 'E * (FEz1 * Y1) =	L _y / 2) + γ⊳ * (f = 14.3 kip_ft PASS - Fo	^E Dz1 * y1) + γL * (FL undation is not s	z1 * y 1) + γs *
Forces on foundation per linea Force in z-axis Moments on foundation per lin Moment in y-axis, about y is 0 Uplift verification Vertical force Stability against sliding	r foot	$F_{dz} = \gamma_D * A *$ kips $M_{dy} = \gamma_D * (A (F_{Sz1} * y_1) + \gamma)$ $F_{dz} = 8.146$ k	(Fswt + Fsoil) + γ * (Fswt + Fsoil) * re * (Fεz1 * y1) = ips	L _y / 2) + γ⊳ * (f = 14.3 kip_ft PASS - Fo	^E Dz1 * y1) + γL * (FL undation is not s	z1 * y 1) + γs *
Forces on foundation per linea Force in z-axis Moments on foundation per lin Moment in y-axis, about y is 0 Uplift verification Vertical force Stability against sliding Resistance due to base friction Bearing resistance	r foot	$F_{dz} = \gamma_D * A *$ kips $M_{dy} = \gamma_D * (A (F_{Sz1} * y_1) + \gamma)$ $F_{dz} = 8.146$ k	(Fswt + Fsoil) + γ * (Fswt + Fsoil) * re * (Fεz1 * y1) = ips	L _y / 2) + γ⊳ * (f = 14.3 kip_ft PASS - Fo	^E Dz1 * y1) + γL * (FL undation is not s	z1 * y 1) + γs *
Forces on foundation per linea Force in z-axis Moments on foundation per lin Moment in y-axis, about y is 0 Uplift verification Vertical force Stability against sliding Resistance due to base friction	r foot ear foot	$F_{dz} = \gamma_D * A *$ kips $M_{dy} = \gamma_D * (A$ (Fsz1 * y1) + γ Fdz = 8.146 k FRFriction = ma	(Fswt + Fsoil) + γ * (Fswt + Fsoil) * re * (Fεz1 * y1) = ips	L _y / 2) + γ _D * (f = 14.3 kip_ft PASS - Fo an(δ _{bb}) = 2.85 1	^E Dz1 * y1) + γL * (FL undation is not s	z1 * y 1) + γs *
Forces on foundation per linea Force in z-axis Moments on foundation per lin Moment in y-axis, about y is 0 Uplift verification Vertical force Stability against sliding Resistance due to base friction Bearing resistance Eccentricity of base reaction Eccentricity of base reaction in y-	r foot ear foot	$F_{dz} = \gamma_D * A *$ kips $M_{dy} = \gamma_D * (A$ (Fsz1 * y1) + γ Fdz = 8.146 k FRFriction = ma	(Fswt + Fsoil) + γ * (Fswt + Fsoil) * re * (Fεz1 * y1) = ips x(Fdz, 0 kN) * ta	L _y / 2) + γ _D * (f = 14.3 kip_ft PASS - Fo an(δ _{bb}) = 2.85 1	^E Dz1 * y1) + γL * (FL undation is not s	z1 * y 1) + γs *
Forces on foundation per linea Force in z-axis Moments on foundation per lin Moment in y-axis, about y is 0 Uplift verification Vertical force Stability against sliding Resistance due to base friction Bearing resistance Eccentricity of base reaction	r foot ear foot	$F_{dz} = \gamma_D * A *$ kips $M_{dy} = \gamma_D * (A$ (Fsz1 * y1) + γ Fdz = 8.146 k FRFriction = mathematical edy = M_{dy} / For	(F _{swt} + F _{soil}) + γ * (F _{swt} + F _{soil}) * ^{re} * (F _{Ez1} * y ₁) = ips x(F _{dz} , 0 kN) * ta z - Ly / 2 = 0.00	L _y / 2) + γ _D * (f = 14.3 kip_ft PASS - Fo an(δ _{bb}) = 2.85 1 00 in	Dz1 * y1) + γι * (Fι undation is not s	z1 * y 1) + γs *
Forces on foundation per linea Force in z-axis Moments on foundation per lin Moment in y-axis, about y is 0 Uplift verification Vertical force Stability against sliding Resistance due to base friction Bearing resistance Eccentricity of base reaction Eccentricity of base reaction in y-	r foot ear foot	$F_{dz} = \gamma_D * A *$ kips $M_{dy} = \gamma_D * (A$ (Fsz1 * y1) + γ Fdz = 8.146 k FRFriction = ma edy = M_dy / Fo q_1 = F_{dz} * (1 + 1)	(Fswt + Fsoil) + γ * (Fswt + Fsoil) * re * (Fεz1 * y1) = ips x(Fdz, 0 kN) * ta	L _y / 2) + γ _D * (f = 14.3 kip_ft PASS - Fo an(δ _{bb}) = 2.85 1 00 in L _y * 1 ft) = 2.32	^E dz1 * y1) + γL * (FL undation is not s I kips 27 ksf	z1 * y 1) + γs *

Fast + Epp	Project Yaroslavsk	y Residence			Job Ref. 8119			
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section Central Wa	II Footing (F4)			Sheet no./rev. 3	Sheet no./rev. 3		
	Calc. by BW	Date 2/24/2021	Chk'd by	Date	App'd by	Date		
Maximum base pressure		q _{max} = max(q ₁ ,q ₂) = 2.327 k	sf				
Allowable bearing capacity								
Allowable bearing capacity		qallow = qallow	Gross = 2.5 ksf					
		qmax / qallow =						
		PASS	- Allowable bea	aring capacity	exceeds desigr	n base press		
FOOTING DESIGN (ACI318)								
In accordance with ACI318-14								
Material details								
Compressive strength of concrete	e	f'c = 4000 p	si					
Yield strength of reinforcement		fy = 60000 p	osi					
Compression-controlled strain lin	nit (21.2.2)	εty = 0.0020	0					
Cover to reinforcement		Cnom = 3 in						
Concrete type		Normal wei	ght					
Concrete modification factor Wall type		$\lambda = 1.00$ Concrete						
Analysis and design of concre	te footing							
Load combinations per ASCE 1.4D (0.094) 1.2D + 1.6L + 0.5Lr (0.179)	7-16							
Combination 2 results: 1.2D +	1.6L + 0.5Lr							
Forces on foundation per linea	r foot							
Ultimate force in z-axis		Fuz = γD * A	* (Fswt + Fsoil) + [,]	γd * Fdz1 + γL *	F _{Lz1} = 9.6 kips			
Moments on foundation per lin	ear foot		. ,					
Ultimate moment in y-axis, about		M _{uy} = γ _D * (A kip_ft	A * (Fswt + Fsoil) *	Ly / 2) + γ _D * (Fdz1 * y1) + γι * (F	Lz1 * y1) = 16 .		
Eccentricity of base reaction								
Eccentricity of base reaction in y-	axis	$e_{uy} = M_{uy} / F$	⁵ uz - Ly / 2 = 0.00	10 in				
Strip base pressures								
		$q_{u1} = F_{uz} * ($	1 - 6 * e _{uy} / L _y) /	(L _y * 1 ft) = 2.7	756 ksf			
			1 + 6 * e _{uy} / L _y) /		756 ksf			
Minimum ultimate base pressure			qu1,qu2) = 2.756					
Maximum ultimate base pressure)		(qu1,qu2) = 2.756	o KST				
		Shear diag	ram (kips)					
	1.4	4.1						
					0			
0					-			

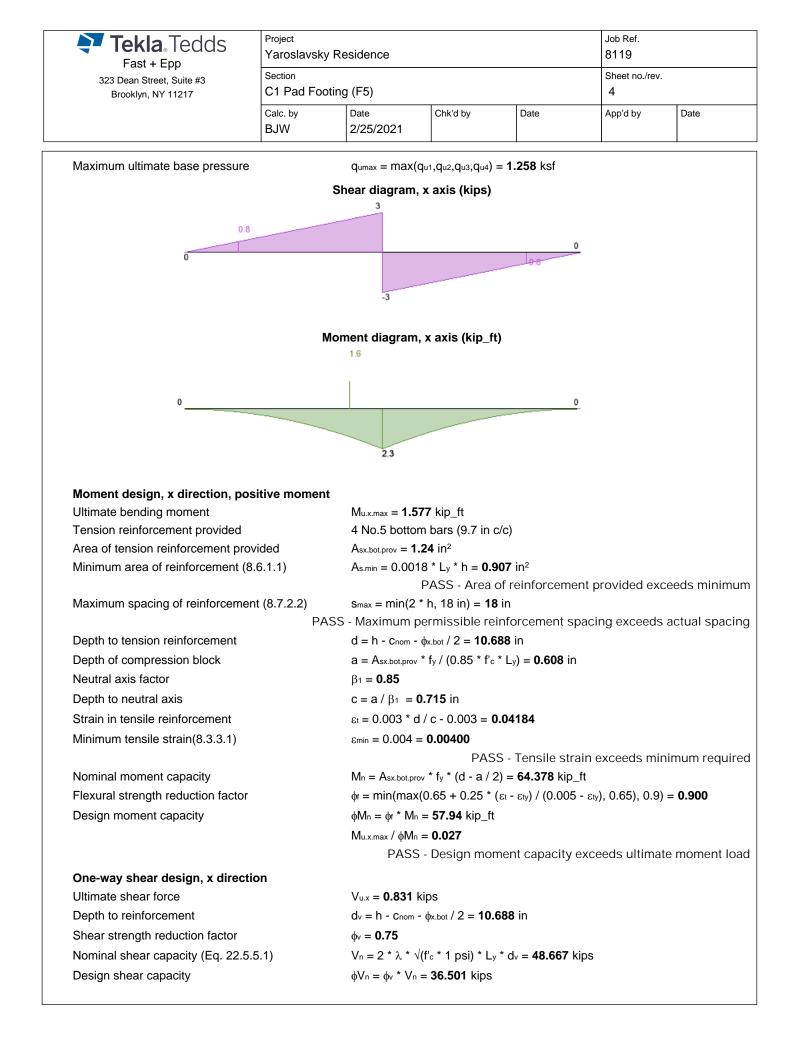


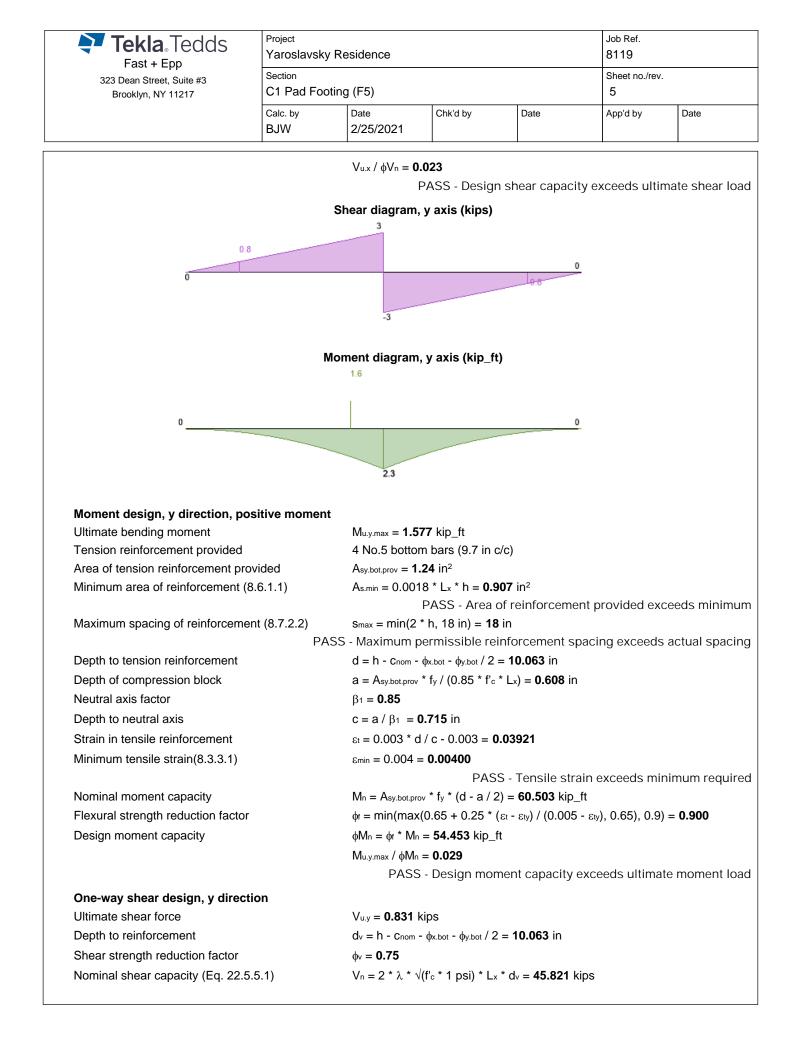




Tekla Tedds	Project Yaroslavsł	ky Residence			Job Ref. 8119	
Fast + Epp 323 Dean Street, Suite #3	Section				Sheet no./rev.	
Brooklyn, NY 11217	C1 Pad Fo	oting (F5)			2	
	Calc. by BJW	Date 2/25/2021	Chk'd by	Date	App'd by	Date
Self weight		Fswt = h * γcon	c = 175 psf			
Soil weight		$F_{soil} = h_{soil} * \gamma$	_{soil} = 180 psf			
Column no.1 loads						
Dead load in z		F _{Dz1} = 1.8 kip	DS			
Live load in z		F _{Lz1} = 2.5 kip	S			
Seismic load in z		F _{Ez1} = 7.9 kip	OS			
Footing analysis for soil and s	tability					
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	3)					
1.0D + 0.75L + 0.75Lr + 0.45W	-					
1.0D + 0.75L + 0.75S + 0.45W (-					
1.0D + 0.75L + 0.75R + 0.45W (-					
$(1.0 + 0.10 * S_{DS})D + 0.75L + 0.75L$		0.516)				
0.6D + 0.6W (0.127)	,	· · · · ·				
(0.6 - 0.14 * Sps)D + 0.7E (0.335)					
Combination 14 results: (1.0 +	0.10 * Sps)D +	+ 0.75L + 0.75S +	0.525E			
Forces on foundation						
Force in z-axis		$F_{dz} = \gamma D * A *$	$(F_{swt} + F_{soil}) + 7$	γL * A * FLsur + γ	γd * Fdz1 + γl * Flz	1 + γε * Fez1 =
		12.1 kips				
Moments on foundation						
Moment in x-axis, about x is 0		NA + / A		$ \sqrt{2} + \sqrt{2} $	* FLsur * Lx / 2 + γ	
		$Mdx = \gamma D^{(1)} (A)$	* (Fswt + Fsoil) *		•	D * (FDz1 * X1)
			* (Fswt + Fsoil) *) + γε * (Fez1 * >			d * (Fdz1 * X1)
Moment in y-axis, about y is 0		γι * (Flz1 * X1) + γε * (Fεz1 * >	(1) = 18.2 kip_f		
		γL * (FLz1 * X1) Mdy = γD * (A) + γε * (Fεz1 * >	(1) = 18.2 kip_f L _y / 2) + γ∟ * A	t * F _{Lsur} * Ly / 2 + γι	
		γL * (FLz1 * X1) Mdy = γD * (A) + γε * (Fez1 * > * (Fswt + Fsoil) *	(1) = 18.2 kip_f L _y / 2) + γ∟ * A	t * F∟sur * Ly / 2 + γι	
Moment in y-axis, about y is 0		γL * (FLz1 * X1) Mdy = γD * (A) + γε * (Fez1 * > * (Fswt + Fsoil) *) + γε * (Fez1 *)	(1) = 18.2 kip_f L _y / 2) + γ _L * A (1) = 18.2 kip_f	t *F _{Lsur} * Ly / 2 + γι t	o * (Fdz1 * y1)
Moment in y-axis, about y is 0 Uplift verification		γι * (F _{Lz1} * x1 M _{dy} = γ _D * (A γι * (F _{Lz1} * y1) + γε * (Fez1 * > * (Fswt + Fsoil) *) + γε * (Fez1 *)	(1) = 18.2 kip_f L _y / 2) + γ _L * A (1) = 18.2 kip_f	t * F∟sur * Ly / 2 + γι	o * (Fdz1 * y1)
Moment in y-axis, about y is 0 Uplift verification		γι * (F _{Lz1} * x1 M _{dy} = γ _D * (A γι * (F _{Lz1} * y1) + γε * (Fez1 * > * (Fswt + Fsoil) *) + γε * (Fez1 *)	(1) = 18.2 kip_f L _y / 2) + γ _L * A (1) = 18.2 kip_f	t *F _{Lsur} * Ly / 2 + γι t	o * (Fdz1 * y1)
Moment in y-axis, about y is 0 Uplift verification Vertical force		γι * (F _{Lz1} * x1 M _{dy} = γ _D * (A γι * (F _{Lz1} * y1) + γε * (Fez1 * > * (Fswt + Fsoil) *) + γε * (Fez1 *)	(1) = 18.2 kip_f L _y / 2) + γ _L * A (1) = 18.2 kip_f	t *F _{Lsur} * Ly / 2 + γι t	o * (Fdz1 * y1)
Moment in y-axis, about y is 0 Uplift verification Vertical force Bearing resistance	-axis	γL * (F _{L21} * X ₁) M _{dy} = γD * (A γL * (F _{L21} * y ₁) F _{dz} = 12.106) + γε * (Fez1 * > * (Fswt + Fsoil) *) + γε * (Fez1 *)	(1) = 18.2 kip_f L _y / 2) + γ _L * A /1) = 18.2 kip_f PASS - Fo	t *F _{Lsur} * Ly / 2 + γι t	o * (Fdz1 * y1)
Moment in y-axis, about y is 0 Uplift verification Vertical force Bearing resistance Eccentricity of base reaction		$\gamma_L * (F_{LZ1} * X_1)$ $M_{dy} = \gamma_D * (A$ $\gamma_L * (F_{LZ1} * y_1)$ $F_{dz} = 12.106$ $e_{dx} = M_{dx} / F_{c}$) + γε * (Fεz1 *) * (Fswt + Fsoil) *) + γε * (Fεz1 *) kips	(1) = 18.2 kip_f L _y / 2) + γ _L * A /1) = 18.2 kip_f PASS - Fo	t *F _{Lsur} * Ly / 2 + γι t	o * (Fdz1 * y1)
Moment in y-axis, about y is 0 Uplift verification Vertical force Bearing resistance Eccentricity of base reaction Eccentricity of base reaction in x		$\gamma_L * (F_{LZ1} * X_1)$ $M_{dy} = \gamma_D * (A$ $\gamma_L * (F_{LZ1} * y_1)$ $F_{dz} = 12.106$ $e_{dx} = M_{dx} / F_{c}$) + γε * (Fε ₂₁ *) * (F _{swt} + F _{soil}) *) + γε * (Fε ₂₁ *) kips	(1) = 18.2 kip_f L _y / 2) + γ _L * A /1) = 18.2 kip_f PASS - Fo	t *F _{Lsur} * Ly / 2 + γι t	o * (Fdz1 * y1)

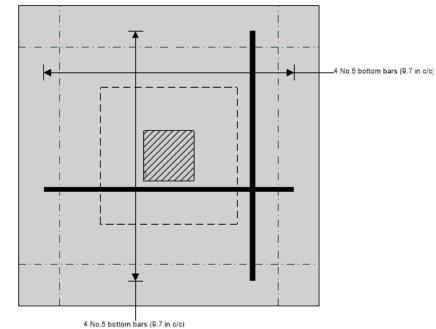
Fast + Epp	Project Yaroslavsk	ky Residence			Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section C1 Pad Fo	oting (F5)			Sheet no./rev 3	
	Calc. by BJW	Date 2/25/2021	Chk'd by	Date	App'd by	Date
		$q_2 = F_{dz} * (1 + 1)$	• 6 * e _{dx} / L _x + 6	5 * e _{dy} / L _y) / (Lx	* L _y) = 1.345 kst	ł
		$q_3 = F_{dz} * (1 + $	+ 6 * e _{dx} / L _x - 6	8 * e _{dy} / L _y) / (L _x	* Ly) = 1.345 kst	f
					× * L _y) = 1.345 ks	f
Minimum base pressure			,q ₂ ,q ₃ ,q ₄) = 1.3			
Maximum base pressure		$q_{max} = max(q)$	1,q2,q3,q4) = 1.	345 KST		
Allowable bearing capacity			<i>//</i> //			
Allowable bearing capacity				* γ _{soil}) / FS _{soil} =	2.607 ksf	
		q _{max} / q _{allow} =		ring conacity	exceeds desig	a baca proc
		PA33 -		ппу сарасну	exceeds design	i base pres
FOOTING DESIGN (ACI318)						
In accordance with ACI318-14						
Material details						
Compressive strength of concrete		f'c = 4000 ps				
Yield strength of reinforcement		fy = 60000 ps	si			
Compression-controlled strain lim	it (21.2.2)	$\epsilon_{ty} = 0.00200$				
Cover to reinforcement		c _{nom} = 3 in				
Concrete type		Normal weig	ht			
Concrete modification factor		$\lambda = 1.00$				
Column type		Concrete				
Analysis and design of concret	e footing					
Load combinations per ASCE 7 1.4D (0.015) 1.2D + 1.6L + 0.5Lr (0.036)	-16					
1.2D + 1.6L + 0.5S (0.036)						
Combination 2 results: 1.2D + 1	.6L + 0.5Lr					
Forces on foundation		F + ^ -		* ^ * ⊏	* - • -	44.01
Ultimate force in z-axis		r uz = γD " Α *	(F swt + Fsoil) +)	γ∟ A ⊓ FLsur + γ	νD * Fdz1 + γl * Fl	z1 = 11.3 KIP
Moments on foundation			*/	L / O) + -	* - * • / ~	* / = +
Ultimate moment in x-axis, about	x IS U			Lx / 2) + γL * A	* FLsur * Lx / 2 + 7	γD¨ (HDz1 * Χ
) = 17.0 kip_ft		* - * - / 2	* / - *
Ultimate moment in y-axis, about	y is u		* (Fswt + Fsoil) *) = 17.0 kip_ft	Ly / Ζ) + γι * Α	* FLsur * Ly / 2 + 🤉	/D " (HDz1 * Y
Eccentricity of base reaction						
Eccentricity of base reaction in x-a Eccentricity of base reaction in y-a			$z - L_x / 2 = 0$ in z - L _y / 2 = 0 in			
Pad base pressures		qu1 = Fuz * (1	- 6 * e _{ux} / L _x - 6	, .	* L _y) = 1.258 ks	
Pad base pressures		$q_{u3} = F_{uz} * (1)$		6 * e _{uy} / L _y) / (L	× * Ly) = 1.258 ks × * Ly) = 1.258 ks -× * Ly) = 1.258 k	sf





Tekla Tedds	Project Yaroslavsky F	Residence			Job Ref. 8119	
Fast + Epp 323 Dean Street, Suite #3 Brooklyn, NY 11217	Section C1 Pad Footin	ng (F5)			Sheet no./rev. 6	
	Calc. by BJW	Date 2/25/2021	Chk'd by	Date	App'd by	Date
Design shear capacity		$\phi V_n = \phi_v * V_n$	= 34.366 kips			
		$V_{u.y} / \phi V_n = 0$.024			
			PASS - Desigr	n shear capaci	ty exceeds ultir	nate shear lo
Two-way shear design at colur	nn 1					
Depth to reinforcement		dv2 = 10.375	in			
Shear perimeter length (22.6.4)		l _{xp} = 16.375	n			
Shear perimeter width (22.6.4)		l _{yp} = 16.375	n			
Shear perimeter (22.6.4)		bo = 2 * (lx1 +	- dv2) + 2 * (ly1 +	+ d _{v2}) = 65.500	in	
Shear area		$A_p = I_{x,perim}$ *	y,perim = 268.14	1 in ²		
Surcharge loaded area		$A_{sur} = A_p - I_{x1}$	* $I_{y1} = 232.141$	in²		
Ultimate bearing pressure at cen	ter of shear area	q _{up.avg} = 1.25	8 ksf			
Ultimate shear load		$F_{up} = \gamma D * F_{D2}$	21 + γL * F Lz1 + γ	νD * Ap * Fswt + γ	D * Asur * Fsoil + γL	* Asur * FLsur
		$q_{up.avg} * A_p =$	4.703 kips			
Ultimate shear stress from vertica	al load	$v_{ug} = max(F_u)$	₀ / (b₀ * dv₂),0 p	osi) = 6.921 psi		
Column geometry factor (Table 2	2.6.5.2)	$\beta = I_{y1} / I_{x1} =$	1.00			
Column location factor (22.6.5.3)		αs =40				
Concrete shear strength (22.6.5.1	2)	Vcpa = (2 + 4	/β) *λ * √(f'c *	1 psi) = 379.47	3 psi	
		$v_{cpb} = (\alpha_s * d)$	ν2 / b₀ + 2) * λ *	^r √(f'c * 1 psi) =	527.207 psi	
		$v_{cpc} = 4 * \lambda *$	√(f'c * 1 psi) = 2	252.982 psi		
		$v_{cp} = min(v_{cp})$	a,Vcpb,Vcpc) = 25	2.982 psi		
Shear strength reduction factor		$\phi_v = 0.75$				
Nominal shear stress capacity (E	q. 22.6.1.2)	$v_n = v_{cp} = 25$	2.982 psi			
Design shear stress capacity (8.5			= 189.737 psi			
· · · ·		v _{ug} / φv _n = 0.	-			
				capacity exce	eds ultimate sh	near stress lo

Fast + Epp	Project Yaroslavsk	y Residence	Job Ref. 8119			
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section C1 Pad For	oting (F5)	Sheet no./rev. 7			
	Calc. by BJW	Date 2/25/2021	App'd by	Date		



Fast + Epp	Project Yaroslavsky Re	esidence	Job Ref. 8119			
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section 10" Cantilever Retaining Wall - Typical				Sheet no./rev. 1	
	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date

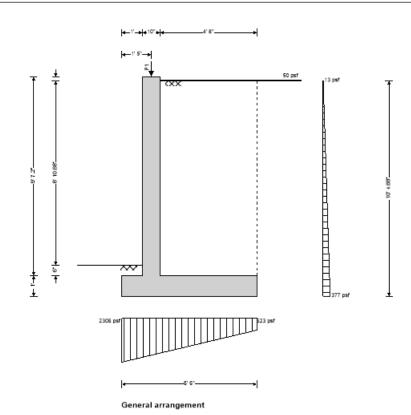
RETAINING WALL ANALYSIS

In accordance with International Building Code 2018

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	Ineel = 4.667 ft
Base thickness	t _{base} = 12 in
Base density	γbase = 150 pcf
Height of retained soil	h _{ret} = 8.89 ft
Angle of soil surface	$\beta = 0 \deg$
Depth of cover	d _{cover} = 0.5 ft
Retained soil properties	
Soil type	Medium dense well graded sand
Moist density	γmr = 135 pcf
Saturated density	γsr = 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	роь = 225 psf/ft
Allowable bearing pressure	Pbearing = 2500 psf
Loading details	
Live surcharge load	Surcharge∟ = 50 psf
Vertical line load at 1.417 ft	P _{D1} = 485 plf
	P _{L1} = 646 plf

Tedds calculation version 2.9.08

Tekla Tedds Fast + Epp	^{Project} Yaroslavsky Re	esidence	Job Ref. 8119			
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section 10" Cantilever I	Retaining Wall -	Typical		Sheet no./rev. 2	
	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date



Calculate retaining wall geometry

Base length

Moist soil height

Length of surcharge load

- Distance to vertical component

Effective height of wall

- Distance to horizontal component Area of wall stem

- Distance to vertical component Area of wall base

- Distance to vertical component

Area of moist soil

- Distance to vertical component

- Distance to horizontal component Area of base soil

- Distance to vertical component

- Distance to horizontal component

Area of excavated base soil

- Distance to vertical component

- Distance to horizontal component

Ibase = Itoe + tstem + Iheel = 6.5 ft $h_{moist} = h_{soil} = 9.39 \text{ ft}$ Isur = Iheel = 4.667 ft $x_{sur_v} = I_{base} - I_{heel} / 2 = 4.167$ ft heff = hbase + dcover + hret = 10.39 ft $x_{sur_h} = h_{eff} / 2 = 5.195 \text{ ft}$ Astem = hstem * $t_{stem} = 8 ft^2$ $x_{stem} = I_{toe} + t_{stem} / 2 = 1.417$ ft Abase = Ibase * tbase = 6.5 ft^2 xbase = Ibase / 2 = 3.25 ft Amoist = $h_{moist} * I_{heel} = 43.82 \text{ ft}^2$ $x_{moist_v} = I_{base} - (h_{moist} * I_{heel}^2 / 2) / A_{moist} = 4.167 \text{ ft}$ xmoist_h = heff / 3 = 3.463 ft Apass = dcover * Itoe = 0.5 ft^2 $x_{\text{pass}_v} = I_{\text{base}} - (d_{\text{cover}} * I_{\text{toe}} * (I_{\text{base}} - I_{\text{toe}} / 2)) / A_{\text{pass}} = 0.5 \text{ ft}$ $x_{pass_h} = (d_{cover} + h_{base}) / 3 = 0.5 ft$ Aexc = hpass * Itoe = 0.5 ft² $x_{exc_v} = I_{base} - (h_{pass} * I_{toe} * (I_{base} - I_{toe} / 2)) / A_{exc} = 0.5 \text{ ft}$ $x_{exc_h} = (h_{pass} + h_{base}) / 3 = 0.5 \text{ ft}$

Fast + Epp	Project Yaroslavsl	ky Residence			Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section 10" Cantile	ever Retaining Wal	l - Typical	Sheet no./rev 3	۷.	
	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date
Soil coefficients						
Coefficient of friction to back of w	all	K _{fr} = 0.300				
Coefficient of friction to front of w	all	K _{fb} = 0.300				
Coefficient of friction beneath bas	se	K _{fbb} = 0.350				
From IBC 2018 cl.1807.2.3 Safe	tv factor	ALSO CHECK	(0.7EQ W/ F.O.S.	= 1.1 FOR O	VERTURNING & SLIE	DING
Load combination 1	.,		• 1.0 * Live + 1.			
Sliding check						
-						
Vertical forces on wall		F 4	* 4000	14		
Wall stem			* γstem = 1200 p			
Wall base			* γ _{base} = 975 plf			
Line loads			0 * PL1 = 485 p			
Moist retained soil			ist * γmr = 5916	DIT		
Base soil		$F_{exc_v} = A_{exc}$	• •		0630 olf	
Total		Ftotal_v = Fsten	h + ⊢ base + ⊢ P_v	+ F moist_v + F	exc_v = 8638 plf	
Horizontal forces on wall			7H * heff = 488.			
Surcharge load		-	γmr * Surcharge		5 plf	
Moist retained soil		•	* h _{eff} ² / 2 = 188	•		
Total		Ftotal_h = Fsur_	h + Fmoist_h = 20	24 plf + Feq	1_h = 2513 plf	
Check stability against sliding						
Base soil resistance		-	(h _{pass} + h _{base}) ²	-		
Base friction			_v * Kfbb = 3023	-		
Resistance to sliding Factor of safety			+ Ffriction = 3277 / Ftotal_h = 1.619	. 0		> 11 OV
Factor of Salety		FUOSI = Frest			77 plf / 2513 plf = 1.3 safety against sli	
Overturning check						
Vertical forces on wall						
Wall stem		Fstem = Astem	* γ _{stem} = 1200 p	lf		
Wall base		Fbase = Abase	* γ _{base} = 975 plf			
Line loads		$F_{P_v} = P_{D1} +$	0 * P _{L1} = 485 p	lf		
Moist retained soil		Fmoist_v = Amo	ist * γmr = 5916	olf		
Base soil		$F_{exc_v} = A_{exc}$	* γь = 63 plf			
Total		F _{total_v} = F _{sten}	n + Fbase + FP_v	+ Fmoist_v + F	exc_v = 8638 plf	
Horizontal forces on wall						
Surcharge load		$F_{sur_h} = p_{Ar} / r$	γmr * Surcharge	∟ * h _{eff} = 135	5 plf	
Moist retained soil		-	* h _{eff} ² / 2 = 188			
Base soil		$F_{exc_h} = -p_{0b}$	* (h _{pass} + h _{base}) ²	/ 2 = -253 p	olf	
Total		Ftotal_h = Fsur_	h + Fmoist_h + Fe	xc_h = 1771 p	olf	
Overturning moments on wall		Mea OT = Fe	q h * heff/2 = 25	39 lb ft/ft		
Surcharge load			r_h * Xsur_h = 700			
Moist retained soil			moist_h * Xmoist_h =		t	
Total		Mtotal OT - Ma	ur OT + Mmoist OT	– 7242 lb f	t/ft + Meq OT = 978	R1 lb ft/ft

Fast + Epp	^{Project} Yaroslavsky Re	esidence	Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section 10" Cantilever F	Retaining Wall -	Sheet no./rev. 4	
	Calc. by BJW	App'd by	Date	

Restoring moments on wall

Wall stem Wall base Line loads Moist retained soil Base soil Total

Check stability against overturning

Factor of safety

Bearing pressure check

Vertical forces on wall

Wall stem Wall base Surcharge load Line loads Moist retained soil Base soil Total

Horizontal forces on wall

Surcharge load Moist retained soil Base soil Total

Moments on wall

Wall stem Wall base Surcharge load Line loads Moist retained soil Base soil Total

Check bearing pressure

Distance to reaction Eccentricity of reaction Loaded length of base Bearing pressure at toe Bearing pressure at heel Factor of safety

Mstem_R = Fstem * Xstem = 1700 lb_ft/ft
Mbase_R = Fbase * Xbase = 3169 lb_ft/ft
M _{P_R} = (abs(P _{D1} + 0 * P _{L1})) * p ₁ = 687 lb_ft/ft
$M_{moist_R} = F_{moist_v} * x_{moist_v} = 24649 \text{ lb_ft/ft}$
Mexc_R = Fexc_v * Xexc_v - Fexc_h * Xexc_h = 158 lb_ft/ft
$M_{total_R} = M_{stem_R} + M_{base_R} + M_{P_R} + M_{moist_R} + M_{exc_R} = 30363 \text{ Ib_ft/ft}$

CHECK WITH EQ:

$$\label{eq:FoSot} \begin{split} \text{FoS}_{\text{ot}} = M_{\text{total_R}} \ / \ M_{\text{total_OT}} = \textbf{4.192} \ > 1.5 \ \text{30363 lb}_{\text{ft/ft}} \ / \ 9781 \ \text{lb}_{\text{ft/ft}} = 3.1 > 1.1 \ \text{OK} \\ \text{PASS} \ - \ \text{Factor of safety against overturning is adequate} \end{split}$$

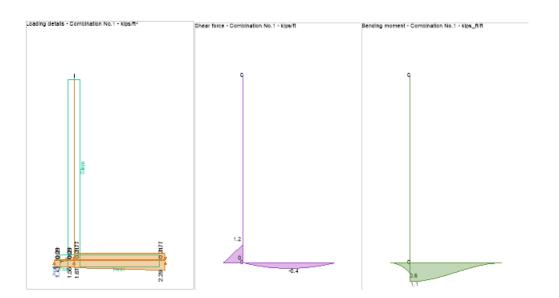
$$\begin{split} F_{stem} &= A_{stem} * \gamma_{stem} = 1200 \text{ plf} \\ F_{base} &= A_{base} * \gamma_{base} = 975 \text{ plf} \\ F_{sur_v} &= Surcharge_{L} * \text{ lneel} = 233 \text{ plf} \\ F_{P_v} &= P_{D1} + P_{L1} = 1131 \text{ plf} \\ F_{moist_v} &= A_{moist} * \gamma_{mr} = 5916 \text{ plf} \\ F_{pass_v} &= A_{pass} * \gamma_b = 63 \text{ plf} \\ F_{total_v} &= F_{stem} + F_{base} + F_{sur_v} + F_{P_v} + F_{moist_v} + F_{pass_v} = 9518 \text{ plf} \end{split}$$

$$\begin{split} F_{sur_h} &= p_{Ar} \ / \ \gamma_{mr} \ * \ Surcharge_L \ * \ h_{eff} = \textbf{135} \ plf \\ F_{moist_h} &= p_{Ar} \ * \ h_{eff^2} \ / \ 2 = \textbf{1889} \ plf \\ F_{pass_h} &= -p_{0b} \ * \ (d_{cover} + h_{base})^2 \ / \ 2 = \textbf{-253} \ plf \\ F_{total_h} &= max(F_{sur_h} + F_{moist_h} + F_{pass_h} - F_{total_v} \ * \ K_{fbb}, \ 0 \ plf) = \textbf{0} \ plf \end{split}$$

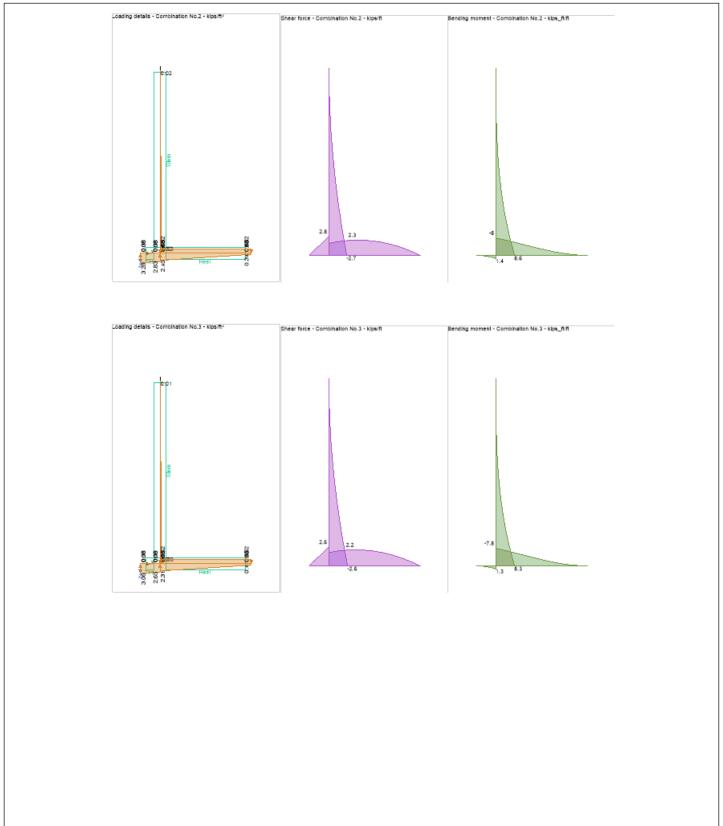
$$\begin{split} & \mathsf{M}_{stem} = \mathsf{F}_{stem} * x_{stem} = \mathbf{1700} \; \mathsf{lb}_{ft} / \mathsf{ft} \\ & \mathsf{M}_{base} = \mathsf{F}_{base} * x_{base} = \mathbf{3169} \; \mathsf{lb}_{ft} / \mathsf{ft} \\ & \mathsf{M}_{sur} = \mathsf{F}_{sur_v} * x_{sur_v} - \mathsf{F}_{sur_h} * x_{sur_h} = \mathbf{273} \; \mathsf{lb}_{ft} / \mathsf{ft} \\ & \mathsf{M}_{P} = \left((\mathsf{P}_{D1} + \mathsf{P}_{L1}) \right) * \mathsf{p}_{1} = \mathbf{1602} \; \mathsf{lb}_{ft} / \mathsf{ft} \\ & \mathsf{M}_{moist} = \mathsf{F}_{moist_v} * x_{moist_v} - \mathsf{F}_{moist_h} * x_{moist_h} = \mathbf{18106} \; \mathsf{lb}_{ft} / \mathsf{ft} \\ & \mathsf{M}_{pass} = \mathsf{F}_{pass_v} * x_{pass_v} - \mathsf{F}_{pass_h} * x_{pass_h} = \mathbf{158} \; \mathsf{lb}_{ft} / \mathsf{ft} \\ & \mathsf{M}_{total} = \mathsf{M}_{stem} + \mathsf{M}_{base} + \mathsf{M}_{sur} + \mathsf{MP} + \mathsf{M}_{moist} + \mathsf{M}_{pass} = \mathbf{25008} \; \mathsf{lb}_{ft} / \mathsf{ft} \\ \end{aligned}$$

 $\overline{x} = M_{total} / F_{total_v} = 2.628 \text{ ft}$ $e = \overline{x} - l_{base} / 2 = -0.622 \text{ ft}$ $l_{load} = l_{base} = 6.5 \text{ ft}$ $q_{toe} = F_{total_v} / l_{base} * (1 - 6 * e / l_{base}) = 2306 \text{ psf}$ $q_{heel} = F_{total_v} / l_{base} * (1 + 6 * e / l_{base}) = 623 \text{ psf}$ $FoS_{bp} = P_{bearing} / max(q_{toe}, q_{heel}) = 1.084$ PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

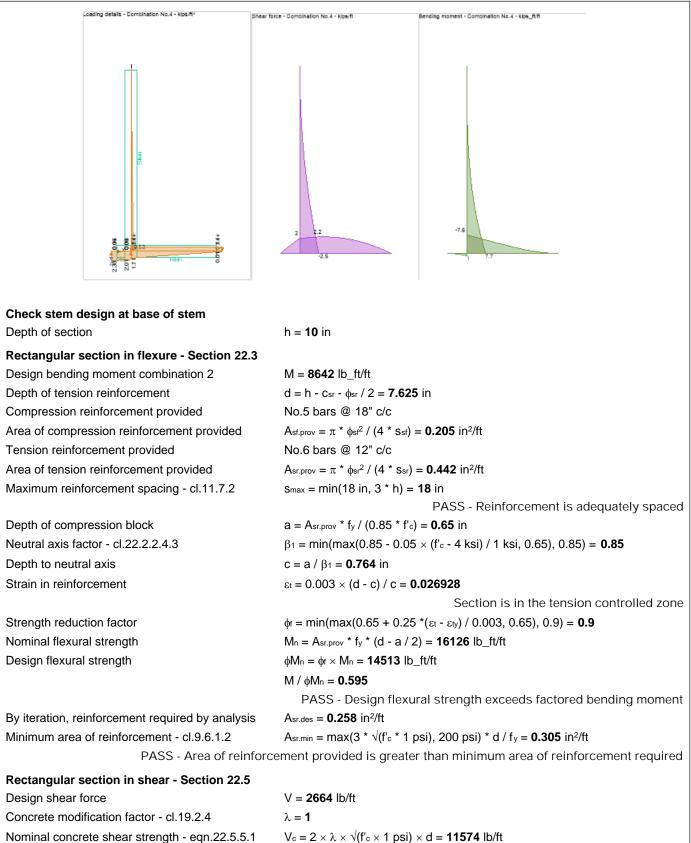
Fast + Epp	Project Yaroslavsk	y Residence			Job Ref. 8119	
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section 10" Cantile	ver Retaining Wal	Sheet no./rev. 5	Sheet no./rev. 5		
	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date
RETAINING WALL DESIGN						
In accordance with ACI 318-14						
					Tedds cald	culation version 2.9
Concrete details						
Compressive strength of concrete	e	f'c = 4000 psi				
Concrete type		Normal weig	ht			
Reinforcement details						
Yield strength of reinforcement		fy = 60000 ps	si			
Modulus of elasticity or reinforce	ment	Es = 290000	00 psi			
Compression-controlled strain lin	nit	εty = 0.002				
Cover to reinforcement						
Front face of stem		_{Csf} = 1.5 in				
Rear face of stem		_{Csr} = 2 in				
Top face of base		c _{bt} = 2 in				
Bottom face of base		сы = 3 іп				
	load combina	ations				
From IBC 2018 cl.1605.2 Basic						
From IBC 2018 cl.1605.2 Basic Load combination no.1		1.4 * Dead				
			1.6 * Live + 1.	.6 * Lateral ear	th	
Load combination no.1		1.2 * Dead +			th e + 1.6 * Lateral e	arth







Tekla Tedds	Project				Job Ref.	
Fast + Epp	Yaroslavsky Residence				8119	
323 Dean Street, Suite #3	Section				Sheet no./rev.	
Brooklyn, NY 11217	10" Cantilever Retaining Wall - Typical				7	
	,	Date 2/17/2021	Chk'd by	Date	App'd by	Date



Tekla Tedds	Project Yaroslavsky	Residence			Job Ref. 8119				
Fast + Epp	Section	-			Sheet no./rev.				
323 Dean Street, Suite #3 Brooklyn, NY 11217		er Retaining Wal	I - Typical		8				
• •	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date			
Strength reduction factor		$\phi_{s} = 0.75$							
Design concrete shear strength -	CI.11.5.1.1	$\phi V_c = \phi_s \times V_c$							
		$V / \phi V_c = 0.3$	07	PASS - No	shear reinforcer	ment is reau			
Horizontal reinforcement paral	lel to face of st	em							
Minimum area of reinforcement -	cl.11.6.1	Asx.req = 0.00	2 * t _{stem} = 0.24	in²/ft					
Transverse reinforcement provided		No.5 bars @	18" c/c each f	ace					
Area of transverse reinforcement	provided	$A_{sx.prov} = 2 * $	π * φsx² / (4 * Ss	() = 0.409 in²/ft	:				
	PASS - Area	of reinforceme	ent provided is	s greater than	area of reinford	ement requ			
Check base design at toe		h 40 in							
Depth of section		h = 12 in							
Rectangular section in flexure			a. 10.						
Design bending moment combine	ation 2	M = 1437 lb_							
Depth of tension reinforcement			фыь / 2 = 8.688 і	n					
Compression reinforcement prov		No.5 bars @ 12" c/c							
	Area of compression reinforcement provided		Abt.prov = $\pi * \phi_{bt}^2 / (4 * s_{bt}) = 0.307 \text{ in}^2/\text{ft}$						
Tension reinforcement provided		No.5 bars @ 12" c/c							
Area of tension reinforcement pro			$\phi_{bb^2} / (4 * S_{bb}) =$						
Maximum reinforcement spacing	- cl.7.7.2.3	smax = min(18 in, 3 * h) = 18 in PASS - Reinforcement is adequately space							
Depth of compression block		a - Abb prov. *	fy / (0.85 * f'c) =		orcement is ade	equatery spa			
Neutral axis factor - cl.22.2.2.4.3					ksi 0.65) 0.85) =	0.85			
Depth to neutral axis		β1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = 0.85 c = a / β1 = 0.531 in							
Strain in reinforcement		$\epsilon = 0.003 \times (d - c) / c = 0.046101$							
		a = 0.000 × 1	(a c) / c = c.c		s in the tension	controlled z			
Strength reduction factor		$\phi_f = \min(\max$	x(0.65 + 0.25 *(εt - εty) / 0.003,	0.65), 0.9) = 0.9				
Nominal flexural strength		$M_n = A_{bb.prov}$	* f _y * (d - a / 2)	= 12980 lb_ft/f	t				
Design flexural strength		$\phi M_n = \phi_f \times M_n = 11682 \text{ lb}_f t/ft$							
		$M / \phi M_n = 0.7$	123						
		PASS - D	esign flexura	l strength exc	eeds factored b	ending mon			
By iteration, reinforcement requir	ed by analysis	Abb.des = 0.03	87 in²/ft						
Minimum area of reinforcement -	cl.7.6.1.1	Abb.min = 0.0018 * h = 0.259 in ² /ft							
PASS -	Area of reinfor	cement provide	ed is greater t	han minimum	area of reinforc	ement requ			
Rectangular section in shear -	Section 22.5								
Design shear force		V = 2799 lb/t	ft						
Concrete modification factor - cl.	19.2.4	$\lambda = 1$							
Nominal concrete shear strength	- eqn.22.5.5.1	$V_c=2\times\lambda\times$	√(f'c × 1 psi) × d	d = 13187 lb/ft					
Strength reduction factor		$\phi_{\rm S}=0.75$							
Design concrete shear strength -	cl.7.6.3.1	$\phi V_c = \phi_s \times V_c$	= 9890 lb/ft						
		$V / \phi V_c = 0.2$	83						
				PASS - No	shear reinforcer	ment is requ			
Check base design at heel									
Depth of section		h = 12 in							

Tekla Tedds Fast + Epp	Project Yaroslavsky	Residence	Job Ref. 8119				
323 Dean Street, Suite #3 Brooklyn, NY 11217	Section 10" Cantilev	Section 10" Cantilever Retaining Wall - Typical				Sheet no./rev. 9	
	Calc. by BJW	Date 2/17/2021	Chk'd by	Date	App'd by	Date	

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} - φ _{bt} / 2 = 9.687 in
Compression reinforcement provided	No.5 bars @ 12" c/c
Area of compression reinforcement provided	Abb.prov = π * φbb ² / (4 * Sbb) = 0.307 in ² /ft
Tension reinforcement provided	No.5 bars @ 12" c/c
Area of tension reinforcement provided	$A_{bt,prov} = \pi * \phi_{bt}^2 / (4 * s_{bt}) = 0.307 \text{ in}^2/\text{ft}$
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	$\beta_1 = min(max(0.85 - 0.05 \times (f_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) = 0.85$
Depth to neutral axis	c = a / β1 = 0.531 in
Strain in reinforcement	$\epsilon_t = 0.003 \times (d - c) / c = 0.051753$
	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	Mn = A _{bt.prov} * f _y * (d - a / 2) = 14514 lb_ft/ft
Design flexural strength	$\phi M_n = \phi t \times M_n = 13063 \text{ lb}_ft/ft$
	M / ϕ Mn = 0.613
	PASS - Design flexural strength exceeds factored bending moment
By iteration, reinforcement required by analysis	A _{bt.des} = 0.186 in ² /ft
Minimum area of reinforcement - cl.7.6.1.1	A _{bt.min} = 0.0018 * h = 0.259 in ² /ft
PASS - Area of reinford	cement provided is greater than minimum area of reinforcement required
Rectangular section in shear - Section 22.5	
Design shear force	V = 2291 lb/ft
-	

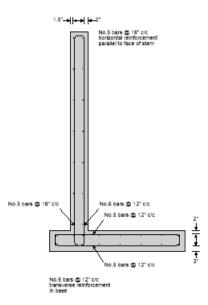
Design shear force Concrete modification factor - cl.19.2.4 Nominal concrete shear strength - eqn.22.5.5.1 Strength reduction factor Design concrete shear strength - cl.7.6.3.1

V = 2291 lb/ft	
$\lambda = 1$	
$V_c = 2 \times \lambda \times \sqrt{(f_c \times 1 \text{ psi}) \times d} = 14705 \text{ lb/ft}$	
$\phi_s = 0.75$	
$\varphi V_{\rm c} = \varphi_{\rm s} \times V_{\rm c} = \textbf{11028} \text{ lb/ft}$	
$V / \phi V_c = 0.208$	
PASS - No shear	reinforcement is required

Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.7.6.1.1	Abx.req = 0.0018 * tbase = 0.259 in ² /ft
Transverse reinforcement provided	No.5 bars @ 12" c/c each face
Area of transverse reinforcement provided	Abx.prov = 2 * π * ϕ bx ² / (4 * Sbx) = 0.614 in ² /ft
PASS - Area	a of reinforcement provided is greater than area of reinforcement required

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		Date 2/17/2021	Chk'd by	Date	App'd by	Date



Reinforcement details

Tekla Tedds Fast + Epp	^{Project} Yaroslavsky Re	esidence	Job Ref. 8119			
323 Dean Street, Suite #3	Section 8" Cantilever Retaining Wall - 6 ft Soil				Sheet no./rev. 1	
	Calc. by BJW	Date 3/3/2021	Chk'd by	Date	App'd by	Date

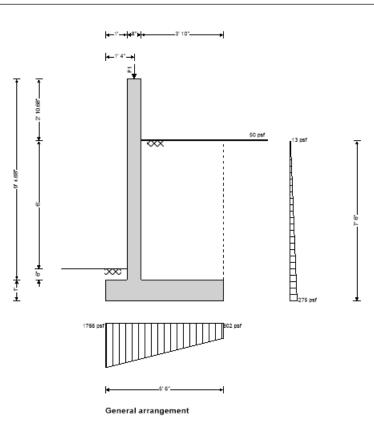
RETAINING WALL ANALYSIS

In accordance with International Building Code 2018

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	γstem = 150 pcf
Toe length	$I_{toe} = 1$ ft
Heel length	Ineel = 3.833 ft
Base thickness	t _{base} = 12 in
Base density	γbase = 150 pcf
Height of retained soil	h _{ret} = 6 ft
Angle of soil surface	$\beta = 0 \deg$
Depth of cover	d _{cover} = 0.5 ft
Retained soil properties	
Soil type	Medium dense well graded sand
Moist density	γmr = 135 pcf
Saturated density	γsr = 145 pcf
Prescribed active lateral soil pressure	p _{Ar} = 35 psf/ft
Base soil properties	
Soil type	Medium dense well graded sand
Soil density	γь = 125 pcf
Prescribed passive lateral soil pressure	роь = 1 psf/ft
Allowable bearing pressure	Pbearing = 2500 psf
Loading details	
Live surcharge load	Surcharge∟ = 50 psf
Vertical line load at 1.333 ft	P _{D1} = 485 plf
	PL1 = 646 plf

Tedds calculation version 2.9.08

Tekla Tedds Fast + Epp	Project Yaroslavsky Residence				Job Ref. 8119	
323 Dean Street, Suite #3	Section 8" Cantilever Retaining Wall - 6 ft Soil				Sheet no./rev. 2	
	Calc. by BJW	Date 3/3/2021	Chk'd by	Date	App'd by	Date



Calculate retaining wall geometry

Base length

Moist soil height

Length of surcharge load - Distance to vertical component

Effective height of wall

- Distance to horizontal component Area of wall stem

- Distance to vertical component Area of wall base

- Distance to vertical component

Area of moist soil

Distance to vertical componentDistance to horizontal component

Area of base soil

- Distance to vertical component

- Distance to horizontal component

Area of excavated base soil

- Distance to vertical component

- Distance to horizontal component

Ibase = Itoe + tstem + Iheel = 5.5 ft $h_{moist} = h_{soil} = 6.5 ft$ Isur = Iheel = 3.833 ft $x_{sur_v} = I_{base} - I_{heel} / 2 = 3.583 \text{ ft}$ $h_{eff} = h_{base} + d_{cover} + h_{ret} = 7.5 \text{ ft}$ $x_{sur_h} = h_{eff} / 2 = 3.75 \text{ ft}$ Astem = hstem * tstem = 6.26 ft² $x_{stem} = I_{toe} + t_{stem} / 2 = 1.333 \text{ ft}$ Abase = $|base * t_{base} = 5.5 \text{ ft}^2$ xbase = Ibase / 2 = 2.75 ft Amoist = hmoist * Iheel = 24.916 ft² xmoist_v = Ibase - (hmoist * Iheel² / 2) / Amoist = 3.583 ft $x_{moist_h} = h_{eff} / 3 = 2.5 ft$ Apass = dcover * Itoe = 0.5 ft^2 xpass_v = lbase - (dcover * ltoe* (lbase - ltoe / 2)) / Apass = 0.5 ft $x_{pass_h} = (d_{cover} + h_{base}) / 3 = 0.5 \text{ ft}$ Aexc = hpass * Itoe = 0.5 ft² $x_{exc_v} = I_{base} - (h_{pass} * I_{toe} * (I_{base} - I_{toe} / 2)) / A_{exc} = 0.5 \text{ ft}$ $x_{exc_h} = (h_{pass} + h_{base}) / 3 = 0.5 \text{ ft}$

Fast + Epp	Project Yaroslavsky Residence				Job Ref. 8119		
323 Dean Street, Suite #3 Section		ver Retaining Wall	Sheet no./rev 3	Sheet no./rev. 3			
	Calc. by BJW	Date 3/3/2021	Chk'd by	Date	App'd by	Date	
Soil coefficients							
Coefficient of friction to back of v	vall	K _{fr} = 0.300					
Coefficient of friction to front of w	vall	K _{fb} = 0.300					
Coefficient of friction beneath ba	se	K _{fbb} = 0.350					
From IBC 2018 cl.1807.2.3 Safe	ety factor	ALSO CHECH	< 0.7EQ W/ F.O.S.	= 1.1 FOR OV	ERTURNING & SLIE	NG	
Load combination 1	,	1.0 * Dead -	+ 1.0 * Live + 1.	0 * Lateral ea	rth		
Sliding check							
-							
Vertical forces on wall		– ,	* 000 //				
Wall stem			* γ _{stem} = 939 plf				
Wall base			* γ _{base} = 825 plf				
Line loads			0 * PL1 = 485 p				
Moist retained soil		$F_{moist_v} = A_{moist} * \gamma_{mr} = 3364 \text{ plf}$					
Base soil		$F_{exc_v} = A_{exc} * \gamma_b = 63 \text{ plf}$					
Total		F total_v = F ster	m + Fbase + FP_v	+ Fmoist_v + Fe>	_{c_v} = 5675 plf		
Horizontal forces on wall		$Feq_h = 0.7^{-1}$	* 7H * heff = 345.	0825 plf			
Surcharge load		$F_{sur_h} = p_{Ar} / \gamma_{mr} * Surcharge_ * h_{eff} = 97 plf$					
Moist retained soil		•	* h _{eff} ² / 2 = 984	•			
Total		Ftotal_h = Fsur	_h + Fmoist_h = 10	82 plf + Feq_	h = 1428 plf		
Check stability against sliding							
Base soil resistance		$F_{exc_h} = p_{0b}$	* (hpass + hbase) ²	/ 2 = 1 plf			
Base friction			u_v * Kfbb = 1986	-			
Resistance to sliding			+ Ffriction = 1987		ECK WITH EQ:		
Factor of safety		F0Ssl = Frest	/ Ftotal_h = 1.838 PASS		7 plf / 1428 plf = 1.3 afety against slie		
Overturning check					5 0	0	
Vertical forces on wall							
Wall stem		F _{stem} = A _{stem}	* γ _{stem} = 939 plf	:			
Wall base			* γ _{base} = 825 plf				
Line loads			0 * PL1 = 485 p				
Moist retained soil			oist * γmr = 3364				
Base soil		F _{exc_v} = A _{exc}	* γь = 63 plf				
Total		Ftotal_v = Fster	m + Fbase + FP_v	+ Fmoist_v + Fex	_{c_v} = 5675 plf		
Horizontal forces on wall							
Surcharge load		$F_{sur h} = n_{Ar}/$	γmr * Surcharge	u * heff = 97 n	f		
Moist retained soil		-	* h _{eff} ² / 2 = 984	-			
Base soil			* $(h_{pass} + h_{base})^2$	-			
Total		•	_h + Fmoist_h + Fe	-	f		
Overturning moments on wall				-			
Surcharge load			eq_h * heff/2 = 12 ɹr_h * Xsur_h = 365	-			
Moist retained soil							
Total	Mmoist_OT = Fmoist_h * xmoist_h = 2461 lb_ft/ft Mtotal_OT = Msur_OT + Mmoist_OT = 2826 lb_ft/ft + Meq_OT = 4121 lb_ft/ft						

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Restoring moments on wall

Wall stem Wall base Line loads Moist retained soil Base soil Total

Check stability against overturning

Factor of safety

Bearing pressure check

Vertical forces on wall

Wall stem Wall base Surcharge load Line loads Moist retained soil Base soil Total

Horizontal forces on wall

Surcharge load Moist retained soil Base soil Total

Moments on wall

Wall stem Wall base Surcharge load Line loads Moist retained soil Base soil Total

Check bearing pressure

Distance to reaction Eccentricity of reaction Loaded length of base Bearing pressure at toe Bearing pressure at heel Factor of safety

Mstem_R = Fstem * Xstem = 1252 lb_ft/ft
Mbase_R = Fbase * Xbase = 2269 lb_ft/ft
M _{P_R} = (abs(P _{D1} + 0 * P _{L1})) * p ₁ = 647 lb_ft/ft
Mmoist_R = Fmoist_v * xmoist_v = 12053 lb_ft/ft
Mexc_R = Fexc_v * Xexc_v - Fexc_h * Xexc_h = 32 lb_ft/ft
$M_{total_R} = M_{stem_R} + M_{base_R} + M_{P_R} + M_{moist_R} + M_{exc_R} = 16252 \text{ Ib_ft/ft}$

CHECK WITH EQ:

$$\label{eq:FoSot} \begin{split} \text{FoS}_{\text{ot}} = M_{\text{total_R}} \ / \ M_{\text{total_OT}} = \textbf{5.752} \ > 1.5 \ 16252 \ \text{lb_ft/ft} \ / \ 4121 \ \text{lb_ft/ft} = 3.9 > 1.1 \ \text{OK} \\ \text{PASS} \ - \ \text{Factor of safety against overturning is adequate} \end{split}$$

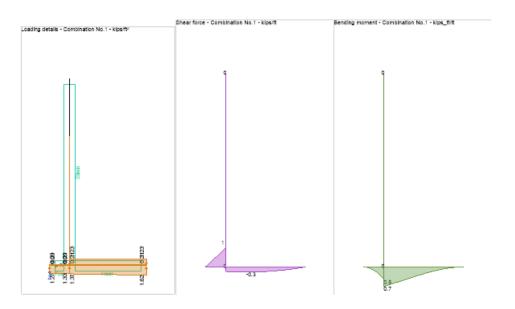
$$\begin{split} F_{stem} &= A_{stem} * \gamma_{stem} = \textbf{939} \text{ plf} \\ F_{base} &= A_{base} * \gamma_{base} = \textbf{825} \text{ plf} \\ F_{sur_v} &= Surcharge_{L} * I_{heel} = \textbf{192} \text{ plf} \\ F_{P_v} &= P_{D1} + P_{L1} = \textbf{1131} \text{ plf} \\ F_{moist_v} &= A_{moist} * \gamma_{mr} = \textbf{3364} \text{ plf} \\ F_{pass_v} &= A_{pass} * \gamma_b = \textbf{63} \text{ plf} \\ F_{total_v} &= F_{stem} + F_{base} + F_{sur_v} + F_{P_v} + F_{moist_v} + F_{pass_v} = \textbf{6513} \text{ plf} \end{split}$$

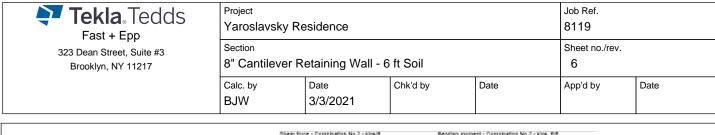
$$\begin{split} F_{sur_h} &= p_{Ar} \ / \ \gamma_{mr} \ * \ Surcharge_L \ * \ h_{eff} = \textbf{97} \ plf \\ F_{moist_h} &= p_{Ar} \ * \ h_{eff}^2 \ / \ 2 = \textbf{984} \ plf \\ F_{pass_h} &= -p_{0b} \ * \ (d_{cover} + h_{base})^2 \ / \ 2 = \textbf{-1} \ plf \\ F_{total_h} &= max(F_{sur_h} + F_{moist_h} + F_{pass_h} - F_{total_v} \ * \ K_{fbb}, \ 0 \ plf) = \textbf{0} \ plf \end{split}$$

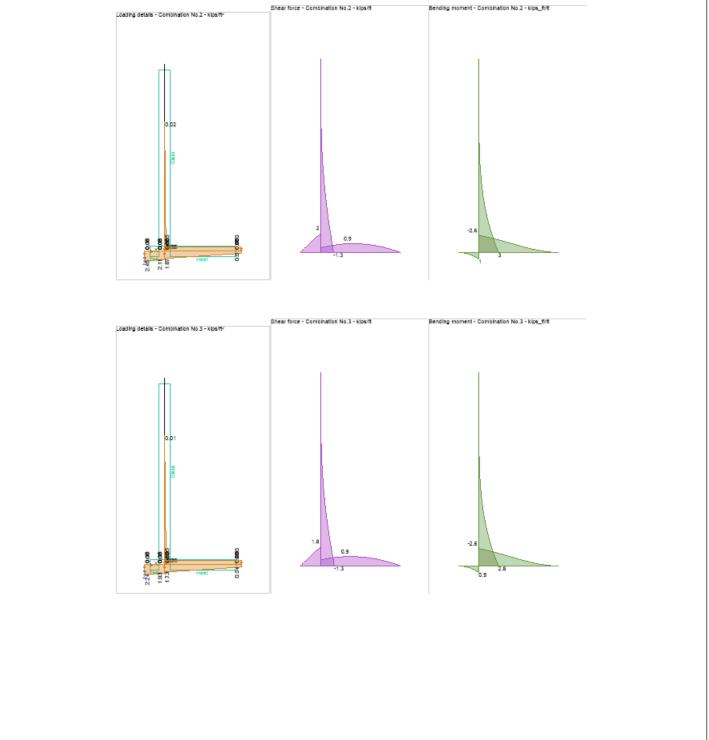
$$\begin{split} M_{stem} &= F_{stem} * x_{stem} = 1252 \ lb_ft/ft \\ M_{base} &= F_{base} * x_{base} = 2269 \ lb_ft/ft \\ M_{sur} &= F_{sur_v} * x_{sur_v} - F_{sur_h} * x_{sur_h} = 322 \ lb_ft/ft \\ M_P &= ((P_{D1} + P_{L1})) * p_1 = 1508 \ lb_ft/ft \\ M_{moist} &= F_{moist_v} * x_{moist_v} - F_{moist_h} * x_{moist_h} = 9592 \ lb_ft/ft \\ M_{pass} &= F_{pass_v} * x_{pass_v} - F_{pass_h} * x_{pass_h} = 32 \ lb_ft/ft \\ M_{total} &= M_{stem} + M_{base} + M_{sur} + M_P + M_{moist} + M_{pass} = 14975 \ lb_ft/ft \end{split}$$

 $\overline{x} = M_{total} / F_{total_v} = 2.299 \text{ ft}$ $e = \overline{x} - l_{base} / 2 = -0.451 \text{ ft}$ $l_{load} = l_{base} = 5.5 \text{ ft}$ $q_{toe} = F_{total_v} / l_{base} * (1 - 6 * e / l_{base}) = 1766 \text{ psf}$ $q_{heel} = F_{total_v} / l_{base} * (1 + 6 * e / l_{base}) = 602 \text{ psf}$ $FoS_{bp} = P_{bearing} / max(q_{toe}, q_{heel}) = 1.415$ PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

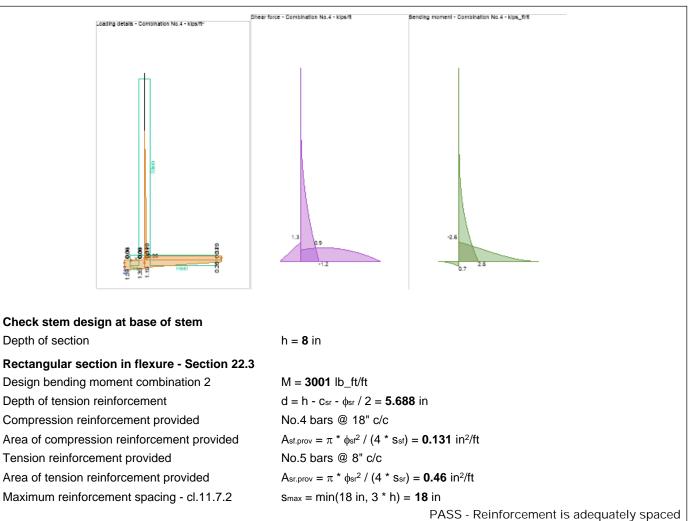
Fast + Epp	y Residence		Job Ref. 8119					
asi + ⊑pp 323 Dean Street, Suite #3	Section				Sheet no./rev.			
Brooklyn, NY 11217	8" Cantilev	er Retaining Wall	- 6 ft Soil		5			
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RETAINING WALL DESIGN								
In accordance with ACI 318-14					Tadda ad	culation version 2.9.		
Concrete details					Tedas calo	culation version 2.9.		
	0	f'c = 4000 ps						
Compressive strength of concrete Concrete type		Normal weight						
Reinforcement details			,					
Yield strength of reinforcement		fy = 60000 psi						
Modulus of elasticity or reinforcement		E _s = 2900000 psi						
Compression-controlled strain limit		$\epsilon_{ty} = 0.002$						
Cover to reinforcement								
Front face of stem		_{Csf} = 1.5 in						
Rear face of stem		c _{sr} = 2 in						
Top face of base		_{Cbt} = 2 in						
Bottom face of base		Cbb = 3 in						
From IBC 2018 cl.1605.2 Basic	load combina	ations						
Load combination no.1		1.4 * Dead						
Load combination no.2		1.2 * Dead + 1.6 * Live + 1.6 * Lateral earth						
Load combination no.2								
Load combination no.2 Load combination no.3		1.2 * Dead +	- 1.0 * Earthqua	ake + 1.0 * Live	e + 1.6 * Lateral e	arth		







323 Dean Street, Suite #3 Brooklyn, NY 11217 Section Sheet no./rev. 2 Cantilever Retaining Wall - 6 ft Soil 7 Calc. by Date Chk'd by Date BJW 3/3/2021 Chk'd by Date	Tekla Tedds Fast + Epp	Project J Yaroslavsky Residence 8					
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Depth of compression block Neutral axis factor - cl.22.2.2.4.3 Depth to neutral axis Strain in reinforcement

Strength reduction factor Nominal flexural strength Design flexural strength

$$\begin{split} M_n &= A_{sr.prov} \, * \, f_y \, * \, (d - a \ / \ 2) = \textbf{12308} \ lb_ft \ / ft \\ \varphi M_n &= \varphi_f \times M_n = \textbf{11077} \ lb_ft \ / ft \\ M \ / \ \varphi M_n = \textbf{0.271} \end{split}$$

 $a = A_{sr.prov} * f_y / (0.85 * f'_c) = 0.677$ in

 $\epsilon_t = 0.003 \times (d - c) / c = 0.01843$

 $c = a / \beta_1 = 0.796$ in

Asr.des = 0.119 in²/ft

PASS - Design flexural strength exceeds factored bending moment

Section is in the tension controlled zone

 $\beta_1 = min(max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) = 0.85$

 $\phi_f = \min(\max(0.65 + 0.25^{*}(\varepsilon_t - \varepsilon_{ty}) / 0.003, 0.65), 0.9) = 0.9$

By iteration, reinforcement required by analysis Minimum area of reinforcement - cl.9.6.1.3

Asr.mod = 4 * Asr.des / 3 = 0.159 in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

V = 1318 lb/ft
$\lambda = 1$
$V_c = 2 \times \lambda \times \sqrt{(f_c \times 1 \text{ psi})} \times d = 8633 \text{ lb/ft}$

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Calc. by		Date 3/3/2021	Chk'd by	Date	App'd by	Date			
Strength reduction factor		φs = 0.75							
Design concrete shear strength -	cl.11.5.1.1	-	c = 6475 lb/ft						
5		V / φVc = 0.2							
		.,		PASS - No	shear reinforcer	nent is reau			
Horizontal reinforcement para	lel to face of st	em							
Minimum area of reinforcement -)2 * t _{stem} = 0.19 2	2 in²/ft					
Transverse reinforcement provid			2 18" c/c each fa						
Area of transverse reinforcement			π * φsx ² / (4 * Ss		t				
	-			-	n area of reinford	ement requ			
Check base design at too				J					
Check base design at toe Depth of section		h = 12 in							
-		11 – 12 111							
Rectangular section in flexure			e. 10.						
Design bending moment combin	ation 2		M = 1045 lb_ft/ft						
Depth of tension reinforcement			$d = h - c_{bb} - \phi_{bb} / 2 = 8.688$ in						
Compression reinforcement prov		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 No.5 bars @ 8" c/c							
Tension reinforcement provided				0.40 := 2/4					
Area of tension reinforcement provided			Abb.prov = $\pi * \phi_{bb}^2 / (4 * s_{bb}) = 0.46 \text{ in}^2/\text{ft}$ smax = min(18 in, 3 * h) = 18 in						
Maximum reinforcement spacing - cl.7.7.2.3		Smax = MIN(1)	$8 \text{ in, } 3^{\circ} \text{ n} = 18$		forcomant is add	austoly one			
Depth of compression block		o – Au	fy / (0.85 * f'c) =		forcement is ade	equatery spa			
Neutral axis factor - cl.22.2.2.4.3					kei 0.65) 0.85) -	0.85			
		$\beta_1 = min(max(0.85 - 0.05 \times (f_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) = 0.85$ c = a / $\beta_1 = 0.796$ in							
Strain in reinforcement	Depth to neutral axis			20724					
Strain in reinforcement		$\epsilon t = 0.003 \times$	(d - c) / c = 0.02		s in the tension	controlled a			
Strength reduction factor		$d_{t} = \min(m_{t})$	V(0 65 1 0 25 */		(0.65), (0.9) = 0.9				
Nominal flexural strength									
Design flexural strength			$M_n = A_{bb,prov} * f_y * (d - a / 2) = 19211 \ lb_ft/ft$ $\phi M_n = \phi_f \times M_n = 17290 \ lb_ft/ft$						
Deelgi nextra etterigin			$\phi_{IVIn} = \phi_{I} \times IVIn = 17290 \text{ ID}_{IVII}$ M / $\phi_{Mn} = 0.060$						
				l strenath exc	eeds factored b	endina mon			
By iteration, reinforcement requir	ed by analysis		PASS - Design flexural strength exceeds factored bending mome Abb.des = 0.027 in ² /ft						
Minimum area of reinforcement - cl.7.6.1.1		Abb.min = $0.0018 * h = 0.259 in^2/ft$							
					n area of reinford	ement requ			
Rectangular section in shear -			0			1			
Design shear force		V = 2031 lb/	′ft						
Concrete modification factor - cl.19.2.4		$\lambda = 1$							
Nominal concrete shear strength - eqn.22.5.5.1		$V_c = 2 \times \lambda \times \sqrt{(f_c \times 1 \text{ psi})} \times d = 13187 \text{ lb/ft}$							
Strength reduction factor	$\phi_{\rm s} = 0.75$								
Design concrete shear strength - cl.7.6.3.1		$\phi_s = 0.75$ $\phi V_c = \phi_s \times V_c = 9890 \text{ lb/ft}$							
200gh controlo anear arenyth.	0.11.0.0.1	$\psi v_c = \psi s \times v$ V / $\phi V_c = 0.2$							
		ν / ψνc - U.		PASS - No	shear reinforcer	ment is reau			
Check hear dealers of the l				17.00-100		iloni ilo roqu			
Check base design at heel		h 40 in							
Depth of section		h = 12 in							

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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	Abb.prov = $\pi * \phi bb^2 / (4 * sbb) = 0.46 in^2/ft$
Tension reinforcement provided	No.5 bars @ 8" c/c
Area of tension reinforcement provided	Abt.prov = $\pi * \phi_{bt}^2 / (4 * s_{bt}) = 0.46$ 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 = Abt.prov * fy / (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	$\epsilon_t = 0.003 \times (d - c) / c = 0.033502$
	Section is in the tension controlled zone
Strength reduction factor	$\phi_{\rm f} = \min(\max(0.65 + 0.25 * (\varepsilon_{\rm t} - \varepsilon_{\rm ty}) / 0.003, 0.65), 0.9) = 0.9$
Nominal flexural strength	$M_n = A_{bt,prov} * f_y * (d - a / 2) = 21512 lb_ft/ft$
Design flexural strength	$\phi M_n = \phi_f \times M_n = 19361 \text{ lb_ft/ft}$
Design nextral strength	$M / \phi M_n = 0.136$
	PASS - Design flexural strength exceeds factored bending moment
By iteration, reinforcement required by analysis	Abudes = 0.06 in ² /ft
Minimum area of reinforcement - cl.7.6.1.1	$A_{bt.min} = 0.0018 * h = 0.259 in^2/ft$
	cement provided is greater than minimum area of reinforcement required
Rectangular section in shear - Section 22.5	
Design shear force	V = 927 lb/ft

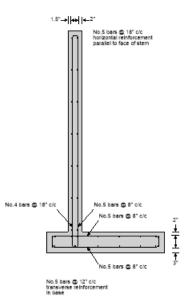
Design shear force
Concrete modification factor - cl.19.2.4
Nominal concrete shear strength - eqn.22.5.5.1
Strength reduction factor
Design concrete shear strength - cl.7.6.3.1

V = 927 lb/ft
$\lambda = 1$
$V_c = 2 \times \lambda \times \sqrt{(f_c \times 1 \text{ psi}) \times d} = 14705 \text{ lb/ft}$
$\phi_{\rm S} = 0.75$
$\phi V_c = \phi_s \times V_c = 11028 \text{ lb/ft}$
V / φVc = 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 in ² /ft
Transverse reinforcement provided	No.5 bars @ 12" c/c each face
Area of transverse reinforcement provided	Abx.prov = $2 * \pi * \phi_{bx^2} / (4 * s_{bx}) = 0.614 \text{ in}^2/\text{ft}$
PASS - Area	of reinforcement provided is greater than area of reinforcement required

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Reinforcement details