1152 Bond Ave Suite B
Rexburg, ID 83440
www.ridgestructural.com

STRUCTURAL CALCULATIONS

Project: ITD Preston Maintenance Building

Client: Myers-Anderson

Project No.: 24-268

Date: November 26, 2024

Engineer : KBB

SEAL:

PROFESSIONAL ENGINEER
LICENSED
14134
STATE OF IDAHO
DAVID B. PORTER

11/26/2024

BASIS FOR DESIGN

Design Gravity Loads

Design Code: 2018 International Building Code

Risk Category = II *[see ASCE 7 Table 1.5-1]*

Roof Loads:

single-ply membrane	1.5				
1.5" B steel deck	2.2				
rigid insulation	1.2				
steel studs at 16" o.c.	1.0				
acoustical tile (metal suspension)	1.5				
mechanical/miscellaneous	7.6		*deflection limits*		
DL =	**15.0**	**psf**	*LL joist*	*LL beam*	*TL*
Roof Snow Load, SL =	**40**	**psf**	*L/360*	*L/360*	*L/240*
Roof Live Load, Lr =	**20**	**psf**	*L/360*	*L/360*	*L/240*

Snow Loads: per ASCE 7 Chapter 7

Importance Factor Is =	1.0	*Importance Factor*
Pg =	53	*psf (ground snow load)*
Ce =	0.9	*exposure Factor*
Ct =	1.1	*thermal factor*
Snow, Pf = 0.70CeCt(Is)Pg =	**36.7**	**psf** (FLAT Roof)

Floor Loads:

3" concrete o/3" steel deck	63.0				
steel beams	added in calculations				
acoustical tile (metal suspension)	1.5				
mechanical/miscellaneous	5.5		*deflection limits*		
DL =	**70.0**	**psf**	*LL joist*	*LL beam*	*TL*
LL (heavy storage) **=**	**250**	**psf**	*L/360*	*L/360*	*L/240*

Exterior Walls: Metal Studs

EIFS	3.0		
1/2" plywood or OSB	1.7		
steel studs at 16" o.c.	0.9		
6" batt insulation (R-25)	1.5		
5/8" gypboard	2.8		
miscellaneous	5.1		*deflection limits*
DL =	**15.0**	**psf**	*L/240*

Exterior Walls: Masonry

8" CMU (grouted 24" o.c.)	58		
steel studs at 16" o.c.	0.9		
5/8" gypboard	2.8		
miscellaneous	3.3		*deflection limits*
DL =	**65.0**	**psf**	*L/240*

Interior Walls:

5/8" gypboard	2.8		
1/2" plywood or OSB (where occurs)	1.7		
steel studs at 16" o.c.	0.9		
5/8" gypboard	2.8		
miscellaneous	1.8		*deflection limits*
DL =	**10.0**	**psf**	*L/240*

Design Lateral Loads

Design Code: 2018 International Building Code
Risk Category = II *[see ASCE 7 Table 1.5-1]*

Seismic Loads:

Analysis Procedure: .... Equivalent Lateral Force Procedure
Importance Factor 1.00
Site Class .... D
Mapped Spectral Response Accelerations:
S_1 .... 0.293 g
S_S .... 0.900 g
Design Spectral Response Accelerations:
S_{D1} .... 0.393 g
S_{DS} .... 0.684 g
Seismic Design Category .... D
Seismic Force-Resisting System(s):
Lateral System .... Light-frame (steel) walls sheathed with wood structural panels rated
R 6.5
Ω_o 3
C_d 4
C_s 0.105

Wind Loads:

Design Wind Speed (ultimate) .... 115 MPH
Wind Exposure C
Enclosure Classification .... Enclosed
Internal Pressure Coefficient 0.18

Foundation Design

Design Code: 2018 International Building Code

Risk Category = II *[see ASCE 7 Table 1.5-1]*

Foundation Design:

Foundation Type .... Conventional Spread Footings

Design Basis:

Source .... Geotechnical Report

Firm .... Atlas Technical Consultants

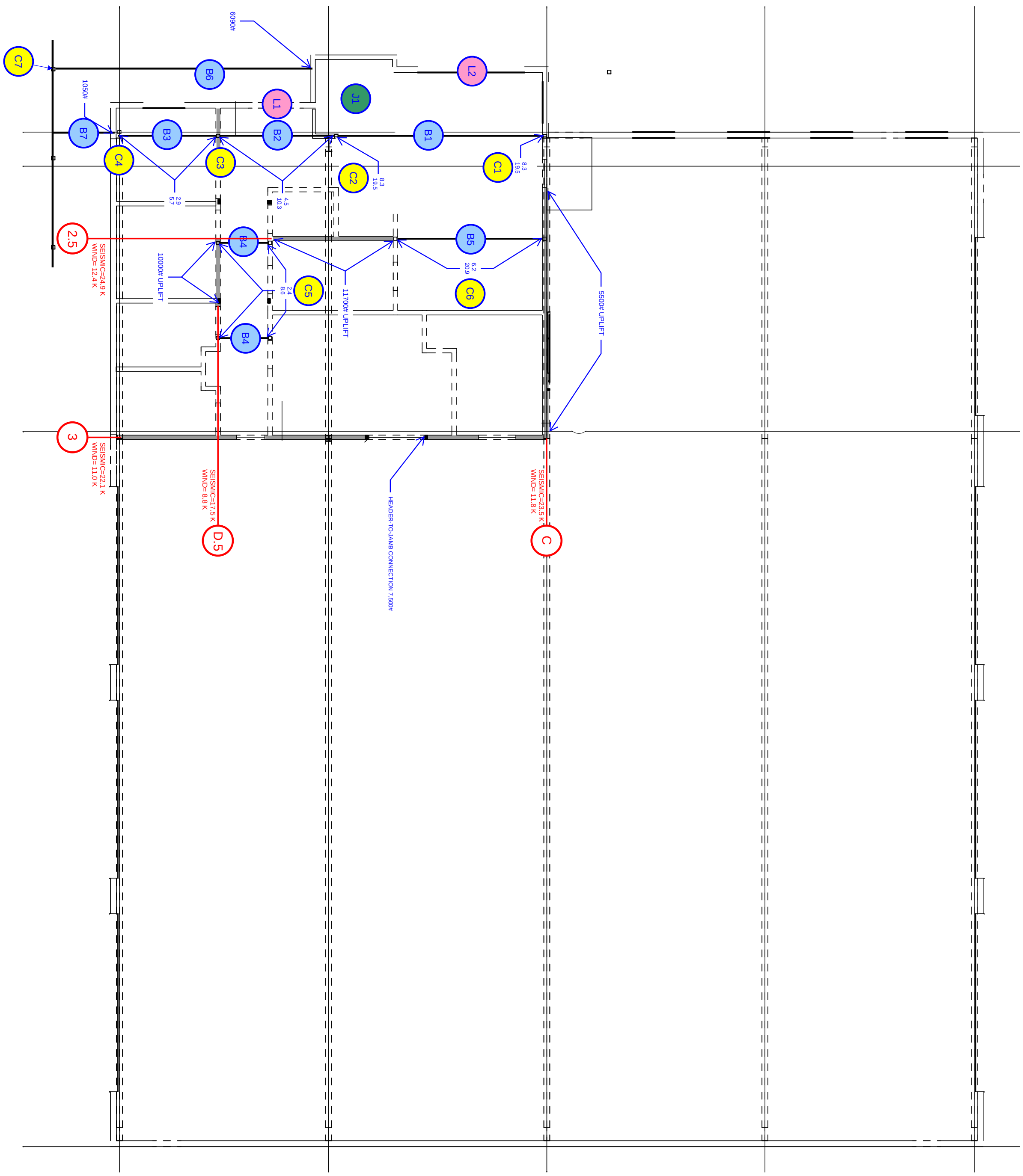
Project No P241629g

Date .... October 28, 2024

Allowable Bearing Pressure...........	1500	psf
Passive Pressure...........	302	pcf
Sliding Coefficient...........	0.35	
Frost Depth...........	30	inches

GRAVITY DESIGN

FRAMING KEY PLAN



Masonry Lintels

Working Stress Design

Mark: L1

Lintel Data:

Span (ft) =	**6.50**			**Special Inspection? (Y/N)**	**Y**		
width (b) =	8	inches		**Horizontal Reinf .:**	**1**	**#4**	
grout depth =	16	inches		**Vertical Reinf.:**	**#5**	**32" o.c.**	
flexural depth =	12	inches		**Simply supported? (Y/N)**	**Y**		
Horizontal Rebar Fy =	60	ksi					
Vertical Rebar Fy =	60	ksi	np = 0.0346				
Masonry f'm =	2000	ksi	k = 0.2306		ΔTL =	0.01	inches
Em =	1800	ksi	j = 0.9231		L /	8830	< L/600
n =	16.1		2/jk = 9.39				

Loads:	DL	LL	tributary	w				
Roof or Floor	15	55	4	280		V(DL) =	1399	lb
Self Weight	78	0	4.75	370.5		V(LL) =	715	lb
misc	0	0	0	0		V(TL) =	2114	lb
			w (uniform load) =	650.5	plf	M =	3435	lb-ft

Stresses:

fb =	336	psi	<	Fb =	666.66667	psi	OK
fs =	18954	psi	<	Fs =	24000	psi	OK
(Unreinforced) fv =	24	psi	<	Fv =	45	psi	OK

Mark: L2

Lintel Data:

Span (ft) =	**12.00**			**Special Inspection? (Y/N)**	**Y**		
width (b) =	8	inches		**Horizontal Reinf .:**	**2**	**#4**	
grout depth =	24	inches		**Vertical Reinf.:**	**#5**	**32" o.c.**	
flexural depth =	20	inches		**Simply supported? (Y/N)**	**Y**		
Horizontal Rebar Fy =	60	ksi					
Vertical Rebar Fy =	60	ksi	np = 0.0415				
Masonry f'm =	2000	ksi	k = 0.2495		ΔTL =	0.11	inches
Em =	1800	ksi	j = 0.9168		L /	1271	< L/600
n =	16.1		2/jk = 8.74				

Loads:	DL	LL	tributary	w				
Roof or Floor	15	55	4	280		V(DL) =	3402	lb
Self Weight	78	0	6.5	507		V(LL) =	1320	lb
misc	0	0	0	0		V(TL) =	4722	lb
			w (uniform load) =	787	plf	M =	14166	lb-ft

Stresses:

fb =	464	psi	<	Fb =	666.66667	psi	OK
fs =	23608	psi	<	Fs =	24000	psi	OK
(Unreinforced) fv =	32	psi	<	Fv =	45	psi	OK

Wood Joist

ASD design per NDS 2015

Mark: J1

Span (ft) =	8.3	l_u (ft) =	2.0		DL(psf)	LL(psf)	LL w(plf)	TL w(plf)	
Spacing (in) =	16.0	C_D =	1.00	*load*	15	55	73	93	
Δ_{LL} < L /	360	C_r =	1.15	*misc.*	0	0	0	0	
Δ_{TOTAL} < L /	240						73 plf	93 plf	

Joist Bearing Length (in) =	1.75
Web Stiffener? (Y/N) :	N

	DL(lbs)	LL(lbs)	x(ft)
Pt. Load	0	0	0
Pt. Load	0	0	0

R_{DL} = 84 lbs
R_{LL} = 308 lbs
R_{TL} (left)= **392 lbs**

R_{DL} = 84 lbs
R_{LL} = 308 lbs
R_{TL} (right) = **392 lbs**

M_{max} = **822 ft-lbs**

	A		B		C		D		E		F	
	(1) 2x8		11 7/8" TJI 210		11 7/8" BCI 5000 1.7		11 7/8" LPI 18		11 7/8" Red-I45		11 7/8" RFPI 20	
	Douglas Fir #2		*Trus Joist*		*Boise Cascade*		*Louisiana-Pacific*		*RedBuilt*		*Roseburg*	
$V_{all.}$ (lbs) =	*1501*	*26%*	*1655*	*24%*	*1625*	*24%*	*1335*	*29%*	*1785*	*22%*	*1420*	*28%*
$R_{all.}$ (lbs) =	*1641*	*24%*	*1005*	*39%*	*950*	*41%*	*870*	*45%*	*1015*	*39%*	*950*	*41%*
$M_{all.}$ (ft-lbs) =	*1360*	*60%*	*3795*	*22%*	*3150*	*26%*	*3100*	*27%*	*4685*	*18%*	*3640*	*23%*
Δ_{LL}	*0.100*	*L / 987*	*0.035*	*L / 1500+*	*0.039*	*L / 1500+*	*0.042*	*L / 1500+*	*0.035*	*L / 1500+*	*0.037*	*L / 1500+*
Δ_{TOTAL}	*0.128*	*L / 776*	*0.045*	*L / 1500+*	*0.049*	*L / 1500+*	*0.054*	*L / 1500+*	*0.045*	*L / 1500+*	*0.047*	*L / 1500+*
Nat. freq., f (Hz)	*21.3*		*43.3*		*39.7*		*38.4*		*43.6*		*41.0*	
	Adequate		**Adequate**		**Adequate**		**Adequate**		**Adequate**		**Adequate**	

Selection (A - F): **A** — **J1 Use: (1) 2x8 at 16" o.c. Douglas Fir #2**

Steel Beams

AISC-Allowable Stress Design (ASD)

Mark: B1

			DL(psf)	LL(psf)	trib(ft)	LL w(plf)	TL w(plf)		
Span (ft) =	23.5	roof	15	55	4	220	280	RDL =	8.3 k
L_b (ft) =	2.0	floor	70	250	5.75	1438	1840	RLL =	19.5 k
LL Deflection < L /	360	wall	15	0	13.5	0	203	RTL (left)=	**27.8 k**
Total Deflection < L /	240	misc.	0	0	0	0	0	RDL =	8.3 k
Fy (ksi) =	**50**					1658 plf	2323 plf	RLL =	19.5 k
C_b =	1.00					beam weight =	40.0 plf	RTL (right) =	**27.8 k**
Roof Snow Load ? :	**Yes**		DL(k)	LL(k)	x(ft)				
		Pt. Load	0.0	0.0	0			Mmax =	**163.1 k-ft**
Reduce Floor LL ? :	**No**	Pt. Load	0.0	0.0	0				

V_n/Ω_V =	*112.8 k*	*25%*	**OK**
M_n/Ω_b =	*195.6 k-ft*	*83%*	**OK**
LL deflection =	*0.64 in*	*82%*	**OK**
TL deflection =	*0.91 in*	*78%*	**OK**

B1 Use: W18X40 — *camber = 0.27 in*

Mark: B2

			DL(psf)	LL(psf)	trib(ft)	LL w(plf)	TL w(plf)		
Span (ft) =	13.3	roof	15	55	1.5	83	105	RDL =	4.5 k
L_b (ft) =	2.0	floor	70	250	5.9	1475	1888	RLL =	10.3 k
LL Deflection < L /	360	wall	15	0	13.5	0	203	RTL (left)=	**14.8 k**
Total Deflection < L /	240	misc.	0	0	0	0	0	RDL =	4.5 k
Fy (ksi) =	**50**					1558 plf	2196 plf	RLL =	10.3 k
C_b =	1.00					beam weight =	40.0 plf	RTL (right) =	**14.8 k**
Roof Snow Load ? :	**Yes**		DL(k)	LL(k)	x(ft)				
		Pt. Load	0.0	0.0	0			Mmax =	**49.1 k-ft**
Reduce Floor LL ? :	**No**	Pt. Load	0.0	0.0	0				

V_n/Ω_V =	*112.8 k*	*13%*	**OK**
M_n/Ω_b =	*195.6 k-ft*	*25%*	**OK**
LL deflection =	*0.06 in*	*14%*	**OK**
TL deflection =	*0.09 in*	*13%*	**OK**

B2 Use: W18X40 — *camber = 0.03 in*

Mark: B3

			DL(psf)	LL(psf)	trib(ft)	LL w(plf)	TL w(plf)		
Span (ft) =	11.1	roof	15	55	1.5	83	105	RDL =	2.9 k
L_b (ft) =	2.0	floor	70	250	3.8	950	1216	RLL =	5.7 k
LL Deflection < L /	360	wall	15	0	13.5	0	203	RTL (left)=	**8.7 k**
Total Deflection < L /	240	misc.	0	0	0	0	0	RDL =	2.9 k
Fy (ksi) =	**50**					1033 plf	1524 plf	RLL =	5.7 k
C_b =	1.00					beam weight =	40.0 plf	RTL (right) =	**8.7 k**
Roof Snow Load ? :	**Yes**		DL(k)	LL(k)	x(ft)				
		Pt. Load	0.0	0.0	0			Mmax =	**24.1 k-ft**
Reduce Floor LL ? :	**No**	Pt. Load	0.0	0.0	0				

V_n/Ω_V =	*112.8 k*	*8%*	**OK**
M_n/Ω_b =	*195.6 k-ft*	*12%*	**OK**
LL deflection =	*0.02 in*	*5%*	**OK**
TL deflection =	*0.03 in*	*5%*	**OK**

B3 Use: W18X40 — *camber = 0.01 in*

Steel Beams

AISC-Allowable Stress Design (ASD)

Mark: B4

Span (ft) =	6.0		DL(psf)	LL(psf)	trib(ft)	LL w(plf)	TL w(plf)	
L_b (ft) =	2.0	roof	15	55	0	0	0	
LL Deflection < L /	360	floor	70	250	11.4	2850	3648	
Total Deflection < L /	240	wall	15	0	0	0	0	
Fy (ksi) =	**50**	misc.	0	0	0	0	0	
C_b =	1.00					2850 plf	3648 plf	
Roof Snow Load ? :	**Yes**					beam weight =	15.0 plf	
			DL(k)	LL(k)	x(ft)			
		Pt. Load	0.0	0.0	0			
Reduce Floor LL ? :	**No**	Pt. Load	0.0	0.0	0			

RDL = 2.4 k
RLL = 8.6 k
RTL (left)= **11.0 k**

RDL = 2.4 k
RLL = 8.6 k
RTL (right) = **27.1 k**

Mmax = **16.5 k-ft**

V_n/Ω_V =	*46.0 k*	*24%*	**OK**
M_n/Ω_b =	*39.9 k-ft*	*41%*	**OK**
LL deflection =	*0.04 in*	*21%*	**OK**
TL deflection =	*0.05 in*	*18%*	**OK**

B4 Use: W10X15 *camber = 0.01 in*

Mark: B5

Span (ft) =	16.8		DL(psf)	LL(psf)	trib(ft)	LL w(plf)	TL w(plf)	
L_b (ft) =	2.0	roof	15	55	0	0	0	
LL Deflection < L /	360	floor	70	250	10	2500	3200	
Total Deflection < L /	240	wall	15	0	0	0	0	
Fy (ksi) =	**50**	misc.	0	0	0	0	0	
C_b =	1.00					2500 plf	3200 plf	
Roof Snow Load ? :	**Yes**					beam weight =	40.0 plf	
			DL(k)	LL(k)	x(ft)			
		Pt. Load	0.0	0.0	0			
Reduce Floor LL ? :	**No**	Pt. Load	0.0	0.0	0			

RDL = 6.2 k
RLL = 20.9 k
RTL (left)= **27.1 k**

RDL = 6.2 k
RLL = 20.9 k
RTL (right) = **27.1 k**

Mmax = **113.6 k-ft**

V_n/Ω_V =	*70.2 k*	*39%*	**OK**
M_n/Ω_b =	*142.2 k-ft*	*80%*	**OK**
LL deflection =	*0.50 in*	*89%*	**OK**
TL deflection =	*0.64 in*	*77%*	**OK**

B5 Use: W12X40 *camber = 0.15 in*

Wood Beam / Header

ASD design per NDS 2015

Mark: B6

			DL(psf)	LL(psf)	trib(ft)	LL w(plf)	TL w(plf)
Span (ft) =	29.0	roof	15	55	6	330	420
l_u (ft) =	2.0	floor	0	0	0	0	0
LL Deflection < L /	360	wall	15	0	0	0	0
Total Deflection < L /	240	misc.	0	0	0	0	0
C_D =	1.00					330 plf	420 plf
Roof Snow Load ? :	Yes		DL(lbs)	LL(lbs)	x(ft)		
		Pt. Load	0	0	0		
Reduce Floor LL ? :	No	Pt. Load	0	0	0		

R_{DL} = 1305 lbs
R_{LL} = 4785 lbs
R_{TL} (left)= **6090 lbs**

R_{DL} = 1305 lbs
R_{LL} = 4785 lbs
R_{TL} (right) = **6090 lbs**

M_{max} = **44153 ft-lbs**

	A		B		C		D		E	
	(3) 6x16		5.125x19.5 GLB		6.75x19.5 GLB		8.75x16.5 GLB		(3) 1.75x20	
Wood Species =	Douglas Fir #2		24F-V4		24F-V4		24F-V4		SCL: 26F 2.0E LVL	
F_v (psi) =	170 psi		265 psi		265 psi		265 psi		285 psi	
$f_{v\text{-}max}$ @ d (psi) =	33 psi	19%	81 psi	31%	62 psi	23%	57 psi	22%	77 psi	27%
F_b =	850 psi		2214 psi		2154 psi		2134 psi		2426 psi	
$f_{b\text{-}max}$ (psi) =	802 psi	94%	1631 psi	74%	1239 psi	58%	1334 psi	63%	1514 psi	62%
E (psi) =	1300000 psi		1800000 psi		1800000 psi		1800000 psi		2000000 psi	
LL deflection =	0.79"	82%	0.92"	95%	0.70"	72%	0.89"	92%	0.75"	78%
TL deflection =	1.00"	69%	1.17"	81%	0.89"	61%	1.13"	78%	0.95"	66%
camber (in) =	n/a		4/8"	std=0.36"	3/8"	std=0.36"	3/8"	std=0.36"	n/a	
	Adequate		Adequate		Adequate		Adequate		Adequate	

Selection (A - E): **C** | **B6 Use: 6.75x19.5 GLB 24F-V4**

Mark: B7

			DL(psf)	LL(psf)	trib(ft)	LL w(plf)	TL w(plf)
Span (ft) =	7.0	roof	15	55	4.25	234	298
l_u (ft) =	2.0	floor	0	0	0	0	0
LL Deflection < L /	360	wall	15	0	0	0	0
Total Deflection < L /	240	misc.	0	0	0	0	0
C_D =	1.00					234 plf	298 plf
Roof Snow Load ? :	Yes		DL(lbs)	LL(lbs)	x(ft)		
		Pt. Load	0	0	0		
Reduce Floor LL ? :	No	Pt. Load	0	0	0		

R_{DL} = 223 lbs
R_{LL} = 818 lbs
R_{TL} (left)= **1041 lbs**

R_{DL} = 223 lbs
R_{LL} = 818 lbs
R_{TL} (right) = **1041 lbs**

M_{max} = **1822 ft-lbs**

	A		B		C		D		E	
	(2) 2x8		3.125x6 GLB		5.125x7.5 GLB		8.75x9 GLB		(3) 1.75x5.5	
Wood Species =	Douglas Fir #2		24F-V4		24F-V4		24F-V4		SCL: 26F 2.0E LVL	
F_v (psi) =	180 psi		265 psi		265 psi		265 psi		285 psi	
$f_{v\text{-}max}$ @ d (psi) =	59 psi	33%	71 psi	27%	33 psi	13%	16 psi	6%	47 psi	16%
F_b =	1077 psi		2392 psi		2396 psi		2399 psi		2888 psi	
f_b (psi) =	832 psi	77%	1166 psi	49%	455 psi	19%	185 psi	8%	826 psi	29%
E (psi) =	1600000 psi		1800000 psi		1800000 psi		1800000 psi		2000000 psi	
LL deflection =	0.08"	36%	0.12"	53%	0.04"	17%	0.01"	6%	0.09"	37%
TL deflection =	0.11"	30%	0.16"	45%	0.05"	14%	0.02"	5%	0.11"	32%
camber (in) =	n/a		1/8"	std=0.02"	1/8"	std=0.02"	1/8"	std=0.02"	n/a	
	Adequate		Adequate		Adequate		Adequate		Adequate	

Selection (A - E): **A** | **B7 Use: (2) 2x8 Douglas Fir #2**

Steel Column

AISC-Allowable Stress Design (ASD)

Mark: C1

Unsupported Height (ft) = 11.50

Use: HSS5x5x5/16

Index = 40

Vertical Loads:

P_{LL} (k)	P_{DL} (k)	e_x (in)	e_y (in)
19.50	8.30	4.50	
		-4.50	
			4.50
			-4.50
19.50	8.30		

Member Properties:

A = 5.26 in^2	F_y = 46 ksi
S_x = 7.62 in^3	S_y = 7.62 in^3
I_x = 19.00 in^4	I_y = 19.00 in^4
r_x = 1.90 in^2	r_y = 1.90 in^2
K_x = 1.00	K_y = 1.00
C_c = 111.6	d = 5.00 in

Stress Check:

KL/ r_x = 72.63	KL/ r_y = 72.63
F_a = 19.32 ksi	F_b = 30 ksi
F_{ex} = 28.31	F_{ey} = 28.31

Load Case	P	M_x	M_y	CSR	
D+L	27.80	125.10	0.00	0.84	OK
D+L(+)	27.80	125.10	0.00	0.84	OK
D+L(-)	8.30	37.35	0.00	0.24	OK

Mark: C2

Unsupported Height (ft) = 11.50

Use: HSS5x5x5/16

Index = 40

Vertical Loads:

P_{LL} (k)	P_{DL} (k)	e_x (in)	e_y (in)
19.50	8.30	4.50	
10.30	4.40	-4.50	
			4.50
			-4.50
29.80	12.70		

Member Properties:

A = 5.26 in^2	F_y = 46 ksi
S_x = 7.62 in^3	S_y = 7.62 in^3
I_x = 19.00 in^4	I_y = 19.00 in^4
r_x = 1.90 in^2	r_y = 1.90 in^2
K_x = 1.00	K_y = 1.00
C_c = 111.6	d = 5.00 in

Stress Check:

KL/ r_x = 72.61	KL/ r_y = 72.61
F_a = 19.32 ksi	F_b = 30 ksi
F_{ex} = 28.32	F_{ey} = 28.32

Load Case	P	M_x	M_y	CSR	
D+L	42.50	58.95	0.00	0.72	OK
D+L(+)	32.20	105.30	0.00	0.81	OK
D+L(-)	23.00	28.80	0.00	0.35	OK

Steel Column

AISC-Allowable Stress Design (ASD)

Mark: C3

Unsupported Height (ft) = 11.50

Use: HSS4x4x1/4

Index = 53

Vertical Loads:

P_{LL} (k)	P_{DL} (k)	e_x (in)	e_y (in)
10.30	4.40	4.00	
5.70	2.80	-4.00	
			4.00
			-4.00
16.00	7.20		

Member Properties:

A = 3.37 in^2	F_y = 46 ksi
S_x = 3.90 in^3	S_y = 3.90 in^3
I_x = 7.80 in^4	I_y = 7.80 in^4
r_x = 1.52 in^2	r_y = 1.52 in^2
K_x = 1.00	K_y = 1.00
C_c = 111.6	d = 4.00 in

Stress Check:

KL/ r_x = 90.79	KL/ r_y = 90.79
F_a = 16.15 ksi	F_b = 30 ksi
F_{ex} = 18.12	F_{ey} = 18.12

Load Case	P	M_x	M_y	CSR	
D+L	23.20	24.80	0.00	0.71	OK
D+L(+)	17.50	47.60	0.00	0.80	OK
D+L(-)	12.90	16.40	0.00	0.39	OK

Mark: C4

Unsupported Height (ft) = 11.50

Use: HSS3.5x3.5x1/4

Index = 58

Vertical Loads:

P_{LL} (k)	P_{DL} (k)	e_x (in)	e_y (in)
		3.75	
5.70	2.80	-3.75	
			3.75
			-3.75
5.70	2.80		

Member Properties:

A = 2.91 in^2	F_y = 46 ksi
S_x = 2.88 in^3	S_y = 2.88 in^3
I_x = 5.04 in^4	I_y = 5.04 in^4
r_x = 1.32 in^2	r_y = 1.32 in^2
K_x = 1.00	K_y = 1.00
C_c = 111.6	d = 3.50 in

Stress Check:

KL/ r_x = 104.86	KL/ r_y = 104.86
F_a = 13.41 ksi	F_b = 30 ksi
F_{ex} = 13.58	F_{ey} = 13.58

Load Case	P	M_x	M_y	CSR	
D+L	8.50	31.88	0.00	0.61	OK
D+L(+)	2.80	10.50	0.00	0.19	OK
D+L(-)	8.50	31.88	0.00	0.61	OK

Steel Column

AISC-Allowable Stress Design (ASD)

Mark: C5

Unsupported Height (ft) = 11.50

Use: HSS3.5x3.5x1/4

Index = 58

Vertical Loads:

P_{LL} (k)	P_{DL} (k)	e_x (in)	e_y (in)
8.60	2.40	3.75	
		-3.75	
			3.75
			-3.75
8.60	2.40		

Member Properties:

A =	2.91 in^2	F_y =	46 ksi
S_x =	2.88 in^3	S_y =	2.88 in^3
I_x =	5.04 in^4	I_y =	5.04 in^4
r_x =	1.32 in^2	r_y =	1.32 in^2
K_x =	1.00	K_y =	1.00
C_c =	111.6	d =	3.50 in

Stress Check:

KL/ r_x =	104.55	KL/ r_y =	104.55
F_a =	13.47 ksi	F_b =	30 ksi
F_{ex} =	13.66	F_{ey} =	13.66

Load Case	P	M_x	M_y	CSR	
D+L	11.00	41.25	0.00	0.83	OK
D+L(+)	11.00	41.25	0.00	0.83	OK
D+L(-)	2.40	9.00	0.00	0.16	OK

Mark: C6

Unsupported Height (ft) = 11.50

Use: HSS5x5x5/16

Index = 40

Vertical Loads:

P_{LL} (k)	P_{DL} (k)	e_x (in)	e_y (in)
		4.50	
20.90	6.20	-4.50	
			4.50
			-4.50
20.90	6.20		

Member Properties:

A =	5.26 in^2	F_y =	46 ksi
S_x =	7.62 in^3	S_y =	7.62 in^3
I_x =	19.00 in^4	I_y =	19.00 in^4
r_x =	1.90 in^2	r_y =	1.90 in^2
K_x =	1.00	K_y =	1.00
C_c =	111.6	d =	5.00 in

Stress Check:

KL/ r_x =	72.61	KL/ r_y =	72.61
F_a =	19.32 ksi	F_b =	30 ksi
F_{ex} =	28.32	F_{ey} =	28.32

Load Case	P	M_x	M_y	CSR	
D+L	27.10	121.95	0.00	0.81	OK
D+L(+)	6.20	27.90	0.00	0.18	OK
D+L(-)	27.10	121.95	0.00	0.81	OK

Masonry Wall

Mark: TYPCAL WALL

Wall Data:

Effective Wall Ht. =	12.00	feet
Parapet =	0.00	feet
Nominal Wall Thickness =	8	inches
Actual Wall Thickness =	7.625	inches
Face shell t =	1.25	inches
f'm =	2000	psi
wall self weight =	78	psf
Vertical Rebar Size =	#5	
Vertical Rebar Spacing =	32"	on center
Reinforcing Fy =	60	ksi
Rebar EACH FACE?(Y/N):	N	
Grout solid? (Y/N):	Y	

As=	0.12	in^2/ft
d =	3.81	inches
Em =	1800	ksi
n =	16.1	
fr =	179	psi
Ig =	443	in^4/ft
S =	116	in^3/ft
Mcr =	20801	in-lbs/ft
Ag =	92	in^2/ft

Point Load: None

Dead Load =	0	lbs
Roof Live Load =	0	lbs
Floor Live Load =	0	lbs
Point Load is =	0.00	inches from wall face
Load Bearing Width =	1.00	ft.
e for Point Load =	3.81	inches
Load Distribution Factor =	1.50	

Uniform Gravity Loads:

Dead Load DL =	0	plf
Roof Live Load LL =	0	plf
Floor Live Load LL =	0	plf
Load Bearing is =	-2.0	inches from wall face
e =	1.81	inches
Concentric DL=	0	plf
Concentric LL from Roof =	0	plf
Concentric LL from Floor =	0	plf

Lateral Loads:

Wind (Ultimate) =	29.50	psf	(governs)
Wind (ASD) =	17.70	psf	
Seismic Fp =	0.37	x W	(governs)
Fp (Ultimate) =	29.09	psf	
Fp (ASD) =	20.37	psf	

Door/Window Jamb: None

Wall Opening Width (left) =	0.00	feet
Wall Opening Width (right) =	0.00	feet
Available Jamb Width =	8	inches
Jamb Width =	0.67	feet
Concentration Factor =	1.00	

Axial Stress Check:

Wall self DL (Pw) =	468	plf	
0.20 f'm =	400	psi	
axial load =	5	psi	OK, < 0.20 f'm

Load Cases:

U = 0.90DL + 1.0Wind

Pu (DL) =	**421**	lbs/ft	
Pu (DL)*e	0	in-lbs/ft	
Mu (wind) =	6372	in-lbs/ft	
Mu (P-delta) =	30	in-lbs/ft	
Mu (total) =	**6402**	in-lbs/ft	OK. < phi*Mn

Service Loading

P(ledger) =	0	lbs/ft	Ase =	0.12	in^2
Mservice =	4405	in-lbs/ft	a =	0.38	in.
Service Deflection =	**0.01**	inches	c =	0.48	in.
Allowable Deflection =	**1.01**	inches	Icr =	22.3	in^4
ϕMn =	23874	in-lbs/ft	Ms =	26526	in-lbs

U = 1.2DL + 0.75*[1.6roof LL + 1.0Wind]

Pu (DL + LL) =	**562**	lbs/ft	
Pu (DL + LL)*e	0	in-lbs/ft	
Mu (wind) =	4779	in-lbs/ft	
Mu (P-delta) =	41	in-lbs/ft	
Mu (total) =	**4820**	in-lbs/ft	OK. < phi*Mn

Service Loading

P(ledger) =	0	lbs/ft	Ase =	0.12	in^2
Mservice =	4405	in-lbs/ft	a =	0.39	in.
Service Deflection =	**0.01**	inches	c =	0.49	in.
Allowable Deflection =	**1.01**	inches	Icr =	22.6	in^4
ϕMn =	24307	in-lbs/ft	Ms =	27007	in-lbs

U = 1.2DL + 1.6[roof LL]

Pu (DL + LL) =	**562**	lbs/ft	
Pu (DL + LL)*e	0	in-lbs/ft	
Mu (P-delta) =	41	in-lbs/ft	
Mu (total) =	**41**	in-lbs/ft	OK. < phi*Mn

Service Loading

P(ledger) =	0	lbs/ft	Ase =	0.12	in^2
Mservice =	4405	in-lbs/ft	a =	0.39	in.
Service Deflection =	**0.01**	inches	c =	0.49	in.
Allowable Deflection =	**1.01**	inches	Icr =	22.6	in^4
ϕMn =	24307	in-lbs/ft	Ms =	27007	in-lbs

Use: 8" thick Masonry Wall w/ #5 @ 32" on center

Steel Beam

Project File: ITD Montpelier.ec6

LIC# : KW-06012053, Build:20.24.02.28 Frost Structural Engineering

DESCRIPTION: Edge Angle

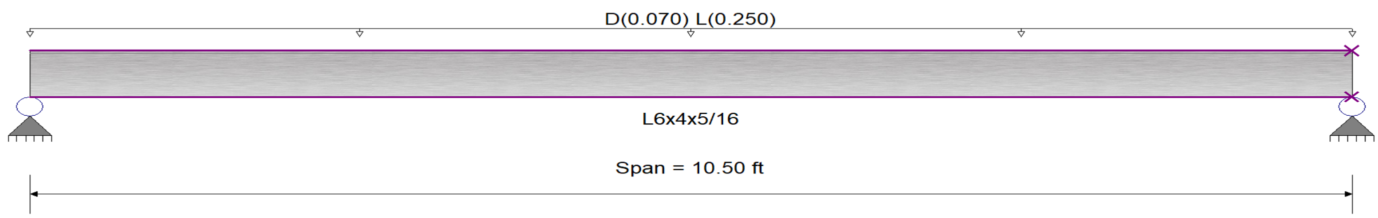
CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	36.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Vertical Leg Up



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.070, L = 0.250 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.696** : 1	Maximum Shear Stress Ratio =	**0.069** : 1
Section used for this span	**L6x4x5/16**	Section used for this span	**L6x4x5/16**
Ma : Applied	4.410 k-ft	Va : Applied	1.680 k
Mn / Omega : Allowable	6.335 k-ft	Vn/Omega : Allowable	24.290 k
Load Combination	+D+L	Load Combination	+D+L
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.208 in	Ratio =	606	>=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.266 in	Ratio =	474	>=240.	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0	<240.0	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 10.50 ft	1	0.152	0.015	0.96		0.96	10.58	6.34	1.00	1.00	0.37	40.56	24.29
+D+L													
Dsgn. L = 10.50 ft	1	0.696	0.069	4.41		4.41	10.58	6.34	1.00	1.00	1.68	40.56	24.29
+D+0.750L													
Dsgn. L = 10.50 ft	1	0.560	0.056	3.55		3.55	10.58	6.34	1.00	1.00	1.35	40.56	24.29
+0.60D													
Dsgn. L = 10.50 ft	1	0.091	0.009	0.58		0.58	10.58	6.34	1.00	1.00	0.22	40.56	24.29

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.2659	5.280		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2	
Max Upward from all Load Conditions	1.680	1.680	54.794
Max Upward from Load Combinations	1.680	1.680	54.794
Max Upward from Load Cases	1.313	1.313	54.794

Steel Beam

Project File: ITD Montpelier.ec6

LIC# : KW-06012053, Build:20.24.02.28 | Frost Structural Engineering | (c) ENERCALC INC 1983-2023

DESCRIPTION: Edge Angle

Vertical Reactions

Support notation : Far left is #: | Values in KIPS

Load Combination	Support 1	Support 2	
D Only	0.368	0.368	54.794
+D+L	1.680	1.680	54.794
+D+0.750L	1.352	1.352	54.794
+0.60D	0.221	0.221	54.794
L Only	1.313	1.313	54.794

Steel Column

Project File: ITD Montpelier.ec6

LIC# : KW-06012053, Build:20.24.02.28 Frost Structural Engineering (c) ENERCALC INC 1983-2023

DESCRIPTION: C7

Code References

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **W6x15**

Analysis Method :	Allowable Strength
Steel Stress Grade	A-992, High Strength, Low Alloy, Fy = 50 ksi
Fy : Steel Yield	50.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height	10.0 ft
Top & Bottom Fixity	Top & Bottom Pinned
Brace condition :	

Unbraced Length for buckling ABOUT X-X Axis = 10.0 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 10.0 ft, K = 1.0

Applied Loads

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

Column self weight included : 150.0 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, Xecc = 2.995 in, Yecc = 2.995 in, D = 6.10 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.2381**	: 1
Load Combination	D Only	
Location of max.above base	9.933	ft
At maximum location values are . . .		
Pa : Axial	6.250	k
Pn / Omega : Allowabl	80.384	k
Ma-x : Applied	-1.512	k-ft
Mn-x / Omega : Allowable	25.365	k-ft
Ma-y : Applied	-1.512	k-ft
Mn-y / Omega : Allowable	10.833	k-ft
PASS Maximum Shear Stress Rati	**0.005525**	: 1
Load Combination	D Only	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	0.1522	k
Vn / Omega : Allowable	27.554	k

Maximum Load Reactions . .

Top along X-X	0.1522 k
Bottom along X-X	0.1522 k
Top along Y-Y	0.1522 k
Bottom along Y-Y	0.1522 k

Maximum Load Deflections . . .

Along Y-Y -0.02017 in at 5.839 ft above base
for load combination : D Only

Along X-X -0.06298 in at 5.839 ft above base
for load combination : D Only

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.238	PASS	9.93 ft	1.66	1.66	46.88	82.76	0.006	PASS	0.00 ft
+0.60D	0.143	PASS	9.93 ft	1.66	1.66	46.88	82.76	0.003	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X @ Top	Y-Y Axis Reaction @ Base	Y-Y @ Top	Mx - End Moments k-ft @ Base	Mx @ Top	My - End Moments @ Base	My @ Top
D Only	6.250	0.152	0.152	-0.152	0.152				
+0.60D	3.750	0.091	0.091	-0.091	0.091				

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X @ Top	Y-Y Axis Reaction @ Base	Y-Y @ Top	Mx - End Moments k-ft @ Base	Mx @ Top	My - End Moments @ Base	My @ Top
Axial @ Base	Maximum	6.250	0.152	0.152	-0.152	0.152				
"	Minimum	3.750	0.091	0.091	-0.091	0.091				
Reaction, X-X Axis Base	Maximum	6.250	0.152	0.152	-0.152	0.152				
"	Minimum	3.750	0.091	0.091	-0.091	0.091				
Reaction, Y-Y Axis Base	Maximum	3.750	0.091	0.091	-0.091	0.091				
"	Minimum	6.250	0.152	0.152	-0.152	0.152				
Reaction, X-X Axis Top	Maximum	6.250	0.152	0.152	-0.152	0.152				
"	Minimum	3.750	0.091	0.091	-0.091	0.091				
Reaction, Y-Y Axis Top	Maximum	6.250	0.152	0.152	-0.152	0.152				

Steel Column

Project File: ITD Montpelier.ec6

LIC# : KW-06012053, Build:20.24.02.28 Frost Structural Engineering (c) ENERCALC INC 1983-2023

DESCRIPTION: C7

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
"	Minimum	3.750	0.091	0.091	-0.091	0.091				
Moment, X-X Axis Base	Maximum	6.250		0.152	-0.152	0.152				
"	Minimum	6.250		0.152	-0.152	0.152				
Moment, Y-Y Axis Base	Maximum	6.250	0.152	0.152	-0.152	0.152				
"	Minimum	6.250	0.152	0.152	-0.152	0.152				
Moment, X-X Axis Top	Maximum	6.250	0.152	0.152	-0.152	0.152				
"	Minimum	6.250	0.152	0.152	-0.152	0.152				
Moment, Y-Y Axis Top	Maximum	6.250	0.152	0.152	-0.152	0.152				
"	Minimum	6.250	0.152	0.152	-0.152	0.152				

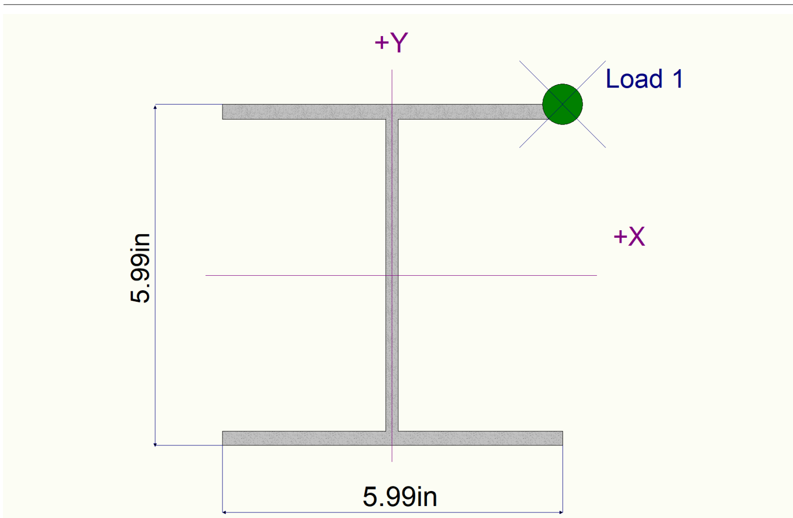
Maximum Deflections for Load Combinations

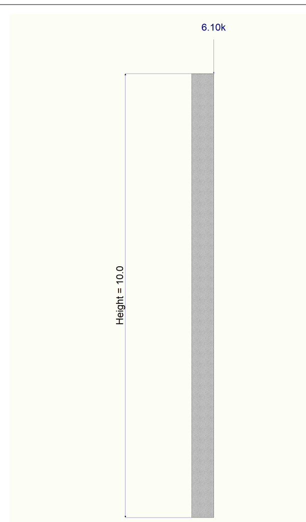
Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	-0.0630 in	5.839 ft	-0.020 in	5.839 ft
+0.60D	-0.0378 in	5.839 ft	-0.012 in	5.839 ft

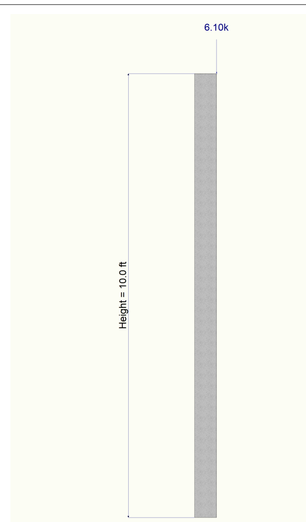
Steel Section Properties : W6x15

Depth	=	5.990 in	I xx	=	29.10 in^4	J	=	0.101 in^4
Web Thick	=	0.230 in	S xx	=	9.72 in^3	Cw	=	76.50 in^6
Flange Width	=	5.990 in	R xx	=	2.560 in			
Flange Thick	=	0.260 in	Zx	=	10.800 in^3			
Area	=	4.430 in^2	I yy	=	9.320 in^4			
Weight	=	15.000 plf	S yy	=	3.110 in^3	Wno	=	8.580 in^2
Kdesign	=	0.510 in	R yy	=	1.450 in	Sw	=	3.340 in^4
K1	=	0.563 in	Zy	=	4.750 in^3	Qf	=	2.150 in^3
rts	=	1.660 in				Qw	=	5.320 in^3
Ycg	=	0.000 in						

Sketches







Steel Beam

Project File: ITD Montpelier.ec6

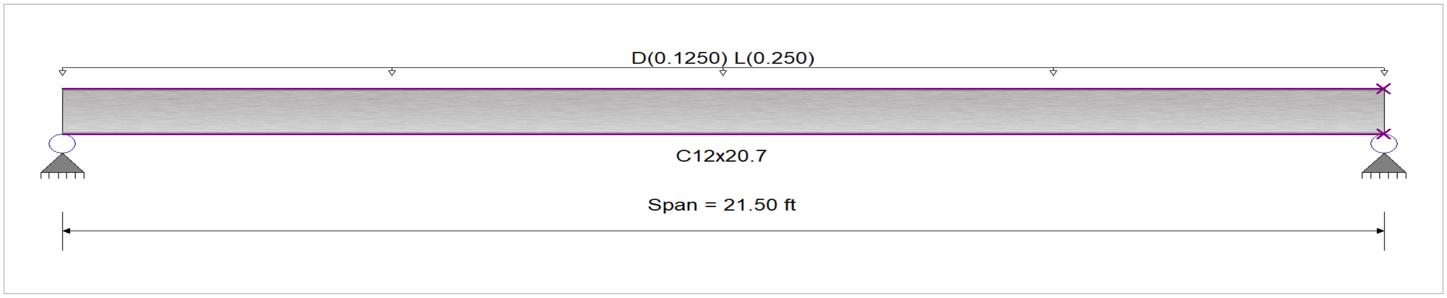
DESCRIPTION: Stair Stringers

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	36.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.050, L = 0.10 ksf, Tributary Width = 2.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.471** : 1	Maximum Shear Stress Ratio =	**0.092** : 1
Section used for this span	**C12x20.7**	Section used for this span	**C12x20.7**
Ma : Applied	21.668 k-ft	Va : Applied	4.031 k
Mn / Omega : Allowable	45.988 k-ft	Vn/Omega : Allowable	43.769 k
Load Combination	+D+L	Load Combination	+D+L
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.323 in	Ratio =	799	>=360.	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360.0	n/a
Max Downward Total Deflection	0.484 in	Ratio =	533	>=240.	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0	<240.0	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 21.50 ft	1	0.157	0.031	7.22		7.22	76.80	45.99	1.00	1.00	1.34	73.09	43.77
+D+L													
Dsgn. L = 21.50 ft	1	0.471	0.092	21.67		21.67	76.80	45.99	1.00	1.00	4.03	73.09	43.77
+D+0.750L													
Dsgn. L = 21.50 ft	1	0.393	0.077	18.06		18.06	76.80	45.99	1.00	1.00	3.36	73.09	43.77
+0.60D													
Dsgn. L = 21.50 ft	1	0.094	0.018	4.33		4.33	76.80	45.99	1.00	1.00	0.81	73.09	43.77

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.4841	10.811		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	4.031	4.031
Max Upward from Load Combinations	4.031	4.031
Max Upward from Load Cases	2.688	2.688
D Only	1.344	1.344

Steel Beam

Project File: ITD Montpelier.ec6

LIC# : KW-06012053, Build:20.24.02.28 Frost Structural Engineering (c) ENERCALC INC 1983-2023

DESCRIPTION: Stair Stringers

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2
L Only	2.688	2.688

LATERAL DESIGN

Design Seismic Lateral Loads

Equivalent Lateral Force Procedure per Chapters 11 and 12 of ASCE 7-16

Key Plan Area = 1
Risk Category = II [ASCE 7-16 table 1.5-1]
Geotech Report Done? : Yes
Site Class = D
Seismic Design Category = D [per ASCE 7-16 tables 11.6-1 and 11.6-2]
Importance Factor I_E = **1.00** [see ASCE 7-16 table 1.5-2]
Seismic Force Resisting System (Table 12.2-1) = A.Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or
Structural System Height Limits (ft): **65**
Response Modification Coefficient, R = **6.5**
Overstrength factor, Ω_0 = **3**
Deflection Amplification Factor, Cd = **4**

Design Spectral Response Accelerations:

S_S =	90.0%	S_1 =	29.3%	[per ATC Hazard by Location Website]
Fa =	1.14	Fv =	2.01	[per ASCE 7-16 table 11.4-1 & 11.4-2]
S_{MS} =	1.026g	S_{M1} =	0.590g	[ASCE 7-16 equation 11.4-1 & 11.4-2]
S_{DS} =	0.684g	S_{D1} =	0.393g	[ASCE 7-16 equation 11.4-3 & 11.4-4]

T_L =	6	C_S =	0.105	T =	0.129	T_0 =	0.115
C_t =	0.020	$C_{s\text{-}max}$ =	0.469	T_a =	0.129	T_S =	0.575
x =	0.750	$C_{s\text{-}min}$ =	0.030	T_{max} =	0.168	S_a =	0.684
		C_S (controls)=	**0.105**	C_U =	1.3	k =	1.00

Main Seismic Force Resisting System:

V = C_SW = 28.9 [per ASCE 7-16 equation 12.8-1]

Vertical Distribution of Main Seismic Force Resisting System [per ASCE 7-16 section 12.8.3]

Level	h_x (ft)	w_x (kip)	$w_x h_x^k$ (kip-ft)	C_{vx}	F_x (kip)	V_x (kip)
Mezzanine	12	275.0	3300	1.000	28.9	28.9
	Totals:	275	3300	1.0	28.9	

Transverse Diaphragm Design Forces [per ASCE 7-16 section 12.10.1.1]

Level	h_x (ft)	w_{px} (kip)	$\sum w_i$ (kip)	$\sum F_i$ (kip)	$F_{px\,(min)}$ (kip)	$F_{px\,(max)}$ (kip)	F_{px} (kip)
Mezzanine	12	269.2	269.2	28.3	36.8	73.7	36.8

Longitudinal Diaphragm Design Forces [per ASCE 7-16 section 12.10.1.1]

Level	h_x (ft)	w_{px} (kip)	$\sum w_i$ (kip)	$\sum F_i$ (kip)	$F_{px\,(min)}$ (kip)	$F_{px\,(max)}$ (kip)	F_{px} (kip)
Mezzanine	12	247.6	247.6	26.1	33.9	67.7	33.9

Seismic Weights

Level = Roof		**Area = 1**	**Total Seismic Weight =**	**275.0**	
	Walls			Roof / Floor	
Long. Wall Length (ft) =	150	40	Area (ft^2) =	1690	690
Trans. Wall Length (ft) =	100	0	Dead Load (psf) =	70	15
Plate Height (ft) =	11.5	12	Live / Snow Load (psf) =	250	55
Parapet Height (ft) =	0	0	% Live / Snow Load =	25%	20%
Wall Weight (psf) =	10	78	Int. Partition Load (psf) =	0	0
Misc. (lbs) =	0	0	Misc. (lbs) =	0	0
Long. Weight (k) =	8.6	18.7	Weight (k) =	223.9	17.9
Trans. Weight (k) =	5.8	0.0	**Transverse Weight (k) =**	**269.2**	
			Longitudinal Weight (k) =	**247.6**	

Torsional Analysis of Rigid Diaphragm

Project File: ITD Montpelier.ec6

LIC# : KW-06012053, Build:20.24.02.28 Frost Structural Engineering

DESCRIPTION: Mezzanine Diaphragm

General Information

IBC 2021, ASCE 7-16

Applied Lateral Force	39.50 k
.....Additional Orthogonal Force	k
Maximum Load Used for Analysis :	39.50 k

Note: This load is the vector resolved from the above two entries and will be applied to the system of elements at angular increments.

Load Orientation Angular Increment	15.0 deg
Load Location Angular Increment	15.0 deg

Center of Rigidity Location (calculated) . . .

"X" dist. from Datum	25.782 ft
"Y" dist. from Datum	44.795 ft

Center of Shear Application :

Distance from "X" datum point	17.250 ft
Distance from "Y" datum point	24.0 ft

Accidental Torsion values per ASCE 7-05 12.8.4.2

Ecc. as % of Maximum Dimension	5.00 %

Maximum Dimensions :

Along "X" Axis	ft
Along "Y" Axis	ft

Accidental Eccentricity +/- from "X" Coord. of Load Application :	0.0 ft
Accidental Eccentricity +/- from "Y" Coord. of Load Application :	0.0 ft

Wall Information

Label	Wall Deflections (Stiffness) for 1.0 kip load : Along Wall "y" Dir	Along Wall "x" Dir	X Wall C.G. Location	Y Wall C.G. Location	Wall Angle CCW	Wall Fixity	Length	Height	Thickness	E - Bending	E - Shear
2.5	5.8247E-004 in	3.0059E+004 in	11.25 ft	24.5 ft	90 deg	Fix-Pin	13.5 ft	11.5 ft	6 in	1 Mpsi	1 Mpsi
3	6.6958E-004 in	3.1827E+004 in	34.25 ft	6.375 ft	90 deg	Fix-Pin	12.75 ft	11.5 ft	6 in	1 Mpsi	1 Mpsi
3 (2)	8.2964E-004 in	3.4683E+004 in	34.25 ft	22.5 ft	90 deg	Fix-Pin	11.7 ft	11.5 ft	6 in	1 Mpsi	1 Mpsi
3 (3)	4.8440E-003 in	6.6523E+004 in	34.25 ft	37.6 ft	90 deg	Fix-Pin	6.1 ft	11.5 ft	6 in	1 Mpsi	1 Mpsi
3 (4)	2.4305E-002 in	1.1594E+005 in	34.25 ft	46.7 ft	90 deg	Fix-Pin	3.5 ft	11.5 ft	6 in	1 Mpsi	1 Mpsi
C	1.2639E-004 in	1.4364E+004 in	20.5 ft	47.75 ft	0 deg	Fix-Pin	28.25 ft	11.5 ft	6 in	1 Mpsi	1 Mpsi
D.5	3.2846E-003 in	5.7970E+004 in	3 ft	11.25 ft	0 deg	Fix-Pin	7 ft	11.5 ft	6 in	1 Mpsi	1 Mpsi
D.5 (2)	2.5469E-003 in	5.2906E+004 in	15.5 ft	11.25 ft	0 deg	Fix-Pin	7.67 ft	11.5 ft	6 in	1 Mpsi	1 Mpsi

Torsional Analysis of Rigid Diaphragm

Project File: ITD Montpelier.ec6

LIC# : KW-06012053, Build:20.24.02.28 Frost Structural Engineering (c) ENERCALC INC 1983-2023

DESCRIPTION: Mezzanine Diaphragm

ANALYSIS SUMMARY

Maximum shear forces applied to resisting elements. Eccentricity with respect to Center of Rigidity

	Max Shear along Member Local "y-y" Axis				Max Shear along Member Local "x-x" Axis			
Resisting Element	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)
2.5	120	8.53	-20.79	**24.875**	0	8.53	-20.79	**0.000**
3	255	8.53	-20.79	**11.219**	0	8.53	-20.79	**0.000**
3 (2)	255	8.53	-20.79	**9.054**	0	8.53	-20.79	**0.000**
3 (3)	255	8.53	-20.79	**1.551**	0	8.53	-20.79	**0.000**
3 (4)	255	8.53	-20.79	**0.309**	0	8.53	-20.79	**0.000**
C	195	8.53	-20.79	**23.496**	90	8.53	-20.79	**0.000**
D.5	345	8.53	-20.79	**7.650**	90	8.53	-20.79	**0.000**
D.5 (2)	345	8.53	-20.79	**9.866**	90	8.53	-20.79	**0.000**

Layout of Resisting Elements

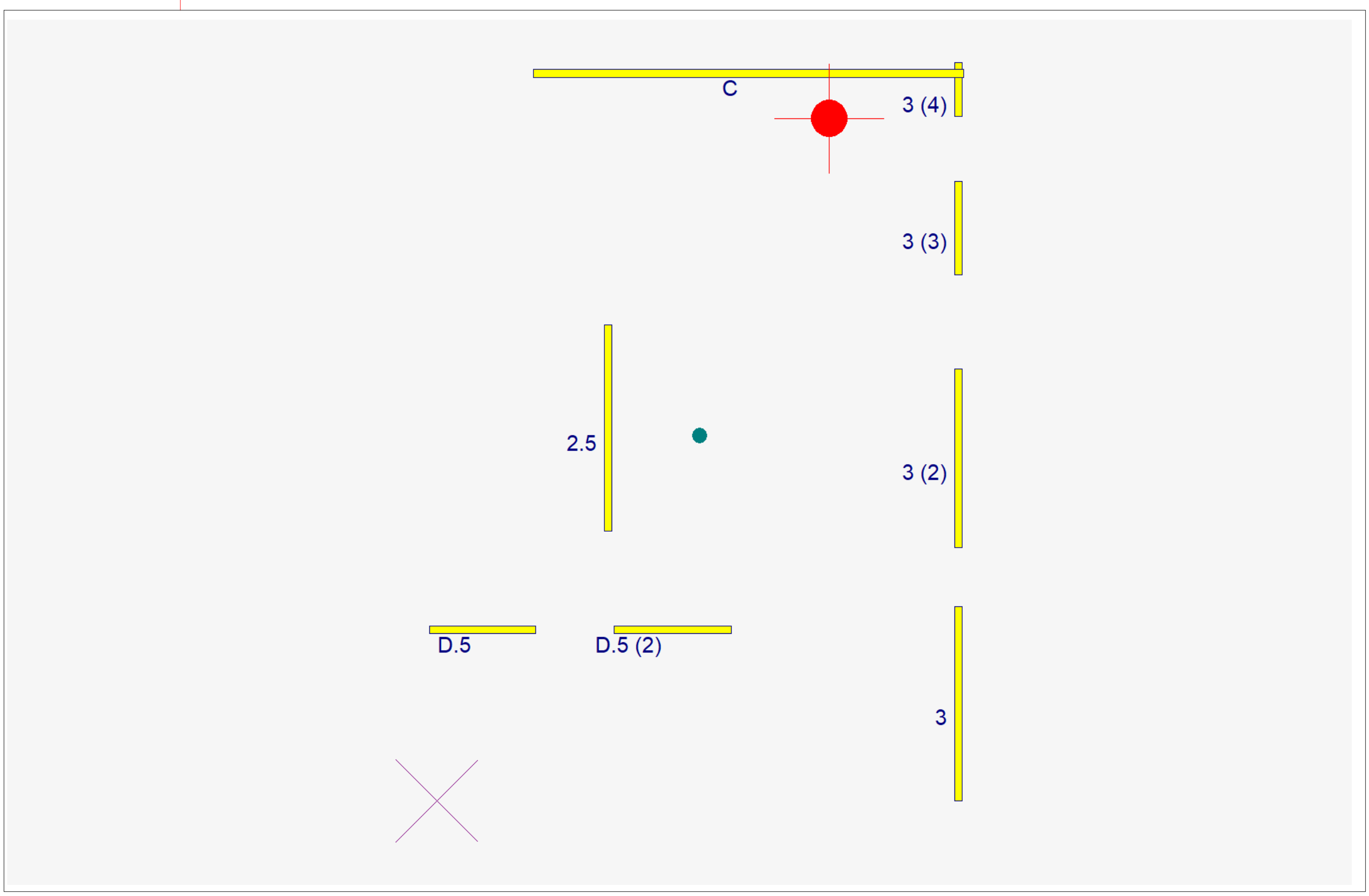
Legend : Defined Wall

X Datum

Center of Rigidity

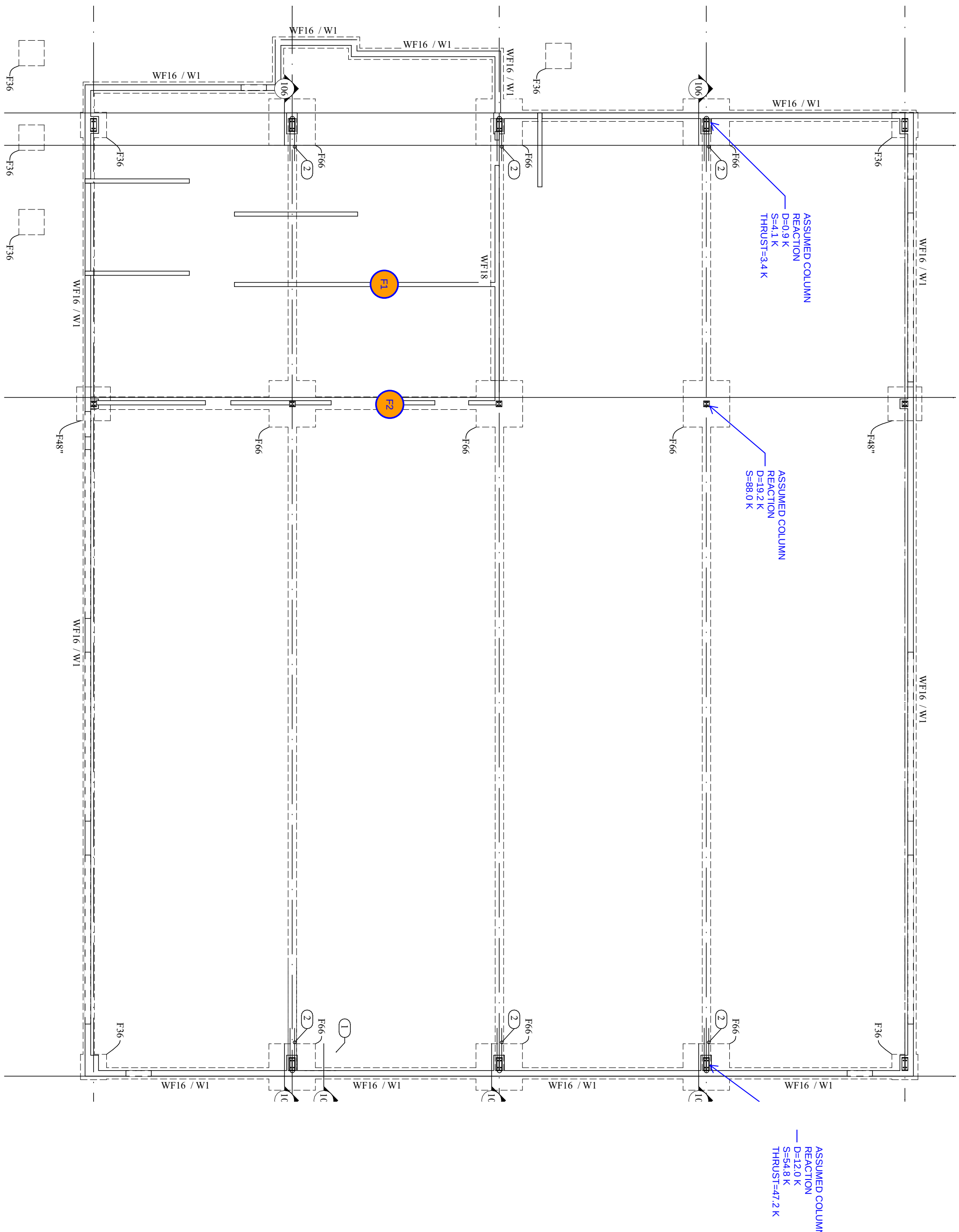
Center of Mass

Accidental eccentricity application boundary



FOUNDATION DESIGN

FOUNDATION KEY PLAN



Continuous Concrete Wall Footings

Strength Design per ACI 318-11

Mark: F1

Allowable q (psf) = 1500

Code Increases (Y/N) = **N**

	DL	LL	trib(ft)	P_{DL} (k/ft)	P_{LL} (k/ft)	P_u (k/ft)
				U = 1.2DL + 1.6LL		
roof	15	55	0	0.00	0.00	0.00
floor	70	250	11.25	0.79	2.81	5.45
wall	10	0	10	0.10	0.00	0.12
misc.	0	0	0	0.00	0.00	0.00
			Total(k/ft) =	0.89	2.81	**5.57**

Footing Dimensions

Width (in) = **30**

Thickness (in) = **10**

Adjusted q (psf) = 1500

Allowable qu (psf) = 2256.081

q_{max} (psf) = 2226 **OK**

Footing Type: Strip(S) or Turndown Edge(T) or Monolithic w/Slab(M) = **S**

Max. Point Load:

stemwall width(in) = 8

stemwall height(in) = 0

Pmax (kips) = **8.1**

unfactored

Continuous Reinforcing

Rebar Size = #4

area of bar(in^2) = 0.20

bar diameter(in) = 0.50

Total Bars = 3

Check p

Min p = 0.0018

Max p = 0.0134

Use p = 0.0018

As req(in^2)= 0.54

Other Parameters

fy (psi) = 60000

f'c (psi) = 2500

B1 = 0.85

Reinforcement Top and Bottom(Y/N) = N

Transverse Reinforcing

Not Required

Rebar Size = #4

area of bar(in^2) = 0.20

bar diameter(in) = 0.50

Spacing (in) = 132

L' (ft) = 0.92

Mu (k-ft) = 0.51

d (in)= 6.25

As req(in^2/ft)= 0.02

Check p

p = 0.0002

Min p = 0.0033

Max p = 0.0134

Use p = 0.0033

Less than pmax, OK

Check Development Length

fy (psi) = 60000

Ld_1 (in) = 24.0

Ld_2 (in) = 12

Ld_3 (in) = 12

Available Ld (in) = 22.3

F1 Use: 30" wide x 10" thick Continuous Concrete Footing
w/ 3 #4 Continuous

Project Name:	Project No. :	Eng. :	Date :	Sheet :
ITD Preston Maintenance Building	24-268	KBB	11/26/24	30 of 31

Continuous Concrete Wall Footings

Strength Design per ACI 318-11

Mark: F2

Allowable q (psf) = 1500

Code Increases (Y/N) = **N**

	DL	LL	trib(ft)	P_{DL} (k/ft)	P_{LL} (k/ft)	P_u (k/ft)
				U = 1.2DL + 1.6LL		
roof	15	55	0	0.00	0.00	0.00
floor	70	250	7	0.49	1.75	3.39
wall	10	0	10	0.10	0.00	0.12
misc.	0	0	0	0.00	0.00	0.00
			Total(k/ft) =	0.59	1.75	**3.51**

Footing Dimensions

Width (in) = **24**

Thickness (in) = **10**

Adjusted q (psf) = 1500

Allowable qu (psf) = 2248.718

q_{max} (psf) = 1754 **OK**

Footing Type: Strip(S) or Turndown Edge(T) or Monolithic w/Slab(M) = **S**

Max. Point Load:

stemwall width(in) = 8

stemwall height(in) = 0

Pmax (kips) = 6.5

unfactored

Continuous Reinforcing

Rebar Size = #4

area of bar(in^2) = 0.20

bar diameter(in) = 0.50

Total Bars = 3

Check p

Min p = 0.0018

Max p = 0.0134

Use p = 0.0018

As req(in^2)= 0.432

Other Parameters

fy (psi) = 60000

f'c (psi) = 2500

B1 = 0.85

Reinforcement Top and Bottom(Y/N) = N

Transverse Reinforcing

Not Required

Rebar Size = #4

area of bar(in^2) = 0.20

bar diameter(in) = 0.50

Spacing (in) = 319

L' (ft) = 0.67

Mu (k-ft) = 0.21

d (in)= 6.25

As req(in^2/ft)= 0.01

Check p

p = 0.0001

Min p = 0.0033

Max p = 0.0134

Use p = 0.0033

Less than pmax, OK

Check Development Length

fy (psi) = 60000

L_{d1} (in) = 24.0

L_{d2} (in) = 12

L_{d3} (in) = 12

Available Ld (in) = 19.3

F2 Use: 24" wide x 10" thick Continuous Concrete Footing
w/ 3 #4 Continuous