Structural System Design

Submittal A

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PROJECT LOCATION INFORMATION

The building shown in Figure 1 is located in Laramie, Wyoming with a latitude and longitude of 41.3108° N, -105.5903° E. This building will be designed as an office building.

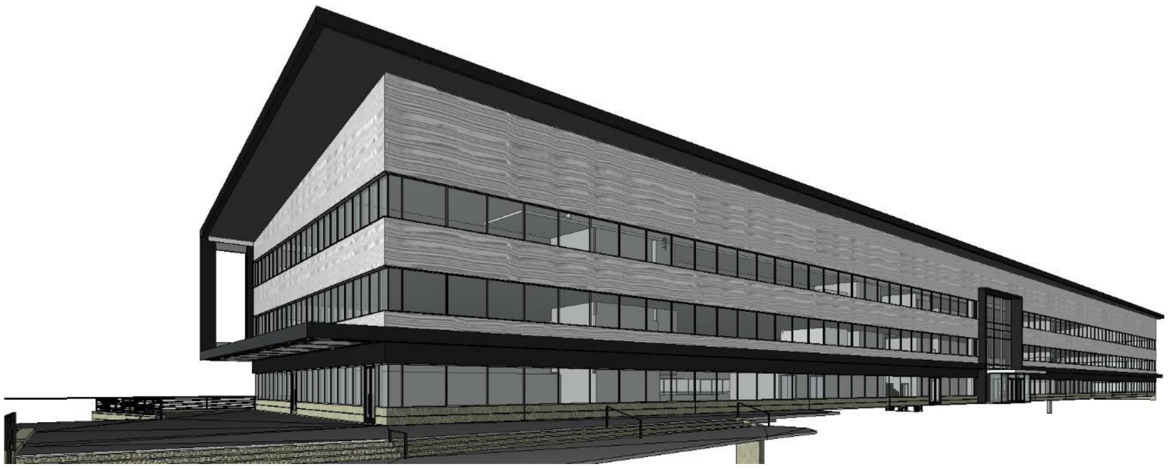


Figure 1. Framing Plan for Structural Design.

Design Criteria

The building is to have three stories above grade. The first story is 14-ft high, the second and third stories are 13-ft and 6-in high, and there is an 8-ft tall roof screen wall on a 1:12 sloped roof. The soil site class was determined to be site class D. The building will be designed per ASCE 7-10. The building is to be framed with steel bracing on a concrete foundation and steel K-bar joists supporting the non-mechanical roof load. In the longitudinal direction, the lateral resisting system will be steel and concrete composite ordinary moment frames. In the transverse direction, the lateral resisting system will be steel and concrete composite ordinary braced frames.

STRUCTURAL SYSTEM NARRATIVE

Structural System Narrative

1. Executive Summary

A. The building is to be framed with steel bracing on a concrete foundation and steel K-bar joists supporting the non-mechanical roof load.

B. In the longitudinal direction, the lateral resisting system will be steel and concrete composite ordinary moment frames. In the transverse direction, the lateral resisting system will be steel and concrete composite ordinary braced frames.

2. Building Code

A. The governing building code for the Project will be the 2015 International Building Code. The fundamental design criteria are anticipated to be as follows:

1) 2015 International Building Code
2) ASCE 7-10

3. Loading & Design Criteria

A. Roof Snow Loads:

1) Design Roof Snow Load = 30 psf (Control of Laramie WY, Building Codes Appendix A.2)
2) Flat Roof Snow Load = 23.1 psf
3) Snow Exposure Factor (Ce) = 1
4) Importance Factor (I) = 1.1
5) Thermal Factor (Ct) = 1
6) Ground Snow Load (Pg) = 30 sf
7) Rain on Snow Surcharge = Na
8) Sloped Roof Factor (Cs) = 1

B. Wind Design Data:

1) Basic Wind Speed = 115 mph
2) Mean Roof Height = 47.5 ft
3) Risk Category = II
4) Exposure Category = C
5) Enclosure Classification = Enclosed building
6) Internal Pressure Coeff. (Cp) = -.5
7) Directionality (Kd) = .85
8) Topographical Factor (Kzt) = 1

C. Earthquake Design Data:

1) Risk Category = II
2) Importance Factor (I) = 1.25
3) Mapped Spectral Response Accelerations:
   a) Ss = .218 g
   b) S1 = .065 g
4) Site Class = B

5) Spectral Response Coef.:

  a) Sds = 2.32 g
  b) Sd1 = .104 g

6) Seismic Design Category = B
7) Basic Structural System = Building Frame Systems
8) Seismic Resisting System = Steel braced frames not specifically detailed for seismic resistance
9) Design Base Shear V = 775 Kips
10) Seismic Response Coef. (Cs) = .0750
11) Response Mod. Factor (R) = 3
12) Analysis Procedure = ELF

D. Design Loads:

1) Dead Load = 85 psf
2) Live Load at Elevated Floors = 80 psf
3) Mechanical Loads = 20 psf

4. New Structure

A. The proposed structural systems described below:

B. Foundation:

1) The foundation will be constructed of conventional shallow spread footings.
  a) Allowable bearing pressure of 3000 psf.
  b) Frost depth is at 36”.
  c) Spread footings will be a minimum of 24” square.

C. Lateral System:
1) Steel and Concrete Composite Ordinary Moment frames (longitudinal; response modification factor =3)
  a) A frame in which members and joins resist lateral forces by flexure and along the axis of the members. (ASCE7-10, Section 11.2)
2) Steel and Concrete Composite Ordinary Brace frames (transverse; response modification factor =3)
  a) An essentially vertical truss, or is equivalent, of the concentric or eccentric type that is provided in a building frame system or dual system to resist seismic forces. (ASCE-10, Section 11.2)

D. Floor Framing:
1) First floor framing is slab on grade concrete and decking.
2) All upper floors will be composite decking supported by steel framing.

E. Roof Framing:

1) For sub-mechanical area, the roof will be composed of composite decking supported by steel framing.
2) Non-mechanical sloped roof will be supported by open-web steel joists.

Design Loads

Dead Loads:

Roof Dead Load		Floor Dead Load		Wall Dead Load		Mech Bay	
Load (psf)	Item	Load (psf)	Item	Load (psf)	Item	Load (psf)	Item
12.0	Roofing	10.0	Partitions	35.0	Exterior Cladding	150	MB
20.0	Ceiling + Mech	15.0	Ceiling +Mech				
		45.0	2 VLI 4.50 (pg.54)				
47.0	**:Total**	**85.0**	**:Total**	**35**	**:Total**	**150**	**:Total**

Structural Load Estimate= 15 psf

Live Loads:

Roof Live & Snow		Floor Live					
Load (psf)	Item	Load (psf)	Item	Load (psf)	Item	Load (psf)	Item
20.0	Ordinary	50.0	Offices				
30.0	Roof Snow	80.0	Corridors				
		15.0	Partitions				
30.0	**:Control**	**80.0**	**:Control**				

Lateral Loads:

Wind Loads		Seismic Loads					
Load (psf)	Item	Load (psf)	Item	Load (psf)	Item	Load (psf)	Item
182	EW Direction	681	Longitudinal Direction				
691	NS Direction	775	Transverse Direction				
691	**:Control**	**775**	**:Control**				

Snow Loads

Snow Design

Wind Load Parameters					
Ground Snow Load	p_g=	30.000	psf		
Surface Roughness	SR=	C		(26.7.2)	
Importance factor	I_s=	1.100		(1.5-1)	
Exposure Factor	C_e=	1.000		(7.3-1)	
Thermal Factor	C_t=	1.000		(7.3-2)	
Slope Factor	C_s=	1.000		(7.4-1)	(C_s of 1.0 is conservative)
Roof Slope	Rs=	1:12			
		4.8	degrees		

Flat Roof Snow Load	$p_{f=}$	23.1	psf <	30	30 Psf Controls for Laramie		
Minimum SL Low-Slope	p_m=	22.0	psf				
Sloped Roof SL	p_s=	23.1	psf				
Drift Coefficient	λ=	17.9	<= 30 psf				
Height Balanced SL	h_b=	1.3	ft	h_c=	2	ft	
Height Drift	h_d=	0.6	ft	l_u=	30	ft	
Drift Width	dW=	2.6	ft				
Base Snow Load	w_b=	**23.1**	psf				
Drift Snow Load	w_d=	**11.6**	psf				
Rain on Snow Surchage	r_s=	NA	psf				

Wind Loads

Wind Design

Wind Load Parameters			
Basic Wind Speed	V=	115	mph
Surface Roughness	=	C	(26.7.2)
Ground Elevation	=	0	ft
Ground Elevation Factor	Ke=	1	(26.9)
Topographic Factor	Kd=	0.85	(26.6-1)
Basic Velocity Pressure	Kzt=	1	(26.8.2)

Geometric and Wind Pressure Parameters (26.1.2)

Level	Height (ft)	Elevation (ft)	Kz	qz (psf)	qh (psf)	Trib H. (ft)
Parapet	0	0	1.29	37	37	0.0
R	0	0	1.27	37	37	0.0
8	0	0	1.248	36	37	0.0
7	0	0	1.21	35	37	0.0
6	0	0	1.155	33	37	0.0
Screen	8.8	52	1.11	32	37	4.4
R	12.6	39.4	1.045	30	37	10.7
3	11.9	27.5	0.97	28	37	12.3
2	13.5	14	0.85	24	37	12.7
1	0	0	0.85	24	37	7.0
Level	46.8					

Wind Strength-Level Story Forces (k)

EW Direction

L/B=		3.19	
Leeward Cp=		-0.325	(27.3-1)
Trib. B (ft)	Trib. A (sf)	Net p (psf)	F (k)
113.8	0	42	0
113.8	0	41	0
113.8	0	41	0
113.8	0	40	0
113.8	0	39	0
113.8	502.6167	38	19
113.8	1219.557	36	44
113.8	1394.05	34	48
113.8	1445.26	32	46
113.8	796.6	32	25
		Total	182

Wind Base Shear Force

NS Direction

L/B=		0.31	
Leeward Cp=		-0.5	(27.3-1)
Trib. B (ft)	Trib. A (sf)	Net p (psf)	F (k)
363.2	0	48	0
363.2	0	48	0
363.2	0	47	0
363.2	0	46	0
363.2	0	45	0
363.2	1604.133	44	71
363.2	3892.293	43	166
363.2	4449.2	41	182
363.2	4612.64	38	176
363.2	2542.4	38	97
		Total	691

Seismic Loads

Seismic Design

Longitudinal Direction

Ground motion parameters

Mapped MCER short period		Ss =	0.218	g
Mapped MCER 1-s period		S1 =	0.065	g
Soil site class		=	D	
Short period site coeff.	(Table-11.4-1)	Fa =	-	
Long period site coeff.	(table-11.4-2)	Fv =	-	
MCE short spec. accel.		SMs =	0.349	g
MCE 1-s spec. accel.		SM1 =	0.156	g
DBE short spec. accel.		SDs =	0.232	g
DBE 1-s spec. accel.		SD1 =	0.104	g

ELF procedure parameters

Seismic Design Category		SDC =	B		Table 12.8-2	
Response modification factor		R =	**3.00**		**$T_a=C_t*h_n^x$**	
Fundamental period		T_a =	**0.661**	s	h_n =	52
Long-period transition		TL =	4	s	C_t =	0.028
Importance factor		I_e =	1.25		x =	0.8
Exponent for fund. period	(12.8.3)	k =	1.080			
Max seismic response coeff.		Cs,max =	0.066		Low Check 12.8.1.1	
Min seismic response coeff.		Cs,min =	0.013		0.01276	
Seismic response coeff.		**Cs =**	**0.066**			

Seismic weight calculations

Unif. Dead Load (psf)			
Roof	Floor	Wall	Parapet
50	85	35	0

Unif. Live Load (psf)		
Roof	Floor	-
20	80	-

Level	Trib. H (ft)	Area (ft²) Roof	Floor	Wall	W (k)
R	12.3	41310.2		11689.23	2475
3	12.7		41310.21	12118.45	3936
2	13.8		41310.21	13115.21	3970
				Total:	**10381**

ELF strength-level seismic story forces (k)

Level	**Elevation** (ft)	**$w_xh_x^k$** (k-ft)	**Cv_x**	**F** (k)
R	39.4	131000	0.38	261.62
3	27.5	141233	0.41	282.06
2	14.0	68709	0.20	137.22
		340943	**1.00**	**681**

V= 681 Kips

Transverse Direction

Ground motion parameters

Mapped MCER short period		Ss =	0.218	g
Mapped MCER 1-s period		S1 =	0.065	g
Soil site class		=	D	
Short period site coeff.	(Table-11.4-1)	Fa =	-	
Long period site coeff.	(table-11.4-2)	Fv =	-	
MCE short spec. accel.		SMs =	0.349	g
MCE 1-s spec. accel.		SM1 =	0.156	g
DBE short spec. accel.		SDs =	0.232	g
DBE 1-s spec. accel.		SD1 =	0.104	g

ELF procedure parameters

Seismic Design Category		SDC =	B		Table 12.8-2	
Response modification factor		R =	3.00		**$T_a=C_t*h_n^x$**	
Fundamental period		T_a =	**0.58**	s	h_n =	51.91667
Long-period transition		TL =	4	s	C_t =	0.03
Importance factor		I_e =	1.25		x =	0.75
Exponent for fund. period	(12.8.3)	k =	1.040			
Max seismic response coeff.		Cs,max =	0.075		Low Check	
Min seismic response coeff.		Cs,min =	0.013		0.01276	
Seismic response coeff.		**Cs =**	**0.075**			

Seismic weight calculations

Unif. Dead Load (psf)			
Roof	Floor	Wall	Parapet
50	85	35	0

Unif. Live Load (psf)		
Roof	Floor	-
20	80	-

Level	Trib. H (ft)	Area (ft^2) Roof	Floor	Wall	W (k)
R	12.3	41310.2		11689.23	2475
3	12.7		41310.21	12118.45	3936
2	13.8		41310.21	13115.21	3970
				Total:	**10381**

ELF strength-level seismic story forces (k)

Level	**Elevation** (ft)	**$w_x h_x^k$** (k-ft)	**Cv_x**	**F** (k)
R	39.4	113012	0.38	293.59
3	27.5	123616	0.41	321.13
2	14.0	61793	0.21	160.53
		298421	**1.00**	**775**

V= 775 Kips

Longitudinal Direction=	113.8	ft
Transverse Direction=	363.2	ft
Screen Wall Height=	8.9	ft
Wall Perimeter=	953.8	ft

Roof Dead Load		**Floor Dead Load**		**Wall Dead Load**		**Parapet Dead Load**	
Load (psf)	Item	Load (psf)	Item	Load (psf)	Item	Load (psf)	Item
15.0	Roofing	10.0	Partitions	35.0	Exterior Cladding	NA	Cladding Wall
20.0	Ceiling + Mech	15.0	Ceiling +Mech				
		45.0	2 VLI 4.50 (pg.54)				
50.0	**:Total**	**85.0**	**:Total**	**35**	**:Total**	**0**	**:Total**

Structural Weight Guess=	15	psf

APPENDIX A: SUPPLEMENTAL LOADING INFORMATION

APPENDIX A.1: SEISMIC PARAMETERS

USGS Design Maps Detailed Report

ASCE 7-10 Standard (41.31086°N, 105.59035°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 0.218 g
From Figure 22-2 [2]	S_1 = 0.065 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_s \le 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_s

For Site Class = D and S_s = 0.218 g, F_a = 1.600

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.065 g, F_v = 2.400

Equation (11.4–1):	$S_{MS} = F_aS_s = 1.600 \times 0.218 = 0.349$ g
Equation (11.4–2):	$S_{M1} = F_vS_1 = 2.400 \times 0.065 = 0.156$ g

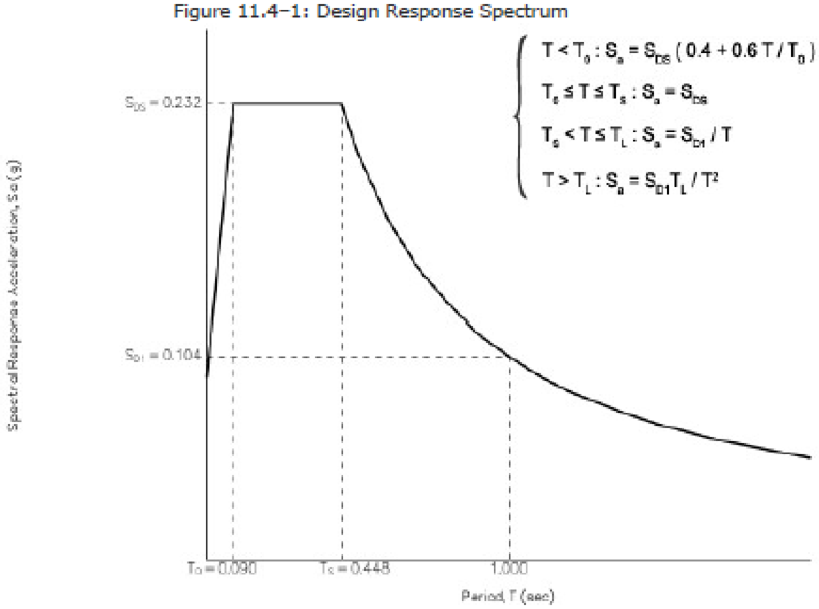
Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3):	$S_{DS} = ⅔ S_{MS} = ⅔ \times 0.349 = 0.232$ g
Equation (11.4–4):	$S_{D1} = ⅔ S_{M1} = ⅔ \times 0.156 = 0.104$ g

Section 11.4.5 — Design Response Spectrum

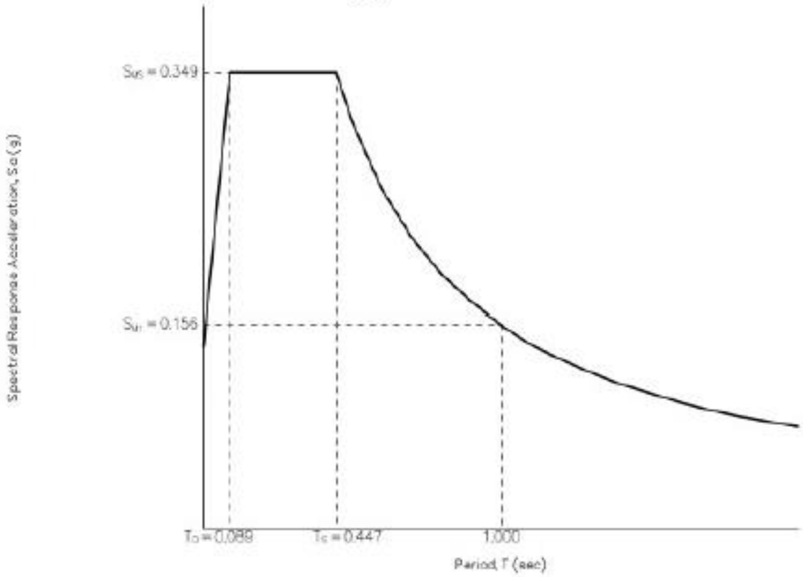
From Figure 22-12 [3]	$T_L = 4$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4]	PGA = 0.115
Equation (11.8-1):	$PGA_M = F_{PGA} PGA = 1.571 \times 0.115 = 0.18$ g

Table 11.8-1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.115 g, F_{PGA} = 1.571

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5]	$C_{RS} = 0.909$
From Figure 22-18 [6]	$C_{R1} = 0.904$

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.232 g, Seismic Design Category = B

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.104 g, Seismic Design Category = B

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = B

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX A.2: BUILDING LOAD REQUIREMENTS

City of Laramie
Code Administration Division

INFORMATIONAL BULLETIN #2

Codes and Design Information

June 2017

PLEASE NOTE: Informational Bulletins should not be used as substitutes for actual codes and regulations. Detailed information regarding codes and regulations can be obtained by calling the Code Administration Division at 307-721-5271.

Building Codes

International Building Code (2015)
International Residential Code (2015)
International Mechanical Code (2015)
International Plumbing Code (2015)
International Fuel Gas Code (2015)
International Fire Code (2015)
International Energy Conservation Code (2015)
National Electrical Code (2017)
Laramie Municipal Code
City of Laramie Standard Details

See the Laramie Municipal Code for amendments to each of the codes.

Design Information

Ground snow load	30 psf
Roof snow load	30 psf
Wind speed (3-second gust)	Figure 1609.3 IBC Exposure Category C
Seismic	Section 1613 IBC Site Class B
Weathering	Severe
Termite	Slight to none
Decay	None to slight
Location	41° 19'N/105° 41'W
Heating degree days	8839
Winter design temperature	-10 degrees F
Air freezing index	1500
Mean annual temperature	40.7 degrees F
Frost depth	42 in.
Rainfall intensity (100 year)	1.8 inches/hour
Ice Shield Underlayment	Yes

Flood hazard:
FIRM Community-Panel Number 560002 0005 D
Map revised May 17, 2017

Building Code

Laramie County has adopted the 2015 Building Codes, effective January 1, 2017, with the following amendments:

- 2015 INTERNATIONAL BUILDING CODE
- 2015 INTERNATIONAL EXISTING BUILDING CODE
- 2015 INTERNATIONAL FIRE CODE
- 2015 INTERNATIONAL FUEL GAS CODE
- 2015 INTERNATIONAL MECHANICAL CODE
- 2015 INTERNATIONAL PLUMBING CODE
- 2015 INTERNATIONAL RESIDENTIAL CODE

At the Laramie County Board of Commissioners public hearing on January 16, 2018, the adoption of the 2017 edition of the National Electrical Code was approved as shown below.

- 2017 National Electrical Code

RESIDENTIAL

Table R301.2(1) CLIMATIC AND GEOGRAPHIC DESIGN CRITERIA

SNOW GROUND LOAD	WIND DESIGN				SEISMIC DESIGN CATEGORY	SUBJECT TO DAMAGE FROM			Winter Design Temp	ICE BARRIER UNDER-LAYMENT REQUIRED	FLOOD HAZARDS	AIR FREEZING INDEX	MEAN ANNUAL TEMP
	SPEED (mph) Vult	Topographic Effects	Special Wind Region	Wind-borne debris zone		Weathering	Frost Line depth	Termite					
30lb/ft^2	115	YES	NO	NO	A-B	SEVERE	36"	NO	-1	NO	2007	2000	46 deg

COMMERCIAL
Design Criteria for all Commercial projects shall be designed in accordance with 2015 IBC Chapter 16.
Ground Snow Load = 30lb/ft^2
Wind Vult = 115 mph
Frost Line Depth = 36"

17 NEC ADOPTED

On January 16, 2018, the Laramie County Commissioners approved the adoption of the 2017 National Electrical Code (17 NEC).

All applications submitted to our office after 01/16/18 shall be reviewed and inspected to the 17 NEC.

Search Results

Query Date: Thu Feb 15 2018
Latitude: 41.3114
Longitude: -105.5911

ASCE 7-10 Windspeeds (3-sec peak gust in mph*):

Risk Category I: 105
Risk Category II: 115
Risk Category III-IV: 120
MRI 10-Year:** 76
MRI 25-Year:** 84
MRI 50-Year:** 90
MRI 100-Year:** 96

ASCE 7-05 Windspeed:
90 (3-sec peak gust in mph)
ASCE 7-93 Windspeed:
87 (fastest mile in mph)

*Miles per hour
**Mean Recurrence Interval

Users should consult with local building officials to determine if there are community-specific wind speed requirements that govern.

Print your results

WINDSPEED WEBSITE DISCLAIMER

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CALCULATIONS:

2nd Floor Beam AB1

Composite Beam Design

Material Properties:

Concrete Strength	**f'_c=**	**4**	ksi
Yield Stress	**F_y=**	50	ksi
Concrete Type	**CT=**	**NWC**	(NWC or LWC)
Slab Thickness	**ST=**	**5**	in.

Calculate Effective Base:

b_e= 45 in.

GUESS:	**Table 3-19**	
Beam Choice	**W18X35**	
Beam Weight	35	plf
(NC) ΦM_n=	249	kip-ft
(LB) ΦM_n=	**416**	kip-ft
(UB) ΦM_n=	**426**	kip-ft
I_x=	510	
(LB) I_x=	1080	$in.^4$
(UB) I_x=	1130	$in.^4$
@Y2 (LB) I_x=	1095	$in.^4$
PNA Location	**BFL**	
(LB) Y2 Location	**4**	in.
(UB) Y2 Location	**4.5**	in.
ΣQn @PNA	**260**	kip

CHECK:			
a=	1.699	in.	
Actual Y2=	4.150	in.	
Actual ΦM_n=	**419**	kip-ft.	$\Phi M_n > M_u$

Check Construction Strength:

M_u =	**86**	kip-ft.	(NC) $\Phi M_n > M_u$

Check Construction Deflection:

Δ DL=	**1.1**	in.	
Camber Limit	1.72	in.	OK

Check In Service Deflections:

Δ LL=	0.73	in.	OK
Δ TL=	1.50	in.	OK

Check Shear:

ΦV_n=	**159**	kips	OK

Span and Applied Loads (Strength-Level)

Loads							
Construction Live Load	**L_C=**	**20**	**psf**				
Construction Dead Load	**D_C=**	**46.97**	**psf**	(Weight of deck+NWC)			
Service Dead Load	**D_S=**	**85.00**	**psf**				
Service Live Load	**L_S=**	**80.00**	**psf**				
Tributary Widths							
Roof/Floor Trib Width	**W_T=**	**3.75**	**ft.**				
Wall Trib Height	**H_T=**	**14.75**	**ft.**	Wall Dead load		**35**	psf
Load							
Factored Dead Load	**W_D=**	**1002**	**plf**				
Factored Live Load	**W_L=**	**480**	**plf**	Area Factor:	**K_{LL}=**	2.00	
Factored Beam Load	**W_u=**	**1482**	**plf**	L_L Reduction	**L_R=**	**N.A.**	
Beam Length							
Length between braced points	**$L_{b.}$ =**	**43**	**ft.**				
Absolute Moment in Unbraced Segment							
Maximum moment	**M_u =**	**343**	**k-ft.**				
Maximum shear	**V_u =**	**32**	**k**				

Deflection Limits

Δ T_L=	L/240	0.179	ft.	2.15	in.
Δ L_L=	L/360	0.119	ft.	1.43	in.
Δ M_L=	L/600	0.072	ft.	0.86	in.

Select Anchorage:			
Final: ΣQn	**260**	kip	**Table 3-21**
Position:	**Weak**	Weak	Strong
Half Span			
Number Of Studs	**16**	(3/4) in TYP.	
Camber	**0**	in.	

2nd Floor Beam AB2

Composite Beam Design

Material Properties:

Concrete Strength	**f'_c=**	**4**	ksi
Yield Stress	**F_y=**	50	ksi
Concrete Type	**CT=**	**NWC**	(NWC or LWC)
Slab Thickness	**ST=**	**5**	in.

Calculate Effective Base:

b_e= 90 in.

GUESS:	**Table 3-19**	
Beam Choice	**W18X35**	
Beam Weight	35	plf
(NC) ΦM_n=	249	kip-ft
(LB) ΦM_n=	**397**	kip-ft
(UB) ΦM_n=	**404**	kip-ft
I_x=	510	
(LB) I_x=	1010	$in.^4$
(UB) I_x=	1050	$in.^4$
@Y2 (LB) I_x=	1025	$in.^4$
PNA Location	**6**	
(LB) Y2 Location	**4.5**	in.
(UB) Y2 Location	**5**	in.
∑Qn @PNA	**194**	kip

CHECK:			
a=	0.634	in.	
Actual Y2=	4.683	in.	
Actual ΦM_n=	**400**	kip-ft.	$\Phi M_n > M_u$

Check Construction Strength:

M_u =	**163**	kip-ft.	(NC) $\Phi M_n > M_u$

Check Construction Deflection:

Δ DL=	**2.0**	in.	Use Camber, Shoring, or Larger Section
Camber Limit	1.72	in.	

Check In Service Deflections:

Δ LL=	1.55	in.	Try Larger Section
Δ TL=	3.20	in.	Consider DL Camber

Check Shear:

ΦV_n=	**159**	kips	OK

Span and Applied Loads (Strength-Level)

Loads							
Construction Live Load	**L_C=**	**20**	**psf**				
Construction Dead Load	**D_C=**	**46.97**	**psf**	(Weight of deck+NWC)			
Service Dead Load	**D_S=**	**85.00**	**psf**				
Service Live Load	**L_S=**	**80.00**	**psf**				
Tributary Widths							
Roof/Floor Trib Width	**W_T=**	**7.5**	**ft.**				
Wall Trib Height	**H_T=**	**0**	**ft.**	Wall Dead load		**35**	psf
Load							
Factored Dead Load	**W_D=**	**765**	**plf**				
Factored Live Load	**W_L=**	**807**	**plf**	Area Factor:	**K_{LL}=**	2.00	
Factored Beam Load	**W_u=**	**1572**	**plf**	L_L Reduction	**L_R=**	**67**	
Beam Length							
Length between braced points	**$L_{b.}$ =**	**43**	**ft.**				
Absolute Moment in Unbraced Segment							
Maximum moment	**M_u =**	**363**	**k-ft.**				
Maximum shear	**V_u =**	**34**	**k**				

Deflection Limits

Δ T_L=	L/240	0.179 ft.	2.15 in.
Δ L_L=	L/360	0.119 ft.	1.43 in.
Δ M_L=	L/600	0.072 ft.	0.86 in.

Select Anchorage:

Final: ∑Qn	**194**	kip	**Table 3-21**
Position:	**Weak**	Weak	Strong
Half Span			
Number Of Studs	**12**	(3/4) in TYP.	
Camber	**1.5**	in.	

2nd Floor Beam BC1

Composite Beam Design

Material Properties:

Concrete Strength	**f'_c=**	**4**	ksi
Yield Stress	**F_y=**	50	ksi
Concrete Type	**CT=**	**NWC**	(NWC or LWC)
Slab Thickness	**ST=**	**5**	in.

Calculate Effective Base:

b_e= 45 in.

GUESS:	**Table 3-19**	
Beam Choice	**W10X22**	
Beam Weight	22	plf
(NC) ΦM_n=	98	kip-ft
(LB) ΦM_n=	**161**	kip-ft
(UB) ΦM_n=	**165**	kip-ft
I_x=	118	
(LB) I_x=	277	$in.^4$
(UB) I_x=	295	$in.^4$
@Y2 (LB) I_x=	281	$in.^4$
PNA Location	**BFL**	
(LB) Y2 Location	**4.5**	in.
(UB) Y2 Location	**5**	in.
∑Qn @PNA	**118**	kip

CHECK:			
a=	0.771	in.	
Actual Y2=	4.614	in.	
Actual ΦM_n=	**162**	kip-ft.	$\Phi M_n > M_u$

Check Construction Strength:

M_u =	**28**	kip-ft.	(NC) $\Phi M_n > M_u$

Check Construction Deflection:

Δ DL=	**0.5**	in.	
Camber Limit	1.00	in.	OK

Check In Service Deflections:

Δ LL=	0.32	in.	OK
Δ TL=	0.67	in.	OK

Check Shear:

ΦV_n=	**73**	kips	OK

Span and Applied Loads (Strength-Level)

Loads						
Construction Live Load	**L_C=**	**20**	**psf**			
Construction Dead Load	**D_C=**	**46.97**	**psf**	(Weight of deck+NWC)		
Service Dead Load	**D_S=**	**85.00**	**psf**			
Service Live Load	**L_S=**	**80.00**	**psf**			
Tributary Widths						
Roof/Floor Trib Width	**W_T=**	**3.75**	**ft.**			
Wall Trib Height	**H_T=**	**14.75**	**ft.**	Wall Dead load	**35**	psf
Load						
Factored Dead Load	**W_D=**	**1002**	**plf**			
Factored Live Load	**W_L=**	**480**	**plf**	Area Factor:	**K_{LL}=**	2.00
Factored Beam Load	**W_u=**	**1482**	**plf**	L_L Reduction	**L_R=**	**N.A.**
Beam Length						
Length between braced points	**$L_{b.}$ =**	**25**	**ft.**			
Absolute Moment in Unbraced Segment						
Maximum moment	**M_u =**	**116**	**k-ft.**			
Maximum shear	**V_u =**	**19**	**k**			

Deflection Limits

Δ T_L=	L/240	0.104	ft.	1.25	in.
Δ L_L=	L/360	0.069	ft.	0.83	in.
Δ M_L=	L/600	0.042	ft.	0.50	in.

Select Anchorage:			
Final: ∑Qn	**118**	kip	**Table 3-21**
Position:	**Weak**	Weak	Strong
Half Span			
Number Of Studs	**7**	(3/4) in TYP.	
Camber	**0**	in.	

2nd Floor Beam BC2

Composite Beam Design

Material Properties:

Concrete Strength	f'_c=	4	ksi
Yield Stress	F_y=	50	ksi
Concrete Type	CT=	NWC	(NWC or LWC)
Slab Thickness	ST=	5	in.

Calculate Effective Base:

b_e= 75 in.

GUESS:	Table 3-19	
Beam Choice	W10X22	
Beam Weight	22	plf
(NC) ΦM_n=	98	kip-ft
(LB) ΦM_n=	161	kip-ft
(UB) ΦM_n=	165	kip-ft
I_x=	118	
(LB) I_x=	277	$in.^4$
(UB) I_x=	295	$in.^4$
@Y2 (LB) I_x=	287	$in.^4$
PNA Location	BFL	
(LB) Y2 Location	4.5	in.
(UB) Y2 Location	5	in.
∑Qn @PNA	118	kip

CHECK:			
a=	0.463	in.	
Actual Y2=	4.769	in.	
Actual ΦM_n=	163	kip-ft.	$\Phi M_n > M_u$

Check Construction Strength:

M_u =	54	kip-ft.	(NC) $\Phi M_n > M_u$

Check Construction Deflection:

Δ DL=	1.0	in.	Use Camber, Shoring, or Larger Section
Camber Limit	1.00	in.	

Check In Service Deflections:

Δ LL=	0.63	in.	OK
Δ TL=	1.31	in.	Consider DL Camber

Check Shear:

ΦV_n=	73	kips	OK

Span and Applied Loads (Strength-Level)

Loads							
Construction Live Load	L_C=	20	psf				
Construction Dead Load	D_C=	46.97	psf	(Weight of deck+NWC)			
Service Dead Load	D_S=	85.00	psf				
Service Live Load	L_S=	80.00	psf				
Tributary Widths							
Roof/Floor Trib Width	W_T=	7.5	ft.				
Wall Trib Height	H_T=	0	ft.	Wall Dead load		35	psf
Load							
Factored Dead Load	W_D=	765	plf				
Factored Live Load	W_L=	960	plf	Area Factor:	K_{LL}=	2.00	
Factored Beam Load	W_u=	1725	plf	L_L Reduction	L_R=	N.A.	
Beam Length							
Length between braced points	$L_{b.}$ =	25	ft.				
Absolute Moment in Unbraced Segment							
Maximum moment	M_u =	135	k-ft.				
Maximum shear	V_u =	22	k				

Deflection Limits

Δ T_L=	L/240	0.104	ft.	1.25	in.
Δ L_L=	L/360	0.069	ft.	0.83	in.
Δ M_L=	L/600	0.042	ft.	0.50	in.

Select Anchorage:				
Final: ∑Qn	118	kip		Table 3-21
Position:	Weak		Weak	Strong
Half Span				
Number Of Studs	7	(3/4) in TYP.		
Camber	0.5	in.		

2nd Floor Spandrel Girder A3-4

Composite Beam Design

Material Properties:

Concrete Strength	**f'_c=**	**4**	ksi
Yield Stress	**F_y=**	50	ksi
Concrete Type	**CT=**	**NWC**	(NWC or LWC)
Slab Thickness	**ST=**	**5**	in.

Calculate Effective Base:

b_e= 90 in.

GUESS:	**Table 3-19**	
Beam Choice	**W21X44**	
Beam Weight	44	plf
(NC) ΦM_n=	358	kip-ft
(LB) ΦM_n=	**576**	kip-ft
(UB) ΦM_n=	**586**	kip-ft
I_x=	843	
(LB) I_x=	1660	$in.^4$
(UB) I_x=	1720	$in.^4$
@Y2 (LB) I_x=	1669	$in.^4$
PNA Location	**6**	
(LB) Y2 Location	**4.5**	in.
(UB) Y2 Location	**5**	in.
∑Qn @PNA	**260**	kip

CHECK:

a=	0.850	in.	
Actual Y2=	4.575	in.	
Actual ΦM_n=	**578**	kip-ft.	$\Phi M_n > M_u$

Check Construction Strength:

M_u =	**220**	kip-ft.	(NC) $\Phi M_n > M_u$

Check Construction Deflection:

Δ DL=	**0.8**	in.	
Camber Limit	1.20	in.	OK

Check In Service Deflections:

Δ LL=	0.65	in.	OK
Δ TL=	1.34	in.	OK

Check Shear:

ΦV_n=	**217**	kips	OK

Span and Applied Loads (Strength-Level)

Loads						
Construction Live Load	**L_C=**	**20**	**psf**			
Construction Dead Load	**D_C=**	**46.97**	**psf**	(Weight of deck+NWC)		
Service Dead Load	**D_S=**	**85.00**	**psf**			
Service Live Load	**L_S=**	**80.00**	**psf**			
Tributary Widths						
Roof/Floor Trib Width	**W_T=**	**21.5**	**ft.**			
Wall Trib Height	**H_T=**	**14.75**	**ft.**	Wall Dead load	**35**	psf
Load						
Factored Dead Load	**W_D=**	**2813**	**plf**			
Factored Live Load	**W_L=**	**1837**	**plf**	Area Factor:	**K_{LL}=**	2.00
Factored Beam Load	**W_u=**	**4650**	**plf**	L_L Reduction	**L_R=**	**53**
Beam Length						
Length between braced points	**$L_{b.}$ =**	**30**	**ft.**			
Absolute Moment in Unbraced Segment						
Maximum moment	**M_u =**	**523**	**k-ft.**			
Maximum shear	**V_u =**	**70**	**k**			

Deflection Limits

Δ T_L=	L/240	0.125	ft.	1.50	in.
Δ L_L=	L/360	0.083	ft.	1.00	in.
Δ M_L=	L/600	0.050	ft.	0.60	in.

Select Anchorage:			
Final: ∑Qn	**260**	kip	**Table 3-21**
Position:	**Weak**	Weak	Strong
Half Span			
Number Of Studs	**16**	(3/4) in TYP.	
Camber	**1.25**	in.	

2nd Floor Girder B3-4

Composite Beam Design

Material Properties:

Concrete Strength	**f'_c=**	**4**	ksi
Yield Stress	**F_y=**	50	ksi
Concrete Type	**CT=**	**NWC**	(NWC or LWC)
Slab Thickness	**ST=**	**5**	in.

Calculate Effective Base:

b_e=	90	in.

GUESS:	**Table 3-19**	
Beam Choice	**W21X55**	
Beam Weight	55	plf
(NC) ΦM_n=	473	kip-ft
(LB) ΦM_n=	**752**	kip-ft
(UB) ΦM_n=	**766**	kip-ft
I_x=	1140	
(LB) I_x=	2290	$in.^4$
(UB) I_x=	2370	$in.^4$
@Y2 (LB) I_x=	2270	$in.^4$
PNA Location	**BFL**	
(LB) Y2 Location	**4.5**	in.
(UB) Y2 Location	**5**	in.
ΣQn @PNA	**381**	kip

CHECK:

a=	1.245	in.	
Actual Y2=	4.377	in.	
Actual ΦM_n=	**749**	kip-ft.	$\Phi M_n > M_u$

Check Construction Strength:

M_u =	**335**	kip-ft.	(NC) $\Phi M_n > M_u$

Check Construction Deflection:

Δ DL=	**0.9**	in.	
Camber Limit	1.20	in.	OK

Check In Service Deflections:

Δ LL=	0.73	in.	OK
Δ TL=	1.51	in.	Consider DL Camber

Check Shear:

ΦV_n=	**234**	kips	OK

Span and Applied Loads (Strength-Level)

Loads							
Construction Live Load	**L_C=**	**20**	**psf**				
Construction Dead Load	**D_C=**	**46.97**	**psf**	(Weight of deck+NWC)			
Service Dead Load	**D_S=**	**85.00**	**psf**				
Service Live Load	**L_S=**	**80.00**	**psf**				
Tributary Widths							
Roof/Floor Trib Width	**W_T=**	**33**	**ft.**				
Wall Trib Height	**H_T=**	**0**	**ft.**	Wall Dead load		**35**	psf
Load							
Factored Dead Load	**W_D=**	**3366**	**plf**				
Factored Live Load	**W_L=**	**2480**	**plf**	Area Factor:	**K_{LL}=**	2.00	
Factored Beam Load	**W_u=**	**5846**	**plf**	L_L Reduction	**L_R=**	**47**	
Beam Length							
Length between braced points	**$L_{b.}$ =**	**30**	**ft.**				
Absolute Moment in Unbraced Segment							
Maximum moment	**M_u =**	**658**	**k-ft.**				
Maximum shear	**V_u =**	**88**	**k**				

Deflection Limits

Δ T_L=	L/240	0.125	ft.	1.50	in.
Δ L_L=	L/360	0.083	ft.	1.00	in.
Δ M_L=	L/600	0.050	ft.	0.60	in.

Select Anchorage:

Final: ΣQn	**381**	kip	**Table 3-21**
Position:	**Weak**	Weak	Strong
Half Span			
Number Of Studs	**23**	(3/4) in TYP.	
Camber	**0**	in.	

Mech Bay Girder B3-4

Composite Beam Design

Material Properties:

Concrete Strength	f'_c=	4	ksi
Yield Stress	F_y=	50	ksi
Concrete Type	CT=	NWC	(NWC or LWC)
Slab Thickness	ST=	5	in.

Calculate Effective Base:

b_e= 90 in.

GUESS:	Table 3-19	
Beam Choice	W24X62	
Beam Weight	62	plf
(NC) ΦM_n=	574	kip-ft
(LB) ΦM_n=	948	kip-ft
(UB) ΦM_n=	967	kip-ft
I_x=	1550	
(LB) I_x=	3160	$in.^4$
(UB) I_x=	3260	$in.^4$
@Y2 (LB) I_x=	3198	$in.^4$
PNA Location	BFL	
(LB) Y2 Location	4	in.
(UB) Y2 Location	4.5	in.
∑Qn @PNA	495	kip

CHECK:			
a=	1.618	in.	
Actual Y2=	4.191	in.	
Actual ΦM_n=	955	kip-ft.	$\Phi M_n > M_u$

Check Construction Strength:

M_u =	336	kip-ft.	(NC) $\Phi M_n > M_u$

Check Construction Deflection:

Δ DL=	0.7	in.	
Camber Limit	1.20	in.	OK

Check In Service Deflections:

Δ LL=	0.52	in.	OK
Δ TL=	1.49	in.	OK

Check Shear:

ΦV_n=	306	kips	OK

Span and Applied Loads (Strength-Level)

Loads							
Construction Live Load	L_C=	20	psf				
Construction Dead Load	D_C=	46.97	psf	(Weight of deck+NWC)			
Service Dead Load	D_S=	150.00	psf				
Service Live Load	L_S=	80.00	psf				
Tributary Widths							
Roof/Floor Trib Width	W_T=	33	ft.				
Wall Trib Height	H_T=	0	ft.	Wall Dead load		35	psf
Load							
Factored Dead Load	W_D=	5940	plf				
Factored Live Load	W_L=	2480	plf	Area Factor:	K_{LL}=	2.00	
Factored Beam Load	W_u=	8420	plf	L_L Reduction	L_R=	47	
Beam Length							
Length between braced points	$L_{b.}$ =	30	ft.				
Absolute Moment in Unbraced Segment							
Maximum moment	M_u =	947	k-ft.				
Maximum shear	V_u =	126	k				

Deflection Limits

Δ T_L=	L/240	0.125	ft.	1.50	in.
Δ L_L=	L/360	0.083	ft.	1.00	in.
Δ M_L=	L/600	0.050	ft.	0.60	in.

Select Anchorage:

Final: ∑Qn	495	kip	Table 3-21
Position:	Weak	Weak	Strong
Half Span			
Number Of Studs	29	(3/4) in TYP.	
Camber	0	in.	

Mech Bay Beam BC3

Composite Beam Design

Material Properties:

Concrete Strength	**f'_c=**	**4**	ksi
Yield Stress	**F_y=**	50	ksi
Concrete Type	**CT=**	**NWC**	(NWC or LWC)
Slab Thickness	**ST=**	**5**	in.

Calculate Effective Base:

b_e= 75 in.

GUESS:	**Table 3-19**	
Beam Choice	**W12X22**	
Beam Weight	22	plf
(NC) ΦM_n=	110	kip-ft
(LB) ΦM_n=	**198**	kip-ft
(UB) ΦM_n=	**204**	kip-ft
I_x=	156	
(LB) I_x=	414	$in.^4$
(UB) I_x=	438	$in.^4$
@Y2 (LB) I_x=	424	$in.^4$
PNA Location	**BFL**	
(LB) Y2 Location	**4.5**	in.
(UB) Y2 Location	**5**	in.
∑Qn @PNA	**153**	kip

CHECK:			
a=	0.600	in.	
Actual Y2=	4.700	in.	
Actual ΦM_n=	**200**	kip-ft.	$\Phi M_n > M_u$

Check Construction Strength:

M_u =	**54**	kip-ft.	(NC) $\Phi M_n > M_u$

Check Construction Deflection:

Δ DL=	**0.7**	in.	
Camber Limit	1.00	in.	OK

Check In Service Deflections:

Δ LL=	0.43	in.	OK
Δ TL=	1.23	in.	OK

Check Shear:

ΦV_n=	**96**	kips	OK

Span and Applied Loads (Strength-Level)

Loads							
Construction Live Load	**L_C=**	**20**	**psf**				
Construction Dead Load	**D_C=**	**46.97**	**psf**	(Weight of deck+NWC)			
Service Dead Load	**D_S=**	**150.00**	**psf**				
Service Live Load	**L_S=**	**80.00**	**psf**				
Tributary Widths							
Roof/Floor Trib Width	**W_T=**	**7.5**	**ft.**				
Wall Trib Height	**H_T=**	**0**	**ft.**	Wall Dead load		**35**	psf
Load							
Factored Dead Load	**W_D=**	**1350**	**plf**				
Factored Live Load	**W_L=**	**960**	**plf**	Area Factor:	**K_{LL}=**	2.00	
Factored Beam Load	**W_u=**	**2310**	**plf**	L_L Reduction	**L_R=**	**N.A.**	
Beam Length							
Length between braced points	**$L_{b.}$ =**	**25**	**ft.**				
Absolute Moment in Unbraced Segment							
Maximum moment	**M_u =**	**180**	**k-ft.**				
Maximum shear	**V_u =**	**29**	**k**				

Deflection Limits

Δ T_L=	L/240	0.104	ft.	1.25	in.
Δ L_L=	L/360	0.069	ft.	0.83	in.
Δ M_L=	L/600	0.042	ft.	0.50	in.

Select Anchorage:			
Final: ∑Qn	**153**	kip	**Table 3-21**
Position:	**Weak**	Weak	Strong
Half Span			
Number Of Studs	**9**	(3/4) in TYP.	
Camber	**0**	in.	

Spandrel Girder A3-4

Composite Beam Design

Material Properties:

Concrete Strength	**f'_c=**	**4**	ksi
Yield Stress	**F_y=**	50	ksi
Concrete Type	**CT=**	**NWC**	(NWC or LWC)
Slab Thickness	**ST=**	**5**	in.

Calculate Effective Base:

b_e= 90 in.

GUESS:	**Table 3-19**	
Beam Choice	**W21X44**	
Beam Weight	44	plf
(NC) ΦM_n=	358	kip-ft
(LB) ΦM_n=	**576**	kip-ft
(UB) ΦM_n=	**586**	kip-ft
I_x=	843	
(LB) I_x=	1660	$in.^4$
(UB) I_x=	1720	$in.^4$
@Y2 (LB) I_x=	1669	$in.^4$
PNA Location	**6**	
(LB) Y2 Location	**4.5**	in.
(UB) Y2 Location	**5**	in.
ΣQn @PNA	**260**	kip

CHECK:			
a=	0.850	in.	
Actual Y2=	4.575	in.	
Actual ΦM_n=	**578**	kip-ft.	$\Phi M_n > M_u$

Check Construction Strength:

M_u =	**220**	kip-ft.	(NC) $\Phi M_n > M_u$

Check Construction Deflection:

Δ DL=	**0.8**	in.	
Camber Limit	1.20	in.	OK

Check In Service Deflections:

Δ LL=	0.65	in.	OK
Δ TL=	1.34	in.	OK

Check Shear:

ΦV_n=	**217**	kips	OK

Span and Applied Loads (Strength-Level)

Loads							
Construction Live Load	**L_C=**	**20**	**psf**				
Construction Dead Load	**D_C=**	**46.97**	**psf**	(Weight of deck+NWC)			
Service Dead Load	**D_S=**	**85.00**	**psf**				
Service Live Load	**L_S=**	**80.00**	**psf**				
Tributary Widths							
Roof/Floor Trib Width	**W_T=**	**21.5**	**ft.**				
Wall Trib Height	**H_T=**	**14.75**	**ft.**	Wall Dead load		**35**	psf
Load							
Factored Dead Load	**W_D=**	**2813**	**plf**				
Factored Live Load	**W_L=**	**1837**	**plf**	Area Factor:	**K_{LL}=**	2.00	
Factored Beam Load	**W_u=**	**4650**	**plf**	L_L Reduction	**L_R=**	**53**	
Beam Length							
Length between braced points	**$L_{b.}$ =**	**30**	**ft.**				
Absolute Moment in Unbraced Segment							
Maximum moment	**M_u =**	**523**	**k-ft.**				
Maximum shear	**V_u =**	**70**	**k**				

Deflection Limits

Δ T_L=	L/240	0.125	ft.	1.50	in.
Δ L_L=	L/360	0.083	ft.	1.00	in.
Δ M_L=	L/600	0.050	ft.	0.60	in.

Select Anchorage:

Final: ΣQn	**260**	kip	**Table 3-21**
Position:	**Weak**	Weak	Strong
Half Span			
Number Of Studs	**16**	(3/4) in TYP.	
Camber	**1.25**	in.	

Exterior Corner Column A1

Column Selection

W-Shape Properties	=	**W14X48**	
Weight	**W =**	48	**lb/ft**
Cross-Sectional Area	**A =**	14.1	**$in.^2$**
Depth	**d =**	13.8	**in.**
Web thickness	**t_w =**	0.34	**in.**
Flange width	**b_f =**	8.03	**in.**
Flange thickness	**t_f =**	0.595	**in.**
X Axis Properties			
Moment of Inertia	**I_x =**	484	**$in.^4$**
Elastic section modulus	**S_x =**	70.2	**$in.^3$**
Radius of gyration	**r_x =**	5.85	**in.**
Plastic section modulus	**Z_x =**	78.4	**$in.^3$**
Y Axis Properties			
Moment of Inertia	**I_y =**	51.4	**$in.^4$**
Elastic section modulus	**S_y =**	12.8	**$in.^3$**
Radius of gyration	**r_y =**	1.91	**in.**
Plastic section modulus	**Z_y =**	19.6	**$in.^3$**
Other Properties			
Effective radius of gyration	**r_{ts} =**	2.2	**in.**
Flange centroid distance	**h_o =**	13.2	**in.**
Torsional moment of inertia	**J =**	1.45	**$in.^4$**
Warping-torsion constant	**C_w =**	2240	**$in.^6$**

Material Selection

ASTM	=	**A992**	
Type	=	High-Strength, Low-Alloy	
Modulus of Elasticity	**E =**	29000	**ksi**
Shear Modulus of Elasticity	**G =**	11200	**ksi**
Min. yield tensile stress	**F_y =**	50	**ksi**
Min. ultimate tensile stress	**F_u =**	65	**ksi**

Loads

Dead Load	**P_D =**	126	**k**
Live Load	**P_L =**	34	**k**
Snow Load	**P_S =**	9.675	**k**
Roof Live Load	**P_{Lr} =**	9.675	**k**
Wind Load	**P_W =**	182	**k**

Applied Loads (Strength-Level)

Loads			
Snow	**S =**	30	**psf**
Roof Live	**L_r =**	30	**psf**
Floor Live	**L_0 =**	80	**psf**
Roof Dead	**D_r =**	47	**psf**
Floor Dead	**D_f =**	85	**psf**
Wall Dead	**D_w =**	35	**psf**
Wind	**W =**	182	**k**
Tributary Area			
Roof/Floor Trib Area	**A_T =**	322.5	**ft^2**
Wall Trib Area	**A_w =**	1606	**ft^2**
Number of Floors			
Roof		1	**floors**
Floors		2	**floors**

Live Load Reduction

Factors			
	K_{LL} =	4	
	$K_{LL}A_T$ =	1290	
Checks			
	$L_0 < 100$	YES	
	$K_{LL}A_T > 400$	YES	
Reduction Factor (RF)			
	RF =	0.6676	
	RF_{min} =	0.4	
Reduced Live Load			
	L =	53	**psf**

Load Combinations

1	**P =**	176.7	**k**
2	**P =**	211.4	**k**
3	**P =**	257.9	**k**
4	**P =**	355.5	**k**
6	**P =**	295.6	**k**
Control	**P_u =**	**355.5**	**k**

Local Buckling

Flange				
Width-To-Thickness Ratio	**$b_f/2t_f$ =**	6.75		**Nonslender**
Ratio Limit	**λ_r=**	13.49		
Unstiffened Reduction Factor	**Q_s =**	1.00		
Web				
Height-To-Thickness Ratio	**h/t_w =**	33.60		**Nonslender**
Ratio Limit	**λ_r=**	35.88		
Effective Height	**b_e =**	11.42	**in.**	(Approximate)
Effective Area	**A_e =**	14.10	**$in.^2$**	
Stiffened Reduction Factor	**Q_a =**	1.00		
Whole Member				
Strength Reduction Factor	**Q =**	1.00		

Flexural Buckling

Compression Members				**Controls**
Effective Length Factor	**K =**	1		
Unbraced Length	**L_b =**	13	**ft.**	
Unbraced Length	**L_b =**	156	**in.**	
Calculations				
Factored Compressive Load	**P_u =**	355	**k**	
Design Compressive Strength	**ΦP_n =**	389.58	**k**	**Okay**
Slenderness Ratio	**KL/r_y =**	81.68		**Okay**
Elastic Buckling Stress	**F_e =**	42.91	**ksi**	
Limiting Ratio (Nonslender)	**F_y/F_e =**	1.17		
Critical Stress (Nonslender)	**F_{cr} =**	30.70		
Limiting Ratio	**$^{(Q)}F_y/F_e$ =**	1.17	**<2.25?**	**Inelastic**
Critical Stress	**$^{(Q)}F_{cr}$ =**	30.70	**ksi**	

Torsional Buckling

Compression Members				**Doesn't Control**
Effective Length Factor	**K_z =**	1		
Unbraced Length	**L_b =**	13	**ft.**	
Unbraced Length	**L_b =**	156	**in.**	
Calculations				
Factored Compressive Load	**P_u =**	355	**k**	
Design Compressive Strength	**ΦP_n =**	632.64	**k**	**Okay**
Slenderness Ratio	**KL/r_y =**	81.68		**Okay**
Elastic Buckling Stress	**F_e =**	7116.00	**ksi**	
Limiting Ratio (Nonslender)	**F_y/F_e =**	0.01		
Critical Stress (Nonslender)	**F_{cr} =**	49.85		
Limiting Ratio	**$^{(Q)}F_y/F_e$ =**	0.01	**<2.25?**	**Inelastic**
Critical Stress	**$^{(Q)}F_{cr}$ =**	49.85	**ksi**	

Base Plate Design

Loading - Plate:

P_D=		kips
P_L=		kips
OR: P_u=	356	kips
F_y=	36	ksi
F_u=	58	ksi
F'_C=	3	ksi

Column Selection **W14X48**

d=	13.8	in.
b_f=	8.03	in.
t_f=	0.595	in.
t_w=	0.34	in.
F_y=	50	ksi
F_u=	65	ksi

LRFD Factored Load

P_u= 356 kips

Plate Design

A1 RQD:	215	$in.^2$
Base b=	15	in.
TRY b=	**20**	in
A1=	400	$in.^2$
Anchor check:	19.8	**<b ok**
A2=	484	$in.^2$
A2/A1=	1.2	**<4 ok**
φcPp=	729	**OK**

m=	3.45	in.
n=	6.79	in.
n'=	2.63	in.
X=	0.454	
λ=	0.78	**<=1**
λn'=	2.04	
L=	6.79	in
f_{pu}=	0.9	ksi
t_{min}=	**1.59**	in.

Coulmn: A1 **USE:** **PL1.75 x 20in. X 20in.**

Exterior Column A3

Column Selection

W-Shape Properties	=	**W14X61**	
Weight	**W =**	61	**lb/ft**
Cross-Sectional Area	**A =**	17.9	**in.^{2}**
Depth	**d =**	13.9	**in.**
Web thickness	**t_w =**	0.375	**in.**
Flange width	**b_f =**	10	**in.**
Flange thickness	**t_f =**	0.645	**in.**
X Axis Properties			
Moment of Inertia	**I_x =**	640	**in.^{4}**
Elastic section modulus	**S_x =**	92.1	**in.^{3}**
Radius of gyration	**r_x =**	5.98	**in.**
Plastic section modulus	**Z_x =**	102	**in.^{3}**
Y Axis Properties			
Moment of Inertia	**I_y =**	107	**in.^{4}**
Elastic section modulus	**S_y =**	21.5	**in.^{3}**
Radius of gyration	**r_y =**	2.45	**in.**
Plastic section modulus	**Z_y =**	32.8	**in.^{3}**
Other Properties			
Effective radius of gyration	**r_{ts} =**	2.78	**in.**
Flange centroid distance	**h_o =**	13.3	**in.**
Torsional moment of inertia	**J =**	2.19	**in.^{4}**
Warping-torsion constant	**C_w =**	4710	**in.^{6}**

Material Selection

ASTM	=	**A992**	
Type	=	High-Strength, Low-Alloy	
Modulus of Elasticity	**E =**	29000	**ksi**
Shear Modulus of Elasticity	**G =**	11200	**ksi**
Min. yield tensile stress	**F_y =**	50	**ksi**
Min. ultimate tensile stress	**F_u =**	65	**ksi**

Loads

Dead Load	**P_D =**	186	**k**
Live Load	**P_L =**	56	**k**
Snow Load	**P_S =**	19.35	**k**
Roof Live Load	**P_{Lr} =**	19.35	**k**
Wind Load	**P_W =**	182	**k**

Applied Loads (Strength-Level)

Loads			
Snow	**S =**	30	**psf**
Roof Live	**L_r =**	30	**psf**
Floor Live	**L_0 =**	80	**psf**
Roof Dead	**D_r =**	47	**psf**
Floor Dead	**D_f =**	85	**psf**
Wall Dead	**D_w =**	35	**psf**
Wind	**W =**	182	**k**
Tributary Area			
Roof/Floor Trib Area	**A_T =**	645	**ft^2**
Wall Trib Area	**A_w =**	1320	**ft^2**
Number of Floors			
Roof		1	**floors**
Floors		2	**floors**

Live Load Reduction

Factors			
	K_{LL} =	4	
	$K_{LL}A_T$ =	2580	
Checks			
	L_0 < 100	YES	
	$K_{LL}A_T$ > 400	YES	
Reduction Factor (RF)			
	RF =	0.5453	
	RF_{min} =	0.4	
Reduced Live Load			
	L =	44	**psf**

Load Combinations

1	**P =**	260.6	**k**
2	**P =**	323.1	**k**
3	**P =**	345.4	**k**
4	**P =**	443.2	**k**
6	**P =**	349.5	**k**
Control	**P_u =**	**443.2**	**k**

Local Buckling

Flange				
Width-To-Thickness Ratio	**$b_f/2t_f$ =**	7.75		**Nonslender**
Ratio Limit	**λ_r=**	13.49		
Unstiffened Reduction Factor	**Q_s =**	1.00		
Web				
Height-To-Thickness Ratio	**h/t_w =**	30.40		**Nonslender**
Ratio Limit	**λ_r=**	35.88		
Effective Height	**b_e =**	11.40	**in.**	(Approximate)
Effective Area	**A_e =**	17.90	**in.^{2}**	
Stiffened Reduction Factor	**Q_a =**	1.00		
Whole Member				
Strength Reduction Factor	**Q =**	1.00		

Flexural Buckling

Compression Members				**Controls**
Effective Length Factor	**K =**	1		
Unbraced Length	**L_b =**	13	**ft.**	
Unbraced Length	**L_b =**	156	**in.**	
Calculations				
Factored Compressive Load	**P_u =**	443	**k**	
Design Compressive Strength	**ΦP_n =**	598.86	**k**	**Okay**
Slenderness Ratio	**KL/r_y =**	63.67		**Okay**
Elastic Buckling Stress	**F_e =**	70.60	**ksi**	
Limiting Ratio (Nonslender)	**F_y/F_e =**	0.71		
Critical Stress (Nonslender)	**F_{cr} =**	37.17		
Limiting Ratio	**$^{(Q)}F_y/F_e$ =**	0.71	**<2.25?**	**Inelastic**
Critical Stress	**$^{(Q)}F_{cr}$ =**	37.17	**ksi**	

Torsional Buckling

Compression Members				**Doesn't Control**
Effective Length Factor	**K_z =**	1		
Unbraced Length	**L_b =**	13	**ft.**	
Unbraced Length	**L_b =**	156	**in.**	
Calculations				
Factored Compressive Load	**P_u =**	443	**k**	
Design Compressive Strength	**ΦP_n =**	803.93	**k**	**Okay**
Slenderness Ratio	**KL/r_y =**	63.67		**Okay**
Elastic Buckling Stress	**F_e =**	10711.36	**ksi**	
Limiting Ratio (Nonslender)	**F_y/F_e =**	0.00		
Critical Stress (Nonslender)	**F_{cr} =**	49.90		
Limiting Ratio	**$^{(Q)}F_y/F_e$ =**	0.00	**<2.25?**	**Inelastic**
Critical Stress	**$^{(Q)}F_{cr}$ =**	49.90	**ksi**	

Base Plate Design

Loading - Plate:

P_D=		kips
P_L=		kips
OR: P_u=	444	kips
F_y=	36	ksi
F_u=	58	ksi
F'_C=	3	ksi

Column Selection **W14X61**

d=	13.9	in.
b_f=	10	in.
t_f=	0.645	in.
t_w=	0.375	in.
F_y=	50	ksi
F_u=	65	ksi

LRFD Factored Load

P_u= 444 kips

Plate Design

A1 RQD:	268	in.^{2}
Base b=	17	in.
TRY b=	**20**	in
A1=	400	in.^{2}
Anchor check:	19.9	**<b ok**
A2=	484	in.^{2}
A2/A1=	1.2	**<4 ok**
φcPp=	729	**OK**

m=	3.40	in.
n=	6.00	in.
n'=	2.95	in.
X=	0.593	
λ=	0.94	**<=1**
λn'=	2.77	
L=	6.00	in
f_{pu}=	1.1	ksi
t_{min}=	**1.57**	in.

Coulmn: A3 **USE:** **PL1.75in. x 20in. X 20in.**

Exterior Column B1

Column Selection

W-Shape Properties	=	W14X61	
Weight	$W =$	61	lb/ft
Cross-Sectional Area	$A =$	17.9	in.^{2}
Depth	$d =$	13.9	in.
Web thickness	$t_w =$	0.375	in.
Flange width	$b_f =$	10	in.
Flange thickness	$t_f =$	0.645	in.
X Axis Properties			
Moment of Inertia	$I_x =$	640	in.^{4}
Elastic section modulus	$S_x =$	92.1	in.^{3}
Radius of gyration	$r_x =$	5.98	in.
Plastic section modulus	$Z_x =$	102	in.^{3}
Y Axis Properties			
Moment of Inertia	$I_y =$	107	in.^{4}
Elastic section modulus	$S_y =$	21.5	in.^{3}
Radius of gyration	$r_y =$	2.45	in.
Plastic section modulus	$Z_y =$	32.8	in.^{3}
Other Properties			
Effective radius of gyration	$r_{ts} =$	2.78	in.
Flange centroid distance	$h_o =$	13.3	in.
Torsional moment of inertia	$J =$	2.19	in.^{4}
Warping-torsion constant	$C_w =$	4710	in.^{6}

Material Selection

ASTM	=	A992	
Type	=	High-Strength, Low-Alloy	
Modulus of Elasticity	$E =$	29000	ksi
Shear Modulus of Elasticity	$G =$	11200	ksi
Min. yield tensile stress	$F_y =$	50	ksi
Min. ultimate tensile stress	$F_u =$	65	ksi

Loads

Dead Load	$P_D =$	216	k
Live Load	$P_L =$	47	k
Snow Load	$P_S =$	15.3	k
Roof Live Load	$P_{Lr} =$	15.3	k
Wind Load	$P_W =$	182	k

Applied Loads (Strength-Level)

Loads			
Snow	$S =$	30	psf
Roof Live	$L_r =$	30	psf
Floor Live	$L_0 =$	80	psf
Roof Dead	$D_r =$	150	psf
Floor Dead	$D_f =$	85	psf
Wall Dead	$D_w =$	35	psf
Wind	$W =$	182	k
Tributary Area			
Roof/Floor Trib Area	$A_T =$	510	ft^2
Wall Trib Area	$A_w =$	1496	ft^2
Number of Floors			
Roof		1	floors
Floors		2	floors

Live Load Reduction

Factors		
	$K_{LL} =$	4
	$K_{LL}A_T =$	2040
Checks		
	$L_0 < 100$	YES
	$K_{LL}A_T > 400$	YES
Reduction Factor (RF)		
	$RF =$	0.5821
	$RF_{min} =$	0.4
Reduced Live Load		
	$L =$	47 psf

Load Combinations

1	$P =$	301.8	k
2	$P =$	342.3	k
3	$P =$	374.2	k
4	$P =$	472.1	k
6	$P =$	376.0	k
Control	$P_u =$	**472.1**	**k**

Local Buckling

Flange				
Width-To-Thickness Ratio	$b_f/2t_f =$	7.75		Nonslender
Ratio Limit	$\lambda_r =$	13.49		
Unstiffened Reduction Factor	$Q_s =$	1.00		
Web				
Height-To-Thickness Ratio	$h/t_w =$	30.40		Nonslender
Ratio Limit	$\lambda_r =$	35.88		
Effective Height	$b_e =$	11.40	in.	(Approximate)
Effective Area	$A_e =$	17.90	in.^{2}	
Stiffened Reduction Factor	$Q_a =$	1.00		
Whole Member				
Strength Reduction Factor	$Q =$	1.00		

Flexural Buckling

Compression Members				Controls
Effective Length Factor	$K =$	1		
Unbraced Length	$L_b =$	13	ft.	
Unbraced Length	$L_b =$	156	in.	
Calculations				
Factored Compressive Load	$P_u =$	472	k	
Design Compressive Strength	$\Phi P_n =$	598.86	k	Okay
Slenderness Ratio	$KL/r_y =$	63.67		Okay
Elastic Buckling Stress	$F_e =$	70.60	ksi	
Limiting Ratio (Nonslender)	$F_y/F_e =$	0.71		
Critical Stress (Nonslender)	$F_{cr} =$	37.17		
Limiting Ratio	$^{(Q)}F_y/F_e =$	0.71	<2.25?	Inelastic
Critical Stress	$^{(Q)}F_{cr} =$	37.17	ksi	

Torsional Buckling

Compression Members				Doesn't Control
Effective Length Factor	$K_z =$	1		
Unbraced Length	$L_b =$	13	ft.	
Unbraced Length	$L_b =$	156	in.	
Calculations				
Factored Compressive Load	$P_u =$	472	k	
Design Compressive Strength	$\Phi P_n =$	803.93	k	Okay
Slenderness Ratio	$KL/r_y =$	63.67		Okay
Elastic Buckling Stress	$F_e =$	10711.36	ksi	
Limiting Ratio (Nonslender)	$F_y/F_e =$	0.00		
Critical Stress (Nonslender)	$F_{cr} =$	49.90		
Limiting Ratio	$^{(Q)}F_y/F_e =$	0.00	<2.25?	Inelastic
Critical Stress	$^{(Q)}F_{cr} =$	49.90	ksi	

Base Plate Design

Loading - Plate:			Column Selection	**W14X61**	
P_D=		kips	d=	13.9	in.
P_L=		kips	b_f=	10	in.
OR: P_u=	472	kips	t_f=	0.645	in.
F_y=	36	ksi	t_w=	0.375	in.
F_u=	58	ksi	F_y=	50	ksi
F'_C=	3	ksi	F_u=	65	ksi

LRFD Factored Load

P_u=	472	kips

Plate Design

A1 RQD:	285	$in.^2$	m=	3.40	in.
Base b=	17	in.	n=	6.00	in.
			n'=	2.95	in.
TRY b=	**20**	in			
A1=	400	$in.^2$	X=	0.630	
Anchor check:	19.9	**<b ok**	λ=	0.99	**<=1**
			λn'=	2.91	
A2=	484	$in.^2$			
A2/A1=	1.2	**<4 ok**	L=	6.00	in
φcPp=	729	**OK**	f_{pu}=	1.2	ksi
			t_{min}=	**1.62**	in.

Coulmn: B1 **USE:** **PL1.75in. x 20in. X 20in.**

Exterior Column B3

Column Selection

W-Shape Properties	=	**W14X68**	
Weight	**W =**	68	**lb/ft**
Cross-Sectional Area	**A =**	20	**in.^{2}**
Depth	**d =**	14	**in.**
Web thickness	**t_w =**	0.415	**in.**
Flange width	**b_f =**	10	**in.**
Flange thickness	**t_f =**	0.72	**in.**
X Axis Properties			
Moment of Inertia	**I_x =**	722	**in.^{4}**
Elastic section modulus	**S_x =**	103	**in.^{3}**
Radius of gyration	**r_x =**	6.01	**in.**
Plastic section modulus	**Z_x =**	115	**in.^{3}**
Y Axis Properties			
Moment of Inertia	**I_y =**	121	**in.^{4}**
Elastic section modulus	**S_y =**	24.2	**in.^{3}**
Radius of gyration	**r_y =**	2.46	**in.**
Plastic section modulus	**Z_y =**	36.9	**in.^{3}**
Other Properties			
Effective radius of gyration	**r_{ts} =**	2.8	**in.**
Flange centroid distance	**h_o =**	13.3	**in.**
Torsional moment of inertia	**J =**	3.01	**in.^{4}**
Warping-torsion constant	**C_w =**	5380	**in.^{6}**

Material Selection

ASTM	=	**A992**	
Type	=	High-Strength, Low-Alloy	
Modulus of Elasticity	**E =**	29000	**ksi**
Shear Modulus of Elasticity	**G =**	11200	**ksi**
Min. yield tensile stress	**F_y =**	50	**ksi**
Min. ultimate tensile stress	**F_u =**	65	**ksi**

Loads

Dead Load	**P_D =**	317	**k**
Live Load	**P_L =**	77	**k**
Snow Load	**P_S =**	29.7	**k**
Roof Live Load	**P_{Lr} =**	29.7	**k**
Wind Load	**P_W =**	182	**k**

Applied Loads (Strength-Level)

Loads			
Snow	**S =**	30	**psf**
Roof Live	**L_r =**	30	**psf**
Floor Live	**L_0 =**	80	**psf**
Roof Dead	**D_r =**	150	**psf**
Floor Dead	**D_f =**	85	**psf**
Wall Dead	**D_w =**	35	**psf**
Wind	**W =**	182	**k**
Tributary Area			
Roof/Floor Trib Area	**A_T =**	990	**ft^2**
Wall Trib Area	**A_w =**	0	**ft^2**
Number of Floors			
Roof		1	**floors**
Floors		2	**floors**

Live Load Reduction

Factors			
	K_{LL} =	4	
	$K_{LL}A_T$ =	3960	
Checks			
	L_0 < 100	YES	
	$K_{LL}A_T$ > 400	YES	
Reduction Factor (RF)			
	RF =	0.4884	
	RF_{min} =	0.4	
Reduced Live Load			
	L =	39	**psf**

Load Combinations

1	**P =**	443.5	**k**
2	**P =**	518.8	**k**
3	**P =**	518.7	**k**
4	**P =**	615.7	**k**
6	**P =**	467.1	**k**
Control	**P_u =**	**615.7**	**k**

Local Buckling

Flange				
Width-To-Thickness Ratio	**$b_f/2t_f$ =**	6.97		**Nonslender**
Ratio Limit	**λ_r=**	13.49		
Unstiffened Reduction Factor	**Q_s =**	1.00		
Web				
Height-To-Thickness Ratio	**h/t_w =**	27.50		**Nonslender**
Ratio Limit	**λ_r=**	35.88		
Effective Height	**b_e =**	11.41	**in.**	(Approximate)
Effective Area	**A_e =**	20.00	**in.^{2}**	
Stiffened Reduction Factor	**Q_a =**	1.00		
Whole Member				
Strength Reduction Factor	**Q =**	1.00		

Flexural Buckling

Compression Members				**Controls**
Effective Length Factor	**K =**	1		
Unbraced Length	**L_b =**	13	**ft.**	
Unbraced Length	**L_b =**	156	**in.**	
Calculations				
Factored Compressive Load	**P_u =**	616	**k**	
Design Compressive Strength	**ΦP_n =**	670.73	**k**	**Okay**
Slenderness Ratio	**KL/r_y =**	63.41		**Okay**
Elastic Buckling Stress	**F_e =**	71.17	**ksi**	
Limiting Ratio (Nonslender)	**F_y/F_e =**	0.70		
Critical Stress (Nonslender)	**F_{cr} =**	37.26		
Limiting Ratio	**$^{(Q)}F_y/F_e$ =**	0.70	**<2.25?**	**Inelastic**
Critical Stress	**$^{(Q)}F_{cr}$ =**	37.26	**ksi**	

Torsional Buckling

Compression Members				**Doesn't Control**
Effective Length Factor	**K_z =**	1		
Unbraced Length	**L_b =**	13	**ft.**	
Unbraced Length	**L_b =**	156	**in.**	
Calculations				
Factored Compressive Load	**P_u =**	616	**k**	
Design Compressive Strength	**ΦP_n =**	898.27	**k**	**Okay**
Slenderness Ratio	**KL/r_y =**	63.41		**Okay**
Elastic Buckling Stress	**F_e =**	10848.50	**ksi**	
Limiting Ratio (Nonslender)	**F_y/F_e =**	0.00		
Critical Stress (Nonslender)	**F_{cr} =**	49.90		
Limiting Ratio	**$^{(Q)}F_y/F_e$ =**	0.00	**<2.25?**	**Inelastic**
Critical Stress	**$^{(Q)}F_{cr}$ =**	49.90	**ksi**	

Base Plate Design

Loading - Plate:

P_D=		kips
P_L=		kips
OR: P_u=	616	kips
F_y=	36	ksi
F_u=	58	ksi
F'_C=	3	ksi

Column Selection **W14X68**

d=	14	in.
b_f=	10	in.
t_f=	0.72	in.
t_w=	0.415	in.
F_y=	50	ksi
F_u=	65	ksi

LRFD Factored Load

P_u=	616	kips

Plate Design

A1 RQD:	372	$in.^2$
Base b=	20	in.
TRY b=	**22**	in
A1=	484	$in.^2$
Anchor check:	20	**<b ok**
A2=	576	$in.^2$
A2/A1=	1.2	**<4 ok**
φcPp=	875	**OK**

m=	4.35	in.
n=	7.00	in.
n'=	2.96	in.
X=	0.684	
λ=	1.06	**<=1**
λn'=	2.96	
L=	7.00	in
f_{pu}=	1.3	ksi
t_{min}=	**1.96**	in.

Coulmn: B1 **USE:** **PL2in. x 22in. X 22in.**

Diaphragm Calculations

W=	461	plf	Deck:	**36/7 1.5 20 side lap 10**
B=	111	ft	Support Fastners:	**3/4 puddle welds**
L=	360	ft	SideLap Fastners :	**welded**
Joist S=	7.5	ft		
Span Condition=	3			
Deck=	1.5B 20			
K_1=	0.068			
K_2=	1056			
D_B=	97			
G'=	115	K/in		
ΔCL=	0.586	in		
R=	83000	lbs		
S=	748	plf		

Wall Studs (Curtian Wall)

Area NS=	18720	ft^2	
Area EW=	5772	ft^2	CW Metal Stud Catalog
V NS=	691		**600S200-68 (33ksi, CP60)**
VEW=	182		
Psf NS=	36.91	psf	
Psf EW=	31.53	psf	
CONTROL=	40	psf	

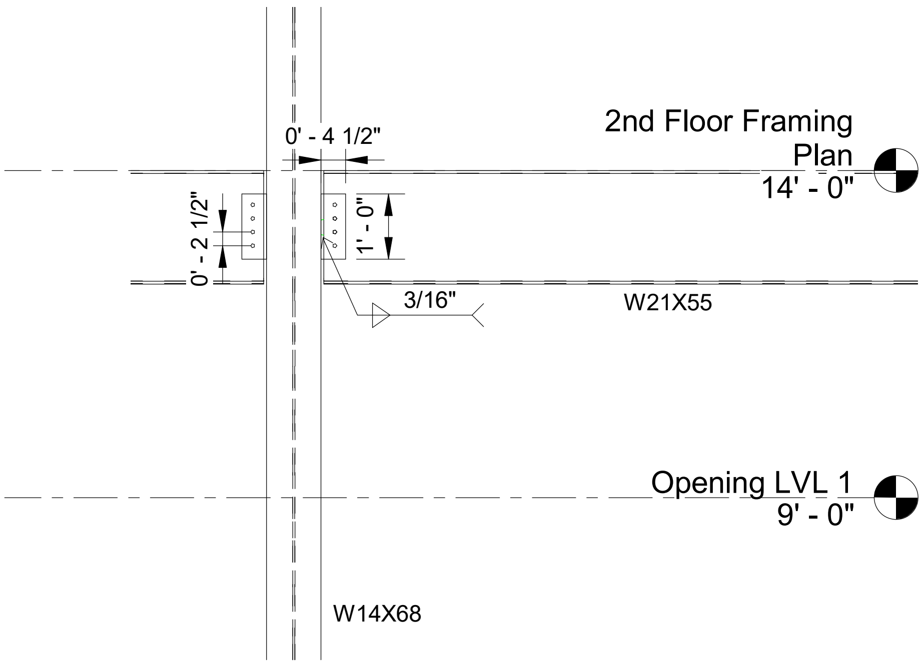
OWSJ

Roof Dead=	47	psf	
Roof Snow=	30	psf	**28K12** is refflected in the plans this is WRONG!!
Span=	43	ft	
Trib W=	7.5	ft	Eonmical joist guide for factored load LRFD
Factored LOAD	**783**	plf	**USE: 36LH13 Load Capacity 876plf**

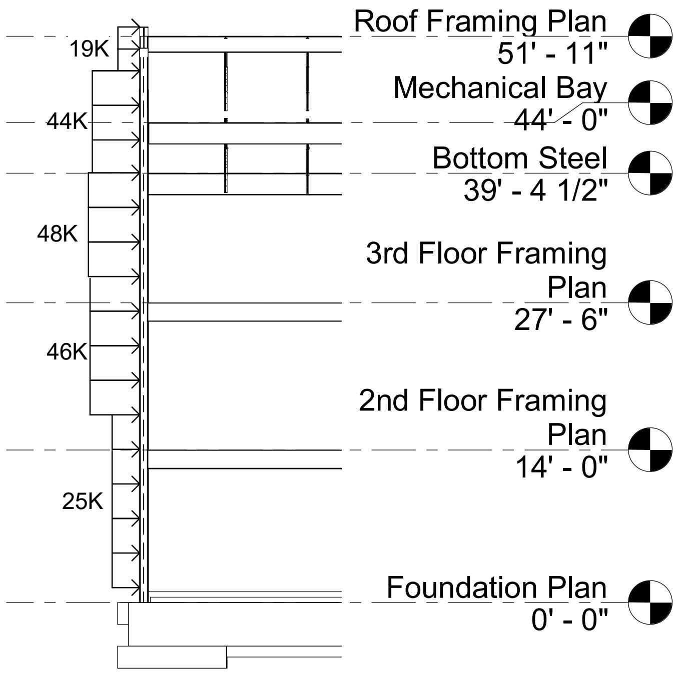
Shear Conection

V @ conection=	44	kip	
Standard Bolt:	3/4	in.	
Loading:	S		
Shear ϕRn=	17.9	kips	(7-1)
Bearing & Tear Out ϕRn=	**134**	kips	(7-4)
Plate thickness=	0.25	in.	
PlateCapacit=	**52.2**	kips	(10-10a)

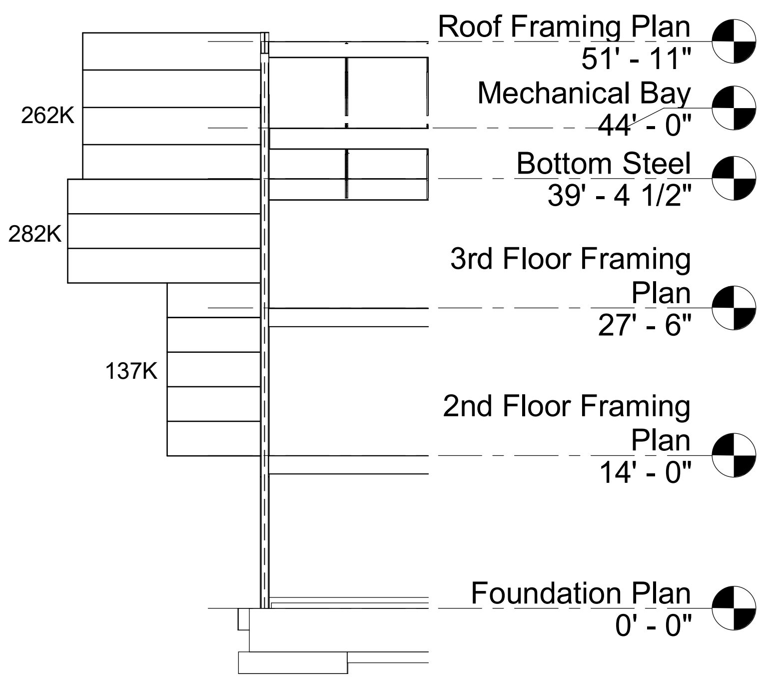
PL1/4in/x4.5in.x12in.
4 3/4in. Bolts @ 2.5in. O.C.
3/16in. Fillet weld



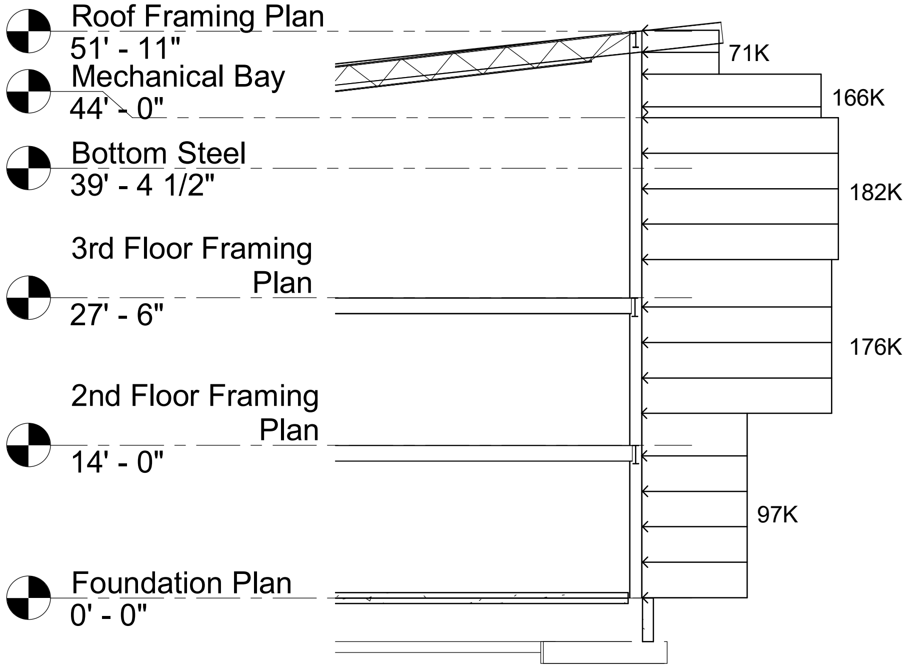
1 Connection Detail
3/8" = 1'-0"



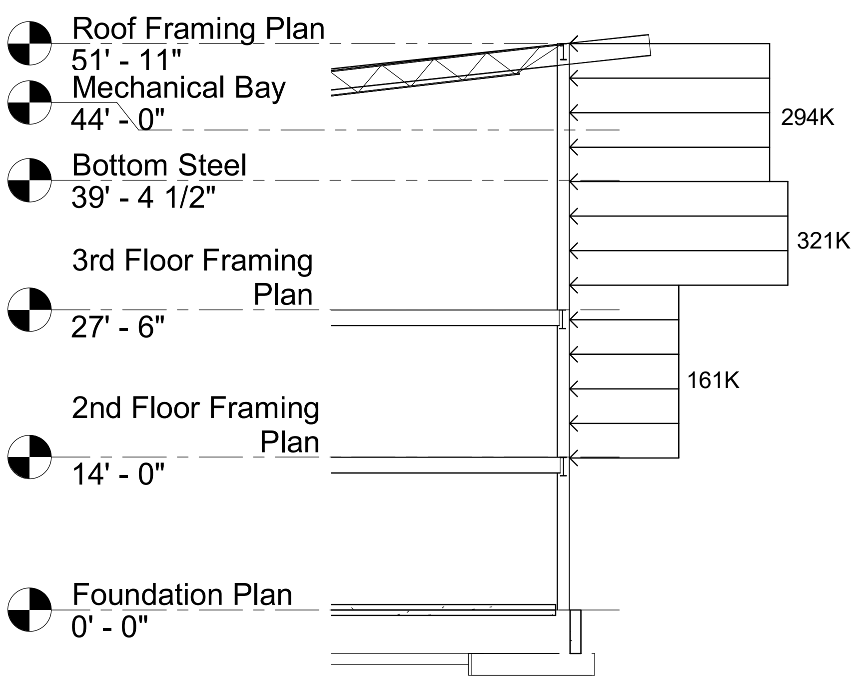
1 EW Wind - Callout 1
1/16" = 1'-0"



2 EW Seismic - Callout 1
1/16" = 1'-0"



1 NS Wind - Callout 1
1/16" = 1'-0"



2 NS Seismic - Callout 1
1/16" = 1'-0"

Concrete Footings

Footing	Pressure (P), psf	A_foot, ft^2	sqrt(area), ft	inches
A1	355500	96.08	9.80	118
A3	443200	119.78	10.94	131
B1	472100	127.59	11.30	136
B3	615700	166.41	12.90	155

Allowable Pressure (q)		4000	psf
Depth of footing (t_foot)		2	ft
p_foot	(t_foot * 150 lb/ft^3)	300	psf
q_net	(q-p_foot)	3700	psf