FINAL DESIGN REPORT

FALL 2014

Design Team: Ana Gouveia, Matthew Laskey, Zach Zavalianos

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Project Summary

Project Riverside is a two-story, steel-framed, office building located in Lowell, MA. GLZ Design has assigned the design team composed by Structural Engineers: Matthew Laskey, Ana Gouveia and Zachary Zavalianos to perform the structural analysis, evaluation and design of this project.

Guided by local regulations, the design team used of the most up to date references, associated to outstanding optimization analysis, along efficient construction practices as the project's design criteria. A more accurate overview of these values can be found in page 7 of this report. Building classifications and initial information can be found in page 8 of this report.

Riverside's design process started through a complete identification vertical and lateral loads, which were further analyzed by the Load & Resistance Factor Design (LRFD). These initial values were used in the preliminary-sizing of project horizontal and vertical members.

In the second phase of the project, these preliminary members were further evaluated and analyzed. Final framing plans for both first and roof floor were obtained. Two different lateral load resisting systems (LLRS) were designed: a moment-frame resists longitudinal loads, and braced-frames are responsible for transverse loads. Given building risk categories, per code used, ordinary moment-frame and braced-frame were allowed in the design, reducing the project's final budget.

In the project's final phase, composite floor deck, roof joists, girders and diaphragms, and connections were designed. For quality assurance and control, all of the project's information, calculations, analysis and drawings were further back checked. The final results of this project can be found in this report. Calculations, Analysis and Drawings are referred in its respective section on the report and can also be found at this report's appendixes.

GLZ Design appreciates your business and hope to continue serving your Structural Engineering needs in the future.

Sincerely,

Ana Gouveia, EIT	Matt Laskey, EIT	Zac Zavalianos, EIT

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DESIGN TEAM

Design Team

We are pleased to introduce you Riverside's design team:

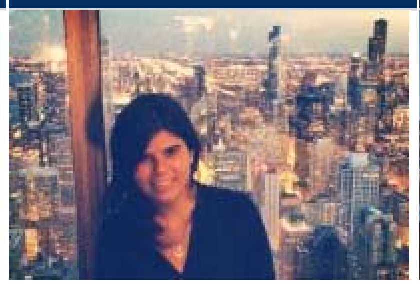
MATTHEW LASKEY, EIT
STRUCTURAL ENGINEER

m.laskey@yahoo.com

Matt continued working as a Structural Engineer at CDM Smith after his summer internship at the firm

Matt has a bachelor's degree in Environmental and Resources Economics from UNH and a Civil Engineering degree from UMass Lowell.

ANA GOUVEIA, EIT
STRUCTURAL ENGINEER



acgouveia@live.com

Ana was born in Brazil where she started her college education in Civil Engineering.

As a challenge-seeker, she immigrated to Massachusetts in 2009. Since then studied at Harvard University Extension School, acquired an associate's degree in Engineering from Bunker Hill Community College, a bachelor's degree in Civil Engineering from the University of Massachusetts Lowell.

ZAC ZAVALIANOS, EIT
STRUCTURAL ENGINEER

Zachary_Zavalianos@student.uml.edu

Hardworking and enthusiastic structural engineer with experience in design and inspection of bridge structures. Certified in bridge inspection and proficient in inspection databases such as PONTIS and 4-D, as well as MDX and CSI-bridge analysis programs. Seeking to progress career by obtaining a masters degree and working towards a PE license through the upcoming years.

Company Information

GLZ Design

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GLZ Design, LLC

1 University Ave, Lowell, MA

www.glzdesign.com

DESIGN CRITERIA

Design Criteria

Design Criteria and Constraints

The following figure represents the design criteria which guided the team:

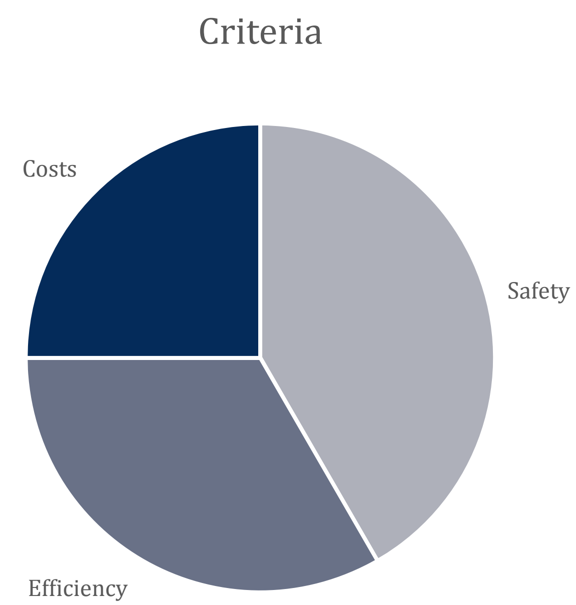

Figure A – Riverside's guiding criteria.

In order to satisfy the above criteria and constraints the following items were used:

References and Regulations

The following literature was used in reference and support of the work done in this project. Please note that names in parentheses are abbreviations commonly cited throughout this project.

- Massachusetts Building Code (CMR 780)
- 2012 International Building Code (2012 IBC)
- Minimum Design Loads for Buildings and Other Structures, (ASCE/SEI 7-10)
- 14th Edition of the Steel Construction Manual (AISC 14th)
- American Concrete Institute Manual (ACI)

DESIGN CRITERIA

- 2007 Steel Deck Institute Design Manual (2007 SDI)
- Vulcraft Joists and Girders (VJG)
- 5th Edition Structural Steel Design, McCormac & Csernak (SSD 5th)

Methods of Design and Analysis

The following methods of design and analysis were thoroughly applied to the different parts of this project.

- Loads: Load & Resistance Factor Design (LRFD) and ASCE 7-10 guidelines.
- Wind Loads: Directional Method (ASCE 7-10)
- Seismic Loads: (ASCE 7-10)
- Preliminary-Sizing: Principles of Structural Engineering Analysis
- Lateral Load Resisting Systems: Direct Analysis Method (DAM) and Effective Length Method (ELM)
- Composite Deck
- Roof Elements
- Connections

Building Information and Requirements

Classifications

- Occupancy: Group B, Office
- Construction: Type IIB
- Risk Category: II
- Seismic Site Class: C
- Environmental Exposure: C

Building Layout

- Total Area = Approx. 17,000 sf (gross)
- Floor Plan: 110 ft. x 74ft
- Column Grid: 36 ft. x 24 ft.

Loads

- The building shall be designed in accordance with 2012 IBC structural provisions and as modified by the Massachusetts Building Code (CMR780).
- Snow, wind, and seismic loads shall be calculated for the Lowell, MA location.

DESIGN CRITERIA

- Minimum Uniform Live Load shall be 100 psf.

Lateral Load Resisting Systems

- Braced Frame in one direction
- Moment Frame in opposite direction
- Rigid Floor and Roof Diaphragms

Floor Systems

- 4,000 psi concrete Floor
- Metal deck
- Composite Steel W-shaped A992 beams
- Steel Girders (W)

Roof Systems

- Single-ply 60 mil EPDM membrane
- 4” rigid insulation
- Metal deck
- Option 1: Open-web steel joists w/ steel girders (W)

Exterior Wall Systems

- Primary Support: 6” Metal stud
- Exterior cladding: 2” Exterior Insulated Finishing Systems
- Fiberglass Batt. Insulation (6”)

Scope of Work

Overview

In order to perform this project, GLZ Design determined the criteria and team mentioned in the previous pages.

During Phase 1 of the design process (30% Submittal), members were given the naming convention mentioned in Figures 1-4 below, load evaluations and analysis were done. Calculations can be found within appendix A, Section A-1 to A-10. Preliminary sizes were determined and are catalogued in Calculations Section A-11.

During Phase 2, preliminary sizes were used to determine the building's lateral load resisting system. New member sections were obtained, results catalogued in Calculations Section A-11, and building framing plans and elevations were drafted. Drawings S-2 to S-4 can be found within appendix B. Further, the building composite deck and roof open-web joists and girders were also designed. Calculations can be found in Section A-21 and A-22, respectively.

During Phase 3, connections and other details were designed. Calculations can be found in Sections A-23 to A-24, while final drawings can be found within appendix B.

Members

The members in the project were labeled as shown in the figures below:

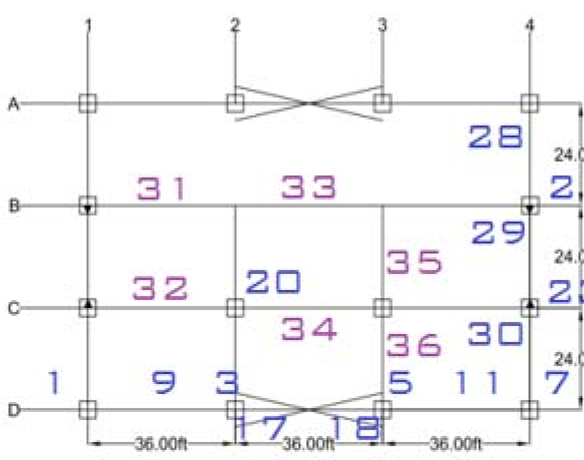

Figure 1 - First Floor Plan

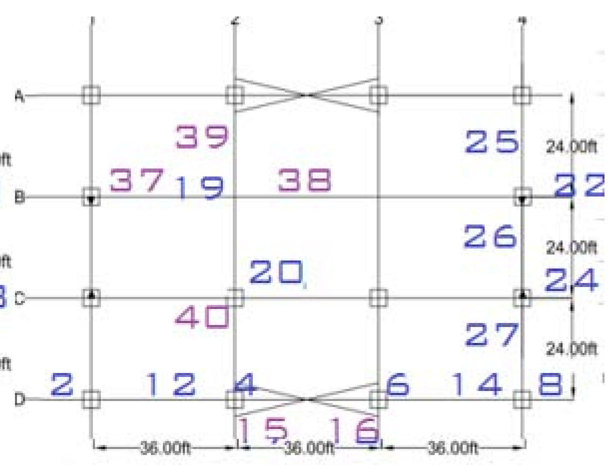

Figure 2 - Roof Floor Plan

SCOPE OF WORK

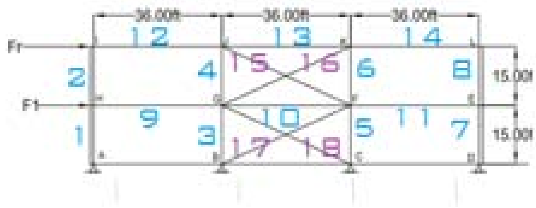

Figure 3 - Braced-Elevation

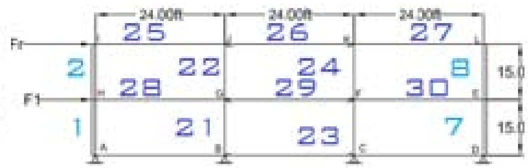

Figure 4 - Moment Frame - Elevation

Lateral Load Resisting Systems

Through the Direct Analysis and Effective Analysis methods, two frame systems were designed. Longitudinally a braced frame can be found, while transversely the project uses of a moment frame. The designed connections calculations can be found within Appendix A, section A-30.

Connections

The designed connections calculations can be found within Appendix A, section A-40.

Composite Deck & Roof Design

Using a parallel panels to the girders the buildings composite floor consists of 6” of concrete poured in 18” g.a. steel sheets. Calculations are found in appendix A. Further, GLZ Design team determined and used of open-web joists and girders for the roof. Calculations are found in appendix A, section A-50 Additional Details.

Schedule

The project was completed within schedule.

Cost Estimate

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Appendix A: Calculations

1. BUILDING SPECIFICATIONS:

A. CLASSIFICATIONS:

Occupancy:	B	Office		
Construction Type:	II	B		
Risk Category:	II			
Seismic Site Class:	C		Importance Factor:	1
Environmental Exposure:	C			

B. BUILDING LAYOUT

Number Columns:	3				End Clearance:	2	in
Total Area:	8140	ft^2					
Floor Plan:	110	ft	x	74	ft		
Column Grid:	36	ft	x	24	ft		
Total Height:	30	ft					
Finish Floor to Fin. Floor Height:	15	ft			# Stories:	2	
Structural Allowance:	4	ft					

C. LATERAL LOAD RESISTING SYSTEMS:

D. FLOOR SYSTEMS:

Concrete Floor Strength:	4	ksi	$\gamma =$	0.15	kcf
Concrete Floor Thickness:	6	in			
Metal Deck					
Composite Steel Beams:	W		Type:	A992	
Steel Girders:	W				

E. ROOF SYSTEM

Single-ply 60 mil EPDM membrane		
Rigid Insulation Thickness:	4	in
Metal Deck		
Option 1:	Open-web steel joists w/ steel girders (W)	

F. EXTERNAL WALL SYSTEM

Primary Suport:	Metal Stud	Thickness:	4	in
Insulation:	Fiberglass Batt. Insulation	Thickness:	6	in
Exterior Cladding:	Ex. Insulated Finishing Systems	Thickness:	2	in

1. BUILDING LOADS AS REQUIRED BY CODE:

Loads are in accordance to:	2012	IBC	
Modified by:	Massachusetts Building Code		(CMR780)
Snow	Lowell	MA	
Wind	Lowell	MA	
Seismic	Lowell	MA	
Construction Live Load:	20	psf	
Uniform Live Load:	100	psf	

2.1 BUILDING VERTICAL LOADS:

	Roof		1st Floor	
2.1. Dead Load (D)	46.5	psf	93	psf
2.2. Live Load (L)	50.0	psf	100	psf
2.3. Roof Live Load (Lr)	20.0	psf		psf
2.4. Snow Load (S)	31.5	psf		psf
2.5. Rain Load (R)		psf		psf
2.6. Seismic Load (E)		psf		psf
2.7. Wind Load (W)	-23.5	psf		psf
Total / Service Load:	124.5		193	psf

2.2 LOAD COMBINATIONS PER LRFD SPECIFICATIONS:

	Roof		1st Floor	
1. 1.4D	65.1	psf	130.2	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	151.6	psf	271.6	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5W)	156.2	psf	211.6	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)	48.1	psf	111.6	psf
5. 1.2D + 1.0E + L + .2S	112.1	psf	211.6	psf
6. 0.9D + 1.0W	18.4	psf	83.7	psf
7. 0.9D + 1.0E	41.9	psf	83.7	psf
Controlling Load:	156.2	psf	271.6	psf

3.1 BUILDING LATERAL LOAD ON LONGITUDINAL DIRECTION: BRACED-FRAME

	Roof			1st Floor		
2.1. Dead Load (D)		kip			kip	
2.2. Live Load (L)		kip			kip	
2.3. Roof Live Load (Lr)		kip			kip	
2.4. Snow Load (S)		psf			psf	
2.5. Rain Load (R)		psf			psf	
2.6. Seismic Load (E)	10.7	kip	10.7	10.4	kip	10.437
2.7. Wind Load (W)	6.4	kip		12.9	kip	
Total / Service Load:	17.1	psf		23.3	psf	

3.2 LOAD COMBINATIONS PER LRFD SPECIFICATIONS:

	Roof		1st Floor	
1. 1.4D	0.0	kip	0.0	kip
2. 1.2D + 1.6L + .5(Lr or S or R)	0.0	kip	0.0	kip
3. 1.2D + 1.6(Lr or S or R) + (L or .5W)	3.2	kip	6.4	kip
4. 1.2D + 1.0W + L + .5(Lr or S or R)	6.4	kip	12.9	kip
5. 1.2D + 1.0E + L + .2S	10.7	kip	10.4	kip
6. 0.9D + 1.0W	6.4	kip	12.9	kip
7. 0.9D + 1.0E	10.7	kip	10.4	kip
Controlling Load:	10.7	kip	12.9	kip

4.1 BUILDING LATERAL LOAD ON TRANSVERSE DIRECTION: MOMENT-FRAME

	Roof			1st Floor		
2.1. Dead Load (D)		kip			kip	
2.2. Live Load (L)		kip			kip	
2.3. Roof Live Load (Lr)		kip			kip	
2.4. Snow Load (S)		kip			kip	
2.5. Rain Load (R)		kip			kip	
2.6. Seismic Load (E)	3.2	kip	8.03	12.6	kip	7.83
2.7. Wind Load (W)	9.8	kip		19.6	kip	
Total / Service Load:	13.0	kip		32.2	kip	

4.2 LOAD COMBINATIONS PER LRFD SPECIFICATIONS:

	Roof		1st Floor	
1. 1.4D	0.0	psf	0.0	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	0.0	psf	0.0	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5W)	4.9	psf	9.8	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)	9.8	psf	19.6	psf
5. 1.2D + 1.0E + L + .2S	3.2	psf	12.6	psf
6. 0.9D + 1.0W	9.8	psf	19.6	psf
7. 0.9D + 1.0E	3.2	psf	12.6	psf
Controlling Load:	9.8	psf	19.6	psf

	Reference:	ASCE	7-10
	Section	*Eq/Fig/Table/Notes*	
DEAD LOAD	3		

Floor:	**Roof**					**2nd**				
	Item	**Quantity** *(Area)*	**Units**	**Unit Weigh** *(ksf or klf)*	**Weight** *(kip)*	**Item**	**Quantity** *(Area)*	**Units**	**Unit Weight** *(ksf or klf)*	**Weight** *(kip)*
						Concrete Slab	*8140*	*sf*	0.075	611
	Metal Deck	8140	*sf*	0.0025	20	**Metal Deck 18 g.a**	8140	*sf*	0.0025	20
	EPDM Membrane	8140	*sf*	0.001	8	**Additional Load**	8140	*sf*	0.0155	126
	Insulation	8140	*sf*	0.006	49					0
	Mechanical Equipment	8140	*sf*	0.005	41					0
										0
Subtotal	*118*			0.047	118	*757*			0.093	757
Cummulative	*118*				118	*875*				**875**

***Unit Weights per ASCE 7-10**

	Reference:	ASCE	7-10
	Section	*Eq/Fig/Table/Notes*	
SNOW LOAD	7		

Exposure Factor	C_e =	0.90		Table	7-2
Thermal Factor	C_t =	1.00		Table	7-3
Importance Factor	I_s =	1.00		Table	1.5-2
Ground Snow Load	ρ_g =	50.00	psf	Figure	7-1
Flat Roof Snow Load	ρ_f =	31.50	psf	Eq	7.3-1

Reference:	ASCE 7-10
Section	*Eq/Fig/Table/Notes*

SEISMIC LOAD 12

Number of Floors: 2

Floor:	Roof					2nd				
	Item	**Quantity** *(Area)*	**Units**	**Unit Weigh** *(ksf or klf)*	**Weight** *(kip)*	**Item**	**Quantity** *(Area)*	**Units**	**Unit Weight** *(ksf or klf)*	**Weight** *(kip)*
	Same as dead l	8140		0.047	379	**Same as dead load**	8140	*sf*	0.093	757
						Partitions	8140	*sf*	0.01	81
	20% Snow	8140	*sf*	0.0063	51					
Subtotal	*430*				**430**	*838*				**838**
Cummulative	*430*				**430**	*1268*				**1268**
Mass (kip*s^2/ft)					107					58

WIND LOAD ANALYSIS

				Reference:	ASCE	7-10
				Section	*Eq/Fig/Table/Notes*	
1. BUILDING INFORMATION RELATED TO WIND LOAD ANALYSIS				26/27/28		
Mean roof height	H_{roof} =	30	ft	*Height of heighest level of structure*		
Floor-Floor Height	h_n =	15	ft			
Building Length	L =	112	ft			
Building Width	W =	76	ft			
Number of Braces/Level		2				
Number of Moment Frames/ Level		2				
2. WIND EXPOSURE, ROUGHNESS AND OCCUPANCY CATEGORY				26.4		
Occupancy Category:		B			Table	1-1
Ground Surface Roughness:		B				26.7.2
Exposure Category:		C				26.7.2
3. ENVIRONMENTAL CHARACTERISTICS AND FACTORS				26.5		
Wind Speed	V =	120	mph	26.5	Figure	26.5-1A
Zone A	P_{s30} =	22.8	psf		Figure	28.6-1
Zone C	P_{s30} =	15.1	psf		Figure	28.6-1
	a_1 =	7.6	ft	*.1*W*		
	a_2 =	12	ft	*.4*H_{roof}*		
	a =	7.6	ft	*Min Value*		
	2.a =	15.2	ft			
Weighted Average for P_{s30}:						
Longitudinal		16.1	psf			
Transverse		16.6	psf			
4. DESIGN WIND PRESSURE				26.8		
Adjustment Factor	λ =	1.4			Figure	28.6-1
	K_{zt} =	1		*26.8*		
Design wind pressure,	$P_{s\text{-}longitudinal}$ =	22.6	kip			
Design wind pressure,	$P_{s\text{-}transverse}$ =	23.3	kip			
5. LOAD APPLIED TO EACH LEVEL				26.8		
Roof	$F_{u\text{-}longitudinal}$ =	12.9	kip		Figure	28.6-1
	$F_{u\text{-}transverse}$ =	19.6	kip	*26.8*		

Level 1 $F_{u\text{-longitudinal}} =$ 25.8 kip
$F_{u\text{-transverse}} =$ 39.1 kip

6. LATERAL LOAD APPLIED TO BRACED AND MOMENT FRAME

		Roof	Level 1	
Braced Frame	(Longitudinal)	6.44	12.88	kip
Moment Frame	(Transverse)	9.78	19.57	kip

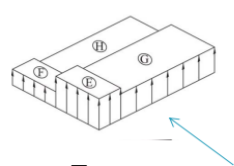
7. VERTICAL UPLIFT PRESSURES ON ROOF

Zone E P_{s30}	-27.4	psf	Figure	28.6-1
Zone F P_{s30}	-15.6	psf	Figure	28.6-1
Zone G P_{s30}	-19.1	psf	Figure	28.6-1
Zone H P_{s30}	-12.1	psf	Figure	28.6-1

Design wind pressure Zone E,	P_s	-38.36	psf
Design wind pressure Zone F,	P_s	-21.84	psf
Design wind pressure Zone G,	P_s	-26.74	psf
Design wind pressure Zone H,	P_s	-16.94	psf

7. UPLIFT PRESSURE (TRANSVERSE LOADING)

Area, Zone E	577.6	ft^2
Area, Zone F	577.6	ft^2
Area, Zone G	3678.4	ft^2
Area, Zone H	3678.4	ft^2
Total Roof Area	8512	ft^2

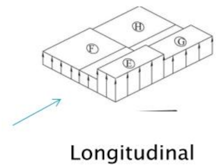


Transverse

Weighted Uplift Pressure from Transverse Wind Load -22.96 psf

8. UPLIFT PRESSURE (LONGITUDINAL LOADING)

Area, Zone E	851.2	ft^2
Area, Zone F	851.2	ft^2
Area, Zone G	3404.8	ft^2
Area, Zone H	3404.8	ft^2
Total Roof Area	8512	ft^2



Weighted Uplift Pressure from Longitudinal Wind Load	-23.49	psf

9. MAXIMUM UPLIFT PRESSURE

Controlling Uplift Pressure	-23.49	psf	Largest Absolute Value

ASSUMPTIONS:

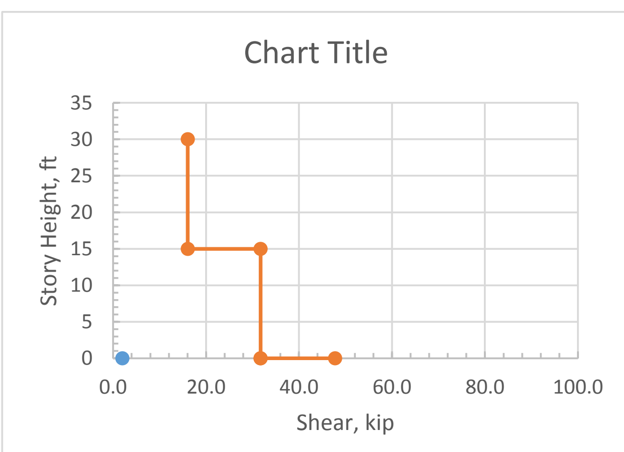
Building Frame System: Steel moment-resisting frame

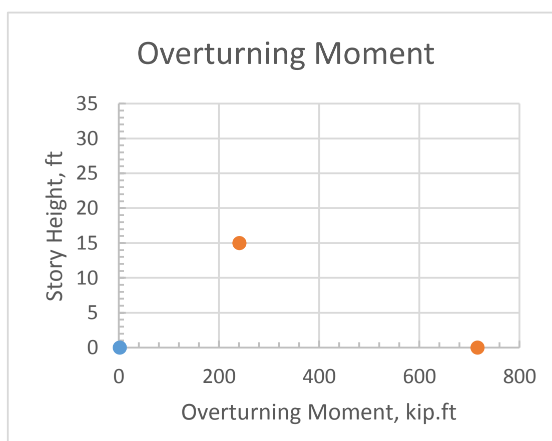
			Reference: Section	ASCE 7-10 Eq/Fig/Table/Notes	
1. SEISMIC GROUD MOTION VALUES			11.4		
Seismic Site Class:	C		11.4.2	*Soil Properties / Ch. 20*	
Maximum Considered Earthquake Spectral Response:					
S_s =	0.250		11.4.1	Fig	22-1 / 22-4
S_1 =	0.077		11.4.1	Fig	22-1 / 22-4
Adjusted MCE Spectral Response:					
F_a =	1.200		11.4.3	Table	11.4-1
F_v =	1.700		11.4.3	Table	11.4-2
$S_{MS} = F_a S_s$ =	0.3		11.4.3	Eq	11.4-1
$S_{M1} = F_v S_1$ =	0.131		11.4.3	Eq	11.4-2
Design Spectral Response Acceleration Parameters:					
$S_{DS} = 2/3\ S_{MS}$ =	0.2		11.4.4	Eq	11.4-3
$S_{D1} = 2/3\ S_{M1}$ =	0.087		11.4.4	Eq	11.4-4
Design Response Spectrum:					
$T_O = 0.2\ S_{D1}/S_{DS}$ =		s	11.4.5		
$T_S = S_{D1}/S_{DS}$ =	0.436	s	11.4.5		
Long Period Transition T_L=	6	s	11.4.5	Fig	22-15
T =	0.60	s	*Fundamental Period of Structure*		
S_a = if $T < T_O : S_{DS}(0.4+0.6T/T_O)$ =			11.4.5	Eq	11.4-5
if $T_O < T < T_S : S_{DS}$ =			11.4.5		
if $T_S < T < T_L : S_{D1}/T$ =	0.147		11.4.5	Eq	11.4-6
if $T > T_L : S_{D1}{*}T_L/T^2$=			11.4.5	Eq	11.4-7

2. IMPORTANCE FACTOR AND OCCUPANCY CATEGORY		11.5		
Occupancy Category:	II		Table	1-1
Importance Factor:	1		Table	11.5-1
3. SEISMIC DESIGN CATEGORY		11.6		
SDC based on short period:	B		Table	11.6-1
SDS based on 1-s period:	B		Table	11.6-2
SDC =	**B**	*Maximum from values above*		
4. EQUIVALENT LATERAL FORCE PROCEDURE		12.8		
R =	8	12.8.1	Table	12.2-1
Ω_O =	3	12.8.1	Table	12.2-1
C_D =	3	12.8.1	Table	12.2-1

Approximate Fundamental Period, T_a:			12.8.2.1		
C_t =	0.028		*Dependent on structur*	Table	12.8-2
x =	0.8			Table	12.8-2
h_n =	30	ft	*Height of heighest level of structure*		
$T_a = C_t h_n^x$ =	0.425	s	12.8.2.1	Eq	12.8-7
Seismic Response Coefficient:			12.8.1.1		
$C_{Scalc} = S_{DS}/(R/I)$ =	0.025			Eq	12.8-2
C_{Smax} = if $T <= T_L : S_{D1}/(T*(R/I))$ =	0.018			Eq	12.8-3
if $T > T_L : S_{D1*}T_L/(T^2*(R/I))$ =				Eq	12.8-4
C_{Smin} =	0.01		*Revised Sup. 2*	Eq	12.8-5/12.8-6
C_S =	0.025		*Beware of mins and max*		
Seismic Base Shear:			12.8.1		
Seismic Weight W =	1268	kip	12.7.2		Table Below
Seismic Base Shear V =	31.7	kip	12.8.1	Eq	12.8-1
Vertical Distribution of Seismic Forces:			12.8.3		
Lateral force per level	$F_x = C_{vx}V$			Eq	12.8-11
$C_{vx} = (w_x h_x^k)/(\Sigma\, w_i h_i^k)$			*Vertical Distribution Factor*		
k =	1.00	0.96	12.8.3		
Horizontal Distribution of Seismic Forces:			12.8.4		
$V_x = \Sigma\, F_i$ =				Eq	12.8-13

Floor	Height *(ft)*	Weight *(kip)*	$w_x h_x^k$	C_{vx}	F_x *(kip)*	V_x *(kip)*	Overturning Moment *(kip.ft)*	Total Height *(ft)*
Roof	15	430	12894	0.506	16.1	16.1		30
						16.1		15
2nd	15	838	12576	0.494	15.7	31.7	241	15
						31.7		0
Podium	0	0	0	0	0	31.7	716	0
SUM	30	1268	25470	1	31.7	47.8		0





ASSUMPTIONS:

Building Frame System: Eccentrically braced steel frame

			Reference:	ASCE	7-10
			Section	*Eq/Fig/Table/Notes*	
1. SEISMIC GROUD MOTION VALUES			11.4		
Seismic Site Class:	C		11.4.2	*Soil Properties / Ch. 20*	
Maximum Considered Earthquake Spectral Response:					
S_s =	0.250		11.4.1	Fig	22-1 / 22-4
S_1 =	0.077		11.4.1	Fig	22-1 / 22-4
Adjusted MCE Spectral Response:					
F_a =	1.200		11.4.3	Table	11.4-1
F_v =	1.700		11.4.3	Table	11.4-2
$S_{MS} = F_aS_s$ =	0.300		11.4.3	Eq	11.4-1
$S_{M1} = F_vS_1$ =	0.131		11.4.3	Eq	11.4-2
Design Spectral Response Acceleration Parameters:					
$S_{DS} = 2/3\ S_{MS}$ =	0.2		11.4.4	Eq	11.4-3
$S_{D1} = 2/3\ S_{M1}$ =	0.087		11.4.4	Eq	11.4-4
Design Response Spectrum:					
$T_O = 0.2\ S_{D1}/S_{DS}$ =		s	11.4.5		
$T_S = S_{D1}/S_{DS}$ =	0.436	s	11.4.5		
Long Period Transition T_L=	6	s	11.4.5	Fig	22-15
T =	0.54	s	*Fundamental Period of Structure*		
S_a = if $T < T_O : S_{DS}(0.4+0.6T/T_O)$ =			11.4.5	Eq	11.4-5
if $T_O < T < T_S : S_{DS}$ =			11.4.5		
if $T_S < T < T_L : S_{D1}/T$ =	0.162		11.4.5	Eq	11.4-6
if $T > T_L : S_{D1}*T_L/T^2$=			11.4.5	Eq	11.4-7

2. IMPORTANCE FACTOR AND OCCUPANCY CATEGORY 11.5

Occupancy Category:	II	Table	1-1
Importance Factor:	1	Table	11.5-1

3. SEISMIC DESIGN CATEGORY 11.6

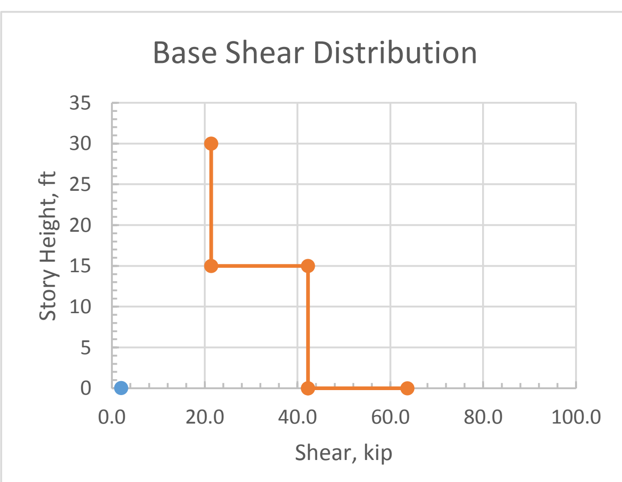
SDC based on short period:	B	Table	11.6-1
SDS based on 1-s period:	B	Table	11.6-2
SDC =	**B**	*Maximum from values above*	

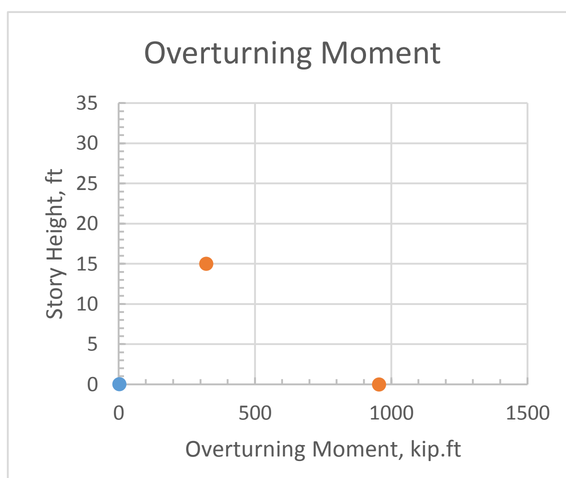
4. EQUIVALENT LATERAL FORCE PROCEDURE 12.8

R =	6	12.8.1	Table	12.2-1
Ω_O =	2	12.8.1	Table	12.2-1
C_D =	3.25	12.8.1	Table	12.2-1

Approximate Fundamental Period, T_a:			12.8.2.1			
C_t =	0.03		*Dependent on structure*	Table	12.8-2
x =	0.75			Table	12.8-2
h_n =	30	ft	*Height of heighest level of structure*		
$T_a = C_t h_n^x$ =	0.385	s	12.8.2.1	Eq	12.8-7
Seismic Response Coefficient:			12.8.1.1		
$C_{Scalc} = S_{DS}/(R/I)$ =	0.033			Eq	12.8-2
C_{Smax} = if T <= T_L : $S_{D1}/(T*(R/I))$ =	0.027			Eq	12.8-3
if T > T_L : $S_{D1*}T_L/(T^2*(R/I))$ =				Eq	12.8-4
C_{Smin} =	0.01		*Revised Sup. 2*	Eq	12.8-5/12.8-6
C_S =	0.033				
Seismic Base Shear:			12.8.1		
Seismic Weight W =	1268	kip	12.7.2		Table Below
Seismic Base Shear V =	42.3	kip	12.8.1	Eq	12.8-1
Vertical Distribution of Seismic Forces:			12.8.3		
Lateral force per level	$F_x = C_{vx}V$			Eq	12.8-11
	$C_{vx} = (w_x h_x^k)/(\Sigma\, w_i h_i^k)$		*Vertical Distribution Factor*		
k =	1.00	0.94	12.8.3		
Horizontal Distribution of Seismic Forces:			12.8.4		
$V_x = \Sigma\, F_i$ =				Eq	12.8-13

Floor	Height (ft)	Weight (kip)	$w_x h_x^k$	C_{vx}	F_x (kip)	V_x (kip)	Overturning Moment (kip.ft)	Total Height (ft)
Roof	15	430	12894	0.506	21.4	21.4		30
						21.4		15
2nd	15	838	12576	0.494	20.9	42.3	321	15
						42.3		0
Podium	0	0	0	0	0	42.3	955	0
SUM	30	1268	25470	1	42.3	63.7		0





1. MOMENT FRAME INFORMATION

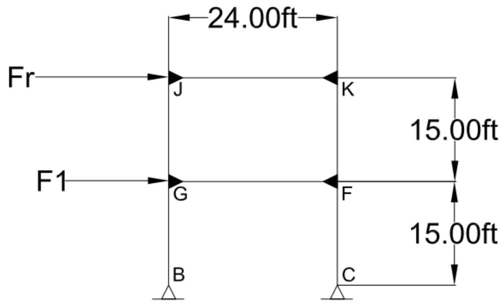


Figure 1 - Moment Frame

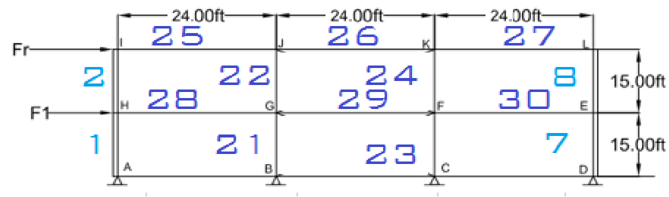


Figure 2 - Member Reference

Story Height, H =	15	ft
Bay Width, W =	24	ft
$F_{r\,=}$	9.78	kip
$F_{1\,=}$	19.57	kip

2.1 MEMBER FORCES DISTRIBUTION

Compute Shear Forces

$\sum F_x=0$

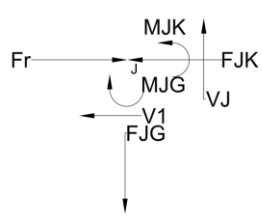
$F_{r\,=}$	9.78	kips
# of shear forces	2	
$V_{1\,=}$	4.89	kips

Compute moments at top of columns

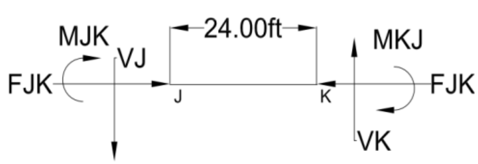
$M_{JG\,=}$	36.69	kip*ft
$M_{KF\,=}$	36.69	kip*ft

METHOD OF JOINTS:

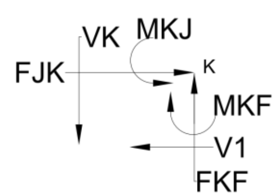
Joint J



Member J-K



Joint K



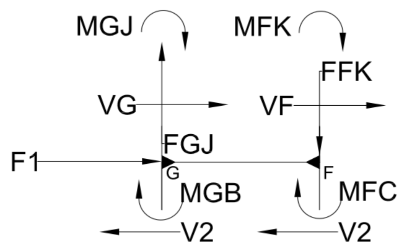
Joint J:

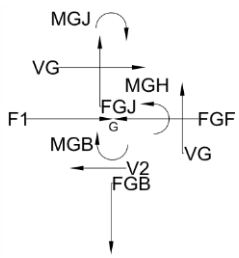
	$M_{JK\,=}$	36.69	kip*ft	
$\sum$Fx=0	$F_{JK\,=}$	4.89	kips	C
$\sum$M/W	$V_{J\,=}$	3.06	kips	
$\sum$Fy=0	$F_{JG\,=}$	3.06	kips	T

Joint K:

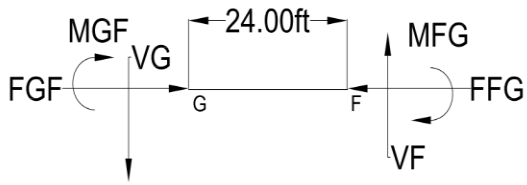
$M_{KJ\,=}$	36.69	kip*ft	
$F_{JK\,=}$	4.89	kips	C
$V_{K\,=}$	3.06	kips	
$F_{KF\,=}$	3.06	kips	C
$M_{KF\,=}$	36.69	kips	

Section 2

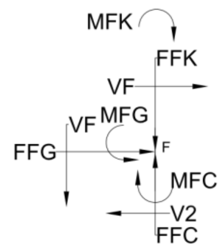




Joint G



Member G-F



Joint F

Joint G:

(F1+FR)/2	$V_{2\,=}$	14.68	kips	
FJG	$F_{GJ\,=}$	3.06	kips	T
V2*H/2	$M_{GB\,=}$	110.1	kip*ft	
(MJG)	$M_{GJ\,=}$	36.69	kip*ft	
(FJG)	$F_{GJ\,=}$	3.06	kips	
∑Fx=0	$F_{GF\,=}$	4.89	kips	C
∑M	$M_{GF\,=}$	146.76	kip*ft	
∑M/W	$V_{G\,=}$	12.23	kips	
∑Fy=0	$F_{GB\,=}$	15.29	kips	

Joint F

(F1+FR)/2	$V_{2\,=}$	14.68	kips	
FKF	$F_{FK\,=}$	3.06	kips	T
V2*H/2	$M_{FC\,=}$	110.1	kip*ft	
MKF	$M_{FK\,=}$	36.69	kip*ft	
FKF	$F_{FK\,=}$	3.06	kips	
FGF	$F_{FG\,=}$	4.89	kips	C
∑M	$M_{FG\,=}$	146.76	kip*ft	
∑M/W	$V_{G\,=}$	12.23	kips	
∑Fy=0	$F_{FC\,=}$	15.29	kips	

3. RESULTS

	Member (#)	Frame (type)	Floor (Units)	Function	Force (kip)	T/C	Moment (kip.ft)
GF	29	Moment	First	Beam	4.89	C	146.76
JK	26	Moment	Roof	Beam	4.89	C	36.69
BG	21	Moment	First	Column	15.29	T	110.07
CF	23	Moment	First	Column	15.29	C	110.07
JG	22	Moment	Roof	Column	4.89	T	36.69
KF	24	Moment	Roof	Column	4.89	C	36.69

1. MOMENT FRAME INFORMATION

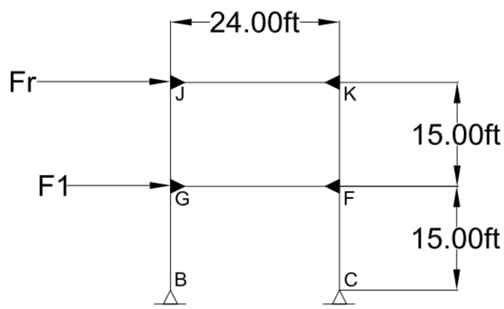


Figure 1 - Moment Frame

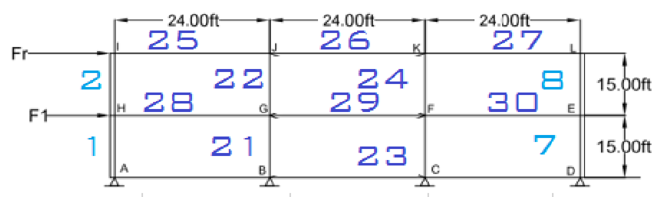


Figure 2 - Member Reference

Story Height, H = 15 ft

Bay Width, W = 24 ft

$F_{r} =$ 3.23 kip

$F_{1} =$ 12.62 kip

2.1 MEMBER FORCES DISTRIBUTION

Compute Shear Forces

$\sum F_x=0$

$F_{r} =$ 3.23 kips

# of shear forces 2

$V_{1} =$ 1.62 kips

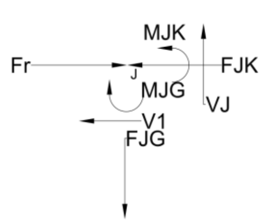
Compute moments at top of columns

$M_{JG} =$ 12.11 kip*ft

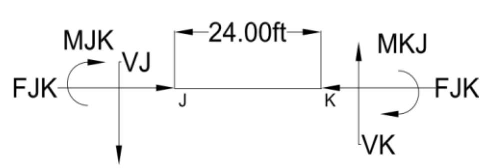
$M_{KF} =$ 12.11 kip*ft

METHOD OF JOINTS:

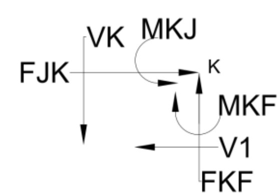
Joint J



Member J-K



Joint K



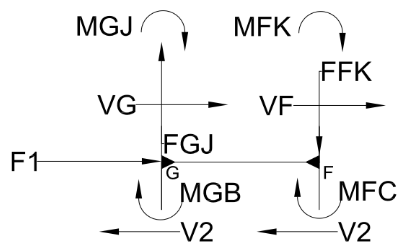
Joint J:

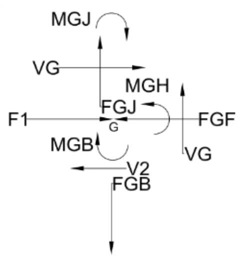
	$M_{JK} =$	12.11	kip*ft	
$\sum Fx=0$	$F_{JK} =$	1.62	kips	C
$\sum M/W$	$V_{J} =$	1.01	kips	
$\sum Fy=0$	$F_{JG} =$	1.01	kips	T

Joint K:

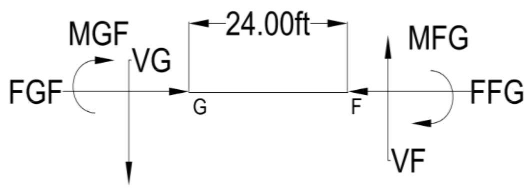
$M_{KJ} =$	12.11	kip*ft	
$F_{JK} =$	1.62	kips	C
$V_{K} =$	1.01	kips	
$F_{KF} =$	1.01	kips	C
$M_{KF} =$	12.11	kips	

Section 2

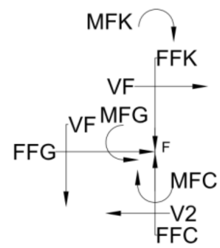




Joint G



Member G-F



Joint F

Joint G:

(F1+FR)/2	$V_{2\,=}$	7.9	kips	
FJG	$F_{GJ\,=}$	1.01	kips	T
V2*H/2	$M_{GB\,=}$	59.44	kip*ft	
(MJG)	$M_{GJ\,=}$	12.11	kip*ft	
(FJG)	$F_{GJ\,=}$	1.01	kips	
$\sum$Fx=0	$F_{GF\,=}$	4.70	kips	T
$\sum$M	$M_{GF\,=}$	71.6	kip*ft	
$\sum$M/W	$V_{G\,=}$	6.0	kips	
$\sum$Fy=0	$F_{GB\,=}$	7.0	kips	

Joint F

(F1+FR)/2	$V_{2\,=}$	7.9	kips	
FKF	$F_{FK\,=}$	1.0	kips	C
V2*H/2	$M_{FC\,=}$	59.4	kip*ft	
MKF	$M_{FK\,=}$	12.1	kip*ft	
FKF	$F_{FK\,=}$	1.01	kips	
FGF	$F_{FG\,=}$	4.70	kips	T
$\sum$M	$M_{FG\,=}$	71.6	kip*ft	
$\sum$M/W	$V_{G\,=}$	6.0	kips	
$\sum$Fy=0	$F_{FC\,=}$	7.0	kips	

3. RESULTS

	Member	Frame	Floor	Function	Force	T/C	Moment
	(#)	*(type)*	*(Units)*		*(kip)*		*(kip.ft)*
GF	29	Moment	First	Beam	4.70	T	71.55
JK	26	Moment	Roof	Beam	1.62	C	12.11
BG	21	Moment	First	Column	6.97	T	59.44
CF	23	Moment	First	Column	6.97	C	59.44
JG	22	Moment	Roof	Column	4.70	T	12.11
KF	24	Moment	Roof	Column	4.70	C	12.11

1. MOMENT FRAME INFORMATION

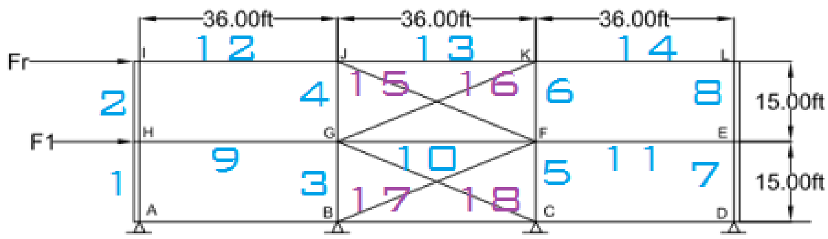


Figure 1 - Braced-Frame Member Reference

Diagonal, D	39	ft	
Story Height, H	15	ft	
Bay Width, W	36	ft	
F_r	9.78	kip	6.44
F_1	19.57	kip	0.00

2.1 MEMBER FORCES DISTRIBUTION

Find Reactions

Cy=		kip	up
By=	0.00	kip	down
Cx=	-14.68	kip	west
Bx=	-14.68	kip	west

Joint B

(Fr+F1)/2	V_1	14.68	kip	
	F_{BFy}	-6.12	kip	
Brace Force	F_{BF}	15.90	kip	T
Vertical Force	F_{BG}	-6.1152	kip	T

Joint C

V_1	14.68	kip	
F_{GCy}	-6.12	kip	C
F_{GC}	15.90	kip	C
F_{FC}	-6.12	kip	C

To solve system:

Moment Equation	15	F_{GF}	27.69	F_{GK}	-36	F_{GJ}	=	-72.06	513.69
Forces in X	-1	F_{GF}	-0.923	F_{GK}	0	F_{GJ}	=	-4.8	-4.89
Forces in Y	0	F_{GF}	-0.385	F_{GK}	1	F_{GJ}	=	4	-12

Inverse Matrix

61.533	922	2215.2
-66.666	-1000	-2400.0
-25.666	-385	-923.0

Solution

F_{GF}	1.13
F_{GK}	3.95
F_{GJ}	5.49

Joint G

Brace Force	F_{GK}	3.95	kip	C
Horizontal Force	F_{GF}	1.13	kip	C
Vertical Force	F_{GJ}	5.49	kip	T

Joint F

F_{FJ}	14.67	kip	C
F_{GF}	1.13	kip	C
F_{FK}	1.52	kip	T

Joint J

Joint K

Brace Force	F_{JF}	14.7	kip	C	F_{GK}	4.0	kip	C
Horizontal Force	$\mathbf{F_{JK}}$	3.6	kip	T	F_{KJ}	3.6	kip	T
Vertical Force	F_{JG}	5.49	kip	T	F_{KF}	1.5	kip	T

3. RESULTS

	Member (#)	Frame (type)	Floor (Units)	Function	Force (kip)	T/C	Moment (kip.ft)
GF	10	Braced	First	Beam	14.67	C	
JK	13	Braced	Roof	Beam	3.65	T	
JF	15	Braced	Roof	Brace	14.67	C	
GK	16	Braced	Roof	Brace	3.95	C	
BF	17	Braced	First	Brace	15.90	T	
GC	18	Braced	First	Brace	15.90	C	
BG	3	Braced	First	Column	-6.12	T	
JG	4	Braced	Roof	Column	5.49	T	
CF	5	Braced	First	Column	-6.12	C	
KF	6	Braced	Roof	Column	1.52	T	

1. MOMENT FRAME INFORMATION

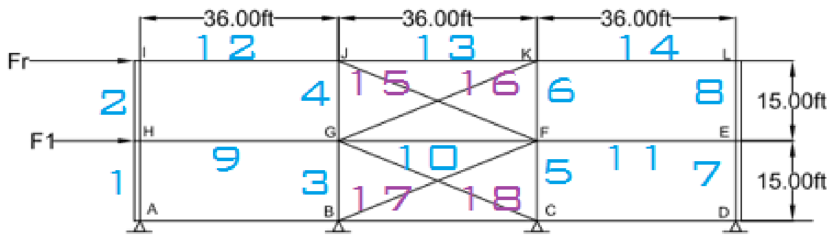


Figure 1 - Braced-Frame Member Reference

Diagonal, D	39	ft
Story Height, H	15	ft
Bay Width, W	36	ft
F_r	4.31	kip
F_1	16.82	kip

2.1 MEMBER FORCES DISTRIBUTION

Find Reactions

Cy=		kip	up
By=	0	kip	down
Cx=	-10.565	kip	west
Bx=	-10.565	kip	west

Joint B

(Fr+F1)/2	V_1	10.57	kip	
	F_{BFy}	-4.40	kip	
Brace Force	F_{BF}	11.45	kip	T
Vertical Force	F_{BG}	-4.40	kip	T

Joint C

V_1	10.57	kip	
F_{GCy}	-4.40	kip	C
F_{GC}	11.45	kip	C
F_{FC}	-4.40	kip	C

To solve system:

Moment Equation	15	F_{GF}	27.69	F_{GK}	-36	F_{GJ}	=	29.1	#####
Forces in X	-1	F_{GF}	-0.923	F_{GK}	0	F_{GJ}	=	-6.25	-6.26
Forces in Y	0	F_{GF}	-0.385	F_{GK}	1	F_{GJ}	=	1.796	-8.80

Inverse Matrix

61.533	922	2215.2
-66.666	-1000	-2400.0
-25.666	-385	-923.0

Solution

F_{GF}	6.61
F_{GK}	-0.38
F_{GJ}	1.66

Joint G

Brace Force	F_{GK}	-0.38	kip	C
Horizontal Force	F_{GF}	6.61	kip	C
Vertical Force	F_{GJ}	1.66	kip	T

Joint F

F_{FJ}	4.29	kip	C
F_{GF}	6.61	kip	C
F_{FK}	-0.15	kip	T

Joint J

Joint K

Brace Force	F_{JF}	4.3	kip	C	F_{GK}	-0.4	kip	C
Horizontal Force	$\mathbf{F_{JK}}$	-0.4	kip	T	F_{KJ}	-0.4	kip	T
Vertical Force	F_{JG}	1.66	kip	T	F_{KF}	-0.1	kip	T

3. RESULTS

	Member *(#)*	Frame *(type)*	Floor *(Units)*	Function	Force *(kip)*	T/C	Moment *(kip.ft)*
GF	10	Braced	First	Beam	4.29	C	
JK	13	Braced	Roof	Beam	-0.35	T	
JF	15	Braced	Roof	Brace	4.29	C	
GK	16	Braced	Roof	Brace	-0.38	C	
BF	17	Braced	First	Brace	11.45	T	
GC	18	Braced	First	Brace	11.45	C	
BG	3	Braced	First	Column	-4.40	T	
JG	4	Braced	Roof	Column	1.66	T	
CF	5	Braced	First	Column	-4.40	C	
KF	6	Braced	Roof	Column	-0.15	T	

GIVEN:

						First	**Roof**
Number of Floors:	2	**Number of Bays/Rov**	3	**Number of Rows:**		2	3
Beam Length	36	**Girder Length**	24	**Interior Girder**		48	
Short Column Height	15	**Long Column Height**	30				

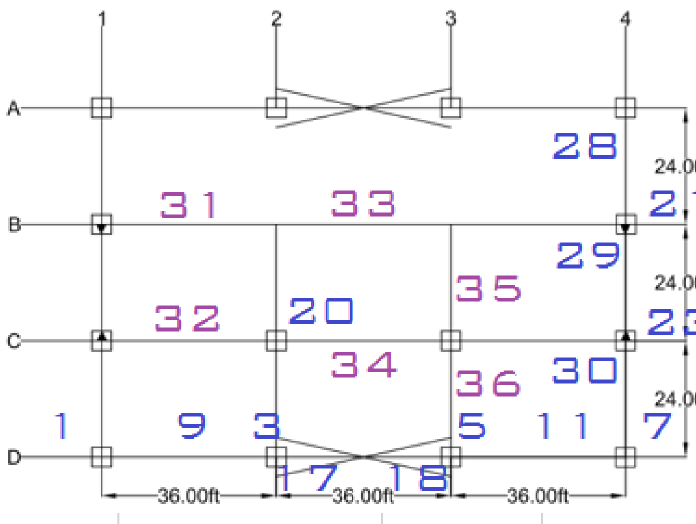


Figure 1 - First Floor Plan

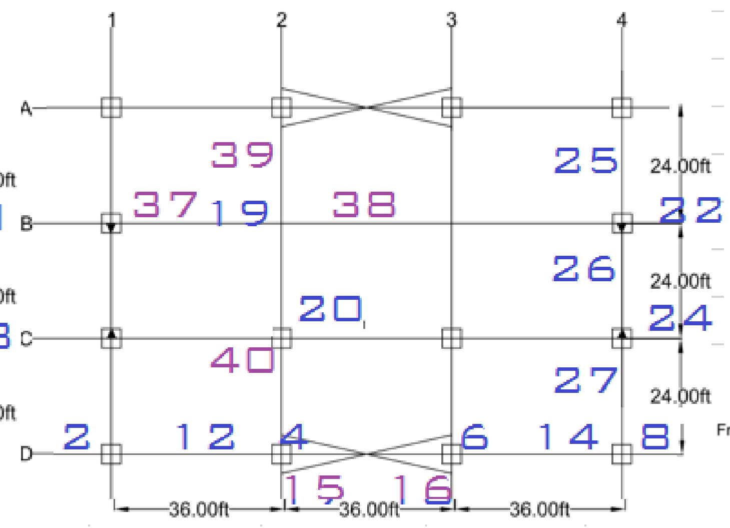


Figure 2 - Roof Floor Plan

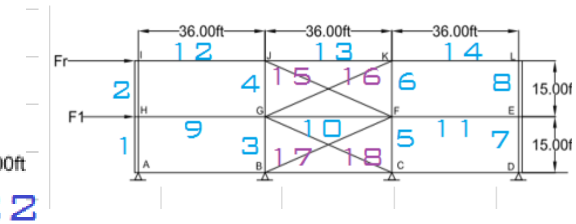


Figure 3 - Braced-Elevation

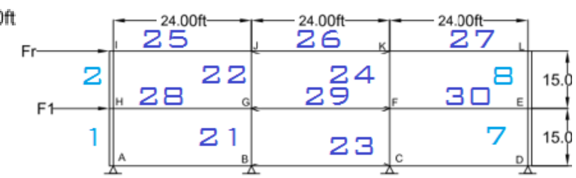


Figure 4 - Moment Frame - Elevation

1. MEMBERS FOR ANALYSIS:

Member Ref. (#)	Frame (type)	Floor (Units)	Member	Section (Shape)	New Section	Length (ft)	Unit Weight (plf)	Spacing or a (ft)	Beams/Bay (Units)	Qty of Members (Units)	Amount of Steel (kips)	New Amount (kips)	Difference (%)
1	Braced	First	Column	W8X40	W12X50	15	40	12		2	1.20	1.50	0.25
2	Braced	Roof	Column	W8X40	W12X50	15	40	1.7		2	1.20	1.50	0.25
3	Braced	First	Column	W12X40	W12X50	15	40	5		2	1.20	1.50	0.25
4	Braced	Roof	Column	W12X40	W12X40	15	40			2	1.20	1.20	0
5	Braced	First	Column	W12X40	W12X50	15	40			2	1.20	1.50	0.25
6	Braced	Roof	Column	W12X40	W12X50	15	40			2	1.20	1.50	0.25
7	Braced	First	Column	W8X40	W12X50	15	40			2	1.20	1.50	0.25
8	Braced	Roof	Column	W8X40	W12X50	15	40			2	1.20	1.50	0.25
1m	Moment	First	Column	W8X40	W10X68	15	40			2	1.20	2.04	0.7
2m	Moment	Roof	Column	W8X40	W10X68	15	40			2	1.20	2.04	0.7
7m	Moment	First	Column	W8X40	W10X68	15	40			2	1.20	2.04	0.7
8m	Moment	Roof	Column	W8X40	W10X68	15	40			2	1.20	2.04	0.7
9	Braced	First	Beam	W14X30	W14X30	36	30			2	2.16	2.16	0
10	Braced	First	nterior Bean	W14X30	W14X30	36	30			2	2.16	2.16	0
11	Braced	First	Beam	W14X30	W14X30	36	30			2	2.16	2.16	0
12	Braced	Roof	Beam	W10X19	W10X19	36	19			2	1.37	1.37	0
13	Braced	Roof	nterior Bean	W10X19	W10X19	36	19			2	1.37	1.37	0
14	Braced	Roof	Beam	W10X19	W10X19	36	19			2	1.37	1.37	0
15	Braced	Roof	Braces	WT9X48.5	W10X33	39	48.5			2	3.78	2.57	-0.32
16	Braced	Roof	Braces	WT9X48.5	W10X33	39	48.5			2	3.78	2.57	-0.32
17	Braced	First	Braces	WT9X48.5	W10X33	39	48.5			2	3.78	2.57	-0.32
18	Braced	First	Braces	WT9X48.5	W10X33	39	48.5			2	3.78	2.57	-0.32
19	Interior	Roof	Roof Columr	W8X40	W8X40	15	40			2	1.20	1.20	0.00
20	Interior	First	ıterior Colum	W8X40	W8X40	30	40			2	2.40	2.40	0.00
21	Moment	First	Column	W10X39	W10X68	15	39			2	1.17	2.04	0.74
22	Moment	Roof	Column	W10X39	W10X68	15	39			2	1.17	2.04	0.74

23	Moment	First	Column	W10X39	W10X68	15	39			2	1.17	2.04	0.74
24	Moment	Roof	Column	W10X39	W10X68	15	39	5		2	1.17	2.04	0.74
25	Moment	Roof	Beam	W14X30	W14X30	24	30	8		2	1.44	1.44	0.00
26	Moment	Roof	nterior Bean	W8X40	W10X39	24	40			2	1.92	1.87	-0.02
27	Moment	Roof	Beam	W14X30	W14X30	24	30			2	1.44	1.44	0.00
28	Moment	First	Beam	W8X40	W8X40	24	40			2	1.92	1.92	0.00
29	Moment	First	nterior Bean	W8X40	W12X53	24	40			2	1.92	2.54	0.33
30	Moment	First	Beam	W8X40	W8X40	24	40			2	1.92	1.92	0.00
31	Interior	First	Beam	W21X44	W21X44	36	44			0	0.00	0.00	
32	Interior	First	Beam	W21X44	W21X44	36	44			0	0.00	0.00	
33	Interior	First	nterior Bean	W21X44	W21X44	36	44			0	0.00	0.00	
34	Interior	First	nterior Bean	W21X44	W21X44	36	44	12	6.00	18	28.51	28.51	0.00
35	Interior	First	Girder	W30X108	W30X108	24	108	8		2	5.18	5.18	0.00
36	Interior	First	Girder	W30X108	W30X108	24	108	8		2	5.18	5.18	0.00
37	Interior	Roof	Beam	W14X22	W14X22	36	22			0	0.00	0.00	
38	Interior	Roof	nterior Bean	W14X22	W14X22	36	22	12	8.00	24	19.01	19.01	0.00
39	Interior	Roof	Girder	W27X84	W27X84	48	84			2	8.06	8.06	0.00
40	Interior	First	Girder	W27X84	W27X84	24	84			2	4.03	4.03	0
TOTAL										118	128.94	133.62	6.55

Moment Frame Connection Design-Roof Level

Given:

Girder	W10x39		**Column**	W10x68	
depth, d	9.92	in	depth, d	10.4	in
t_f	0.53	in	t_f	0.77	in
b_f	7.99	in	b_f	10.1	in
t_w	0.315	in	t_w	0.47	in
			k	1.27	in

Flange Plates

t	1/2	in	F_y	36	ksi	F_u	58	ksi
w	8	in	<0.15*bf (column)?	CHECK FLANGE LOCAL BENDING			J10.1	
unbraced length, L_p	3	in						
# of rows	2		Gage	6	in	Spacing	3	in
A325N, bolt dia.	7/8	in	Hole	std		# bolts	4	per flange
						Conn. Length	3	in
						Edge Distance	2	in

Flange-Column Weld

Electrode Strength, F_u	70	ksi
Flange CJP Weld:		
Length	8	in
Size	1/2	in

Double Angle Connection

Thickness, t	1/4	in	weld size	1/4	in	
# bolts/row, n	3		Weld Lengt	8.5	in	(L)
A325N, bolt dia.	3/4	in	Hole	std		

Beam End Forces (LRFD)

Shear, V	35.73	kips
Moment, M	77.18	kip*ft

Determine:

Connection adequacy and need for column stiffeners

Solution

Determine Flange and Web Shear Forces

Flange Force, T=C	93.36	kips	M/d
Web Shear Force	35.73	kips	Given

Tensile Strength of Flange Plate

a. Tensile Yield, P_n

Gross Area, A_g	4	in^2	
Nom. Tensile Yield, P_n	144	kips	Eq D2-1
Φ	0.9		
ΦP_n	129.6	kips	
	OK		

b. Tensile Rupture, P_n

Effective Area, A_e	3.00	in^2
Nom. Tensile Rupt., P_n	174.00	kips
Φ	0.75	
ΦP_n	130.50	kips
	OK	

Flange Plate Block Shear

A_{nt}	1.00	in^2
A_{nv}	7.00	in^2
A_{gv}	10.00	in^2
x=y for WT6x26.5	1.02	in
U	1.00	

$0.6*F_u*A_{nv}+U*F_u*A_{nt}$	$0.6*F_y*A_{gv}+U*F_u*A_{nt}$	Eq J4-5
301.6 kips	274 kips	

R_n	274.00	kips
Φ	0.75	
ΦR_n	205.50	kips
	OK	

Flange Plate Bolt Bearing

Find Minimum clear distance, L_c

Interior L_c	2.00	in			
Edge L_c	1.50	in			
Min L_c	1.50	in			
	Tearout		Deformation		
	52.20	kips/bolt	60.9	kips/bolt	Eq J3-6A
Nominal Strength, R_n	52.20	kips/bolt			
Nominal Plate Bearing Strength, P_n	208.8	kips			
Φ	0.75				
ΦP_n	156.6	kips			
	OK				

Bolt Shear

Bolt Area, A_b	0.60	in^2	
A325N, F_{nv}	54	ksi	Table J3.2
R_n	32.47	kips/bolt	
Bolt Shear Strength	129.89	kips	
Φ	0.75		
ΦP_n	97.41		
	OK		

Flange Plate Compressive Strength

K	0.65		Table C-A-7.1
r_y	0.14	in	t/sqrt(12)
KL_p/r	13.51		Eq J-4-6 Applies
Effective Plate width, l_w	8	in	
Effective gross area, A_{ge}	4	in^2	
P_n	144	kips	Nominal Compressive Strength, Eq J4-6
F_{cr}	32.3	ksi	Table 4-22
Φ	0.9		
ΦP_n	129.6		
	OK		

Evaluate Double-angle connection

Strength of welds	48.1	kips			Table 10-2 (Case II)
	OK				
Bolt/Angle Strength	76.4	kips			Table 10-1
	OK				
Bolt bearing on web	263	k/in	82.85	kips	Table 10-1
	OK				

Check Column with concentrated forces

Flange local bending

R_n	133.40	kips	J10-1
Φ	0.9		
ΦR_n	120.06	kips	
	OK		

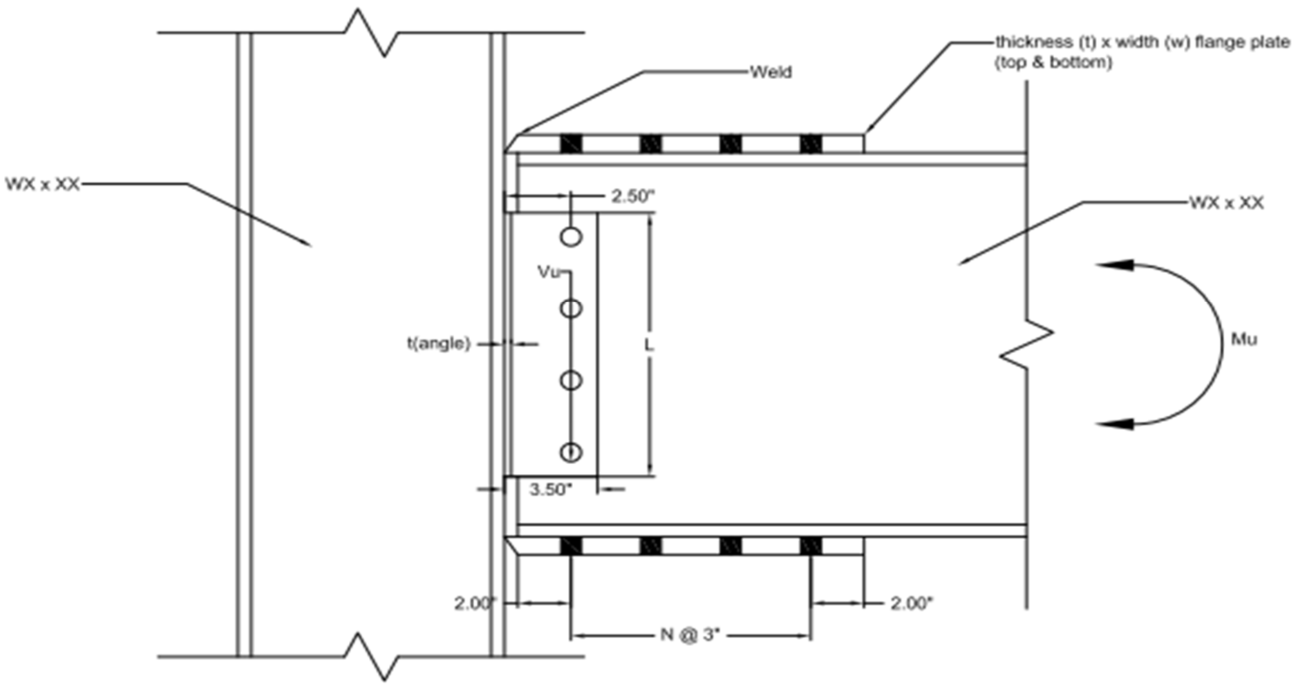
Web Local Yielding

R_n	104.48	kips	J10-3
Φ	1		
ΦR_n	104.48	kips	
	OK		

Web Crippling Parameters

R_n	326.50		J10-4
Φ	0.75		
ΦR_n	244.87		
	OK		

Summary


Weld
thickness (t) x width (w) flange plate
(top & bottom)
WX x XX
2.50"
WX x XX
Vu
t(angle)
L
Mu
3.50"
2.00"
2.00"
N @ 3"

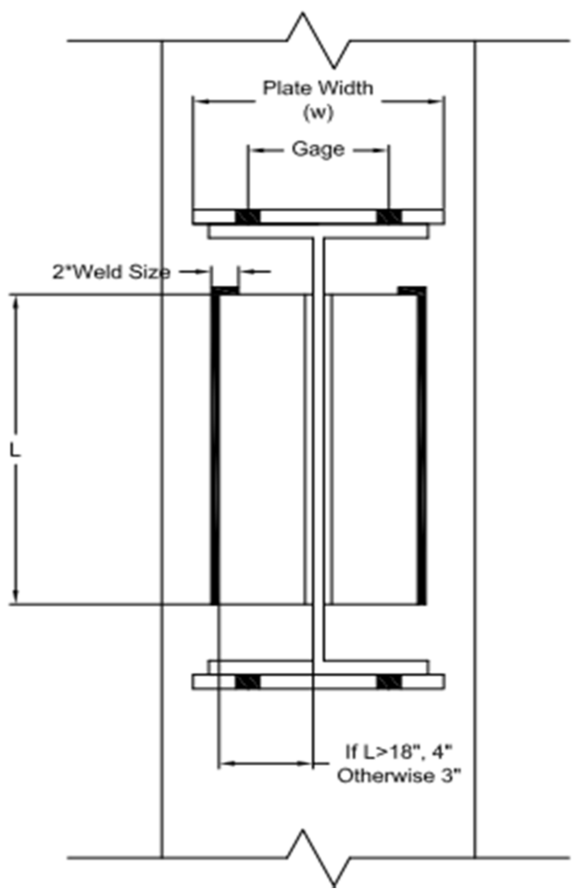


Plate Width
(w)
Gage
2"Weld Size
L
If L>18", 4"
Otherwise 3"

Girder:	W10x39	M_u	77.18 kip*ft
Column:	W10x68	V_u	35.73 kips

Flange Plate:

Thickness, t	1/2	in	F_y	36	ksi	F_u	58 ksi
width, w	8	in					
# of rows	2						
Hole	std						
A325N, bolt dia.	7/8	in					
# bolts	4	per flange					
Gage	6	in					

Flange-Column Weld:

Electrode Strength, F_u	70	ksi
Flange CJP Weld:		
Length	8	in
Size	1/2	in

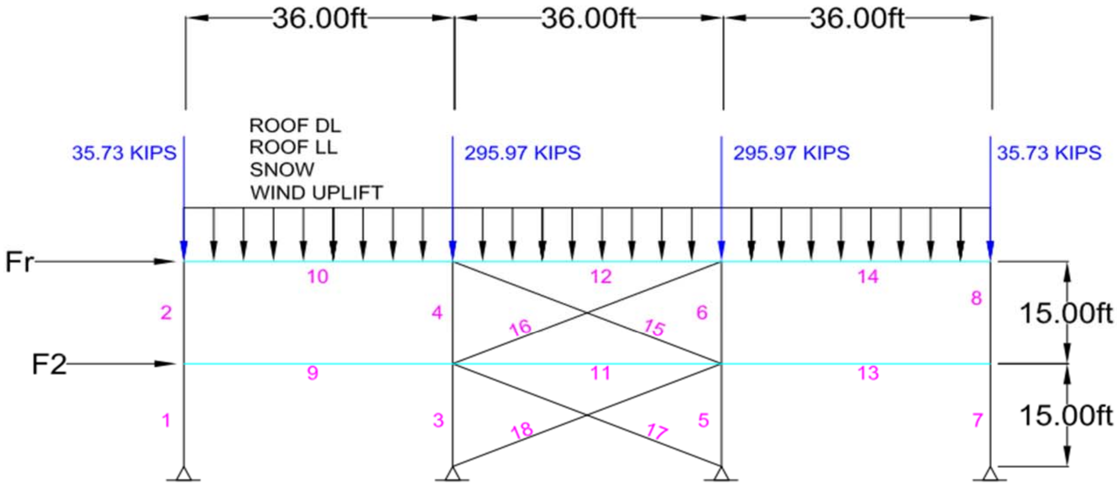
Double Angle Connection:

Weld to column, bolt to girder

Angle properties:

t	1/4	in
L	8.5	in
Leg Connect. to Column	3	in
Leg connect. To girder	3.5	in
weld size	1/4	in
Hole	std	
# bolts/row, n	3	(Bolts spaced at 3" O.C.)
A325N, bolt dia.	3/4	in

North Braced Frame Design



NORTH BRACED FRAME
(COLUMN LINE A)

F_y	50	ksi

Sizing (From 30%)

Level 2 Beams/Girders	W14x132
Roof Level Beams/Girders	W14x90
Columns	W12x40
Bracing	W10x33

Vertical Loading

Roof

Roof Live Load, W_{RLL}	0.6	klf
Snow, W_s	0.38	klf
Wind Uplift, W_{WU}	-0.28	klf
Roof Dead Load=self weight of beam		

Level 2

Floor Dead Load, W_{DL}	1.12	klf
Floor Live Load, W_{FIL}	1.2	klf
Girder Reactions on Interior Columns	295.97	kips
Girder Reactions on Exterior Columns	35.73	kips

Lateral Loading

Seismic Force, Roof Level	4.31	kips
Seismic Force, Level 2	16.82	kips
Wind Force, Roof Level	6.27	kips
Wind Force, Level 2	12.54	kips

Determine Notional Loads

Roof

Roof Live Load	0.6	klf
Snow Load	0.38	klf
Wind Uplift	-0.28	klf
Dead Load (Beam Self Weight)	0.09	klf
$W_{u\text{-}roof}$	1.07	klf
Girder Reactions	331.70	kips

Level 2

Floor System Load	1.12	klf	
Beam Self weight	0.132	klf	
Dead Load	1.25	klf	
Live Load	1.20	klf	
$W_{u\text{-}2}$	3.42	klf	
Y_R	447.26	kips	
Y_2	369.36	kips	
α	1		
N_R	0.895	kips	Eq C2-2
N_2	0.739	kips	Eq C2-1

Eccentricity

Column Web Thickness, t_w	0.295	in
Connection Distance (Assumed)	2.5	in
e	2.6475	**in**
Say 3" eccentricity		

Run Computer model in SAP2000

Design of Bottom Columns (Members 3 & 5)

First Order Analysis Forces	Top		Bottom	
Ultimate Axial Load, P_{unt}	283.85	kips	283.85	kips
Ultimate Moment, M_{unt}	14.2	kip*ft	0	kip*ft

Unbraced Length, L_b	15	ft			
Braced Frame Bay Width, W_b	36	ft	Plan Length, L	108	ft
Moment Frame Bay Width, W_m	24	ft	Plan Width, W	72	ft

Section Used *W12x50*

Section Properties (Table 1-1)

Plastic Section Modulus strong axis, Z_x	71.9	in^3	S_x	64.2	in^3
Moment of Inertia, I_x	391	in^4	r_{ts}	2.25	in
r_x	5.18	in	h_o	11.6	in
A_g	14.6	in2	J	1.71	in^4
b_f	8.08	in	c_w	1880	in^6
t_f	0.64	in			
h	9.25	in			
t_w	0.37	in			

Beam Properties

Full plastic yielding unbraced length, L_p	6.92	ft	Table 3-2
Elastic/Inelastic LTB unbraced length, L_r	23.8	ft	Table 3-2
ϕBF	5.98	kips	Table 3-2
ϕM_{px}	270	kip*ft	Table 3-2

Slenderness Characteristics	Flexure λ_p	Flexure λ_r	Compression λ_r
Flanges	9.15	24.08	13.49
	NONCOMPACT		NONSLENDER
Web	90.55	137.27	35.88
	COMPACT		NONSLENDER

Axial Capacity

K	1		K=1 for columns in DAM
Kl	15	ft	
P_c	355	kips	Table 4-1

Flexure Capacity

Zone?	ZONE 2			
M_{max}	14.20	kip*ft	M_{ult}	
M_A	4.50	kip*ft	SAP	
M_B	10.40	kip*ft	SAP	
M_C	15.59	kip*ft	SAP	
C_b	1.29			Eq F1-1
Zone 1, M_c	270	kip*ft		Eq F2-1
Zone 2, M_c	270.00	kip*ft		Eq F2-2
Zone 3, F_{cr}	84.66	ksi		Eq F2-4
Zone 3, M_c	452.92	kip*ft		Eq F2-3
M_c	270.00	kip*ft		

Determine τ_b

P_r	283.85	kips	P_{unt}
P_y	730.00	kips	A_g*F_y
α	1		
$\alpha P_r/P_y$	0.39		
τ_b	1.00		Eq C2-2a

Determine B1

P_{e1}	2763.25	kips	*Eq A-8-5*
Modification Coefficient, C_{mx}	0.60		*0.6-0.4*(M1/M2)*
B_1	1.00		*Eq A-8-3*

Summary

M_r	14.20	kip*ft	*Eq A-8-1*
M_c	270.00	kip*ft	*Referenced Above*
P_r	283.9	kips	*Eq A-8-2*
P_c	355	kips	*Referenced Above*

Combined Forces Ineraction Equation

P_r/P_c	0.80		
$P_r/P_c>0.2$	0.85		Eq H1-1a
$P_r/P_c<0.2$	0.45		Eq H1-1b
Design Check	0.85	**SECTION OK**	

Design of Top Columns (Members 4 & 6)

First Order Analysis Forces	Top		Bottom	
Ultimate Axial Load, P_{unt}	284.8	kips	284.8	kips
Ultimate Moment, M_{unt}	15.09	kip*ft	0	kip*ft

Unbraced Length, L_b	15	ft

Braced Frame Bay Width, W_b	36	ft	Plan Length, L	108	ft
Moment Frame Bay Width, W_m	24	ft	Plan Width, W	72	ft

Section Used	*W12x50*

Section Properties (Table 1-1)

Plastic Section Modulus strong axis, Z_x	71.9	in^3	S_x	64.2	in^3
Moment of Inertia, I_x	391	in^4	r_{ts}	2.25	in
r_x	5.18	in	h_o	11.6	in
A_g	14.6	in2	J	1.71	in^4
b_f	8.08	in	c_w	1880	in^6
t_f	0.64	in			
h	9.25	in			
t_w	0.37	in			

Beam Properties

Full plastic yielding unbraced length, L_p	6.92	ft	Table 3-2
Elastic/Inelastic LTB unbraced length, L_r	23.8	ft	Table 3-2
ϕBF	5.98	kips	Table 3-2
ϕM_{px}	270	kip*ft	Table 3-2

	Flexure		Compression
Slenderness Characteristics	λ_p	λ_r	λ_r
Flanges	9.15	24.08	13.49
	NONCOMPACT		NONSLENDER
Web	90.55	137.27	35.88
	COMPACT		NONSLENDER

Axial Capacity

K	1		K=1 for columns in DAM
Kl	15	ft	
P_c	355	kips	Table 4-1

Flexure Capacity

Zone?	ZONE 2		
M_{max}	15.09	kip*ft	M_{ult}
M_A	5.42	kip*ft	SAP
M_B	10.84	kip*ft	SAP
M_C	16.26	kip*ft	SAP
C_b	1.29		Eq F1-1
Zone 1, M_c	270	kip*ft	Eq F2-1
Zone 2, M_c	270.00	kip*ft	Eq F2-2
Zone 3, F_{cr}	84.57	ksi	Eq F2-4
Zone 3, M_c	452.47	kip*ft	Eq F2-3
M_c	270.00	kip*ft	

Determine τ_b

P_r	284.80	kips	P_{unt}
P_y	730.00	kips	A_g*F_y
α	1		
$\alpha P_r/P_y$	0.39		
τ_b	1.00		Eq C2-2a

Determine B1

P_{e1}	2763.25	kips	*Eq A-8-5*
Modification Coefficient, C_{mx}	0.60		*0.6-0.4*(M1/M2)*
B_1	1.00		*Eq A-8-3*

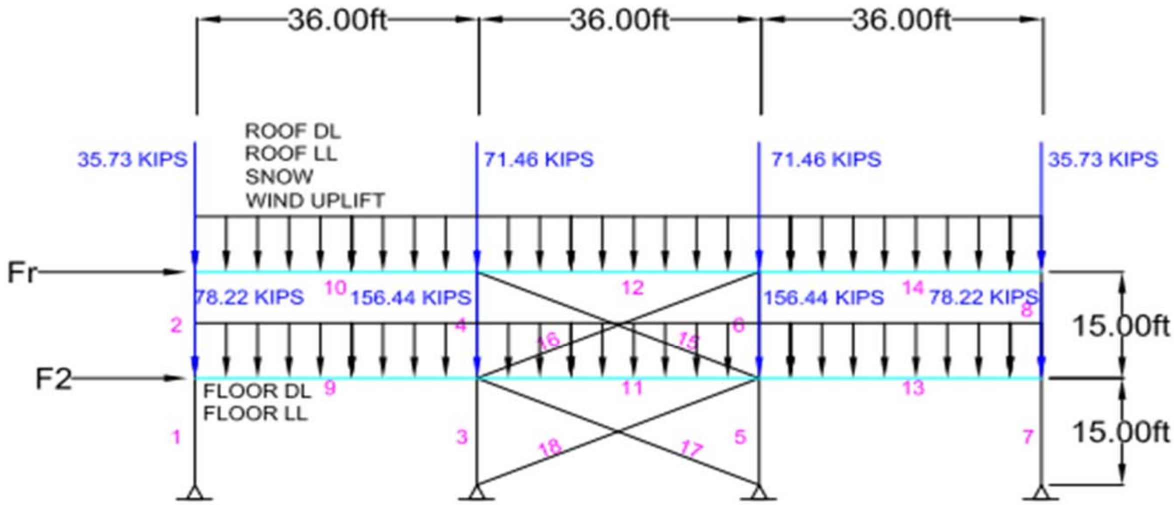
Summary

M_r	15.09	kip*ft	*Eq A-8-1*
M_c	270.00	kip*ft	*Referenced Above*
P_r	284.8	kips	*Eq A-8-2*
P_c	355	kips	*Referenced Above*

Combined Forces Ineraction Equation

P_r/P_c	0.80		
$P_r/P_c>0.2$	0.85		Eq H1-1a
$P_r/P_c<0.2$	0.46		Eq H1-1b
Design Check	0.85	**SECTION OK**	

South Braced Frame Design



SOUTH BRACED FRAME
(COLUMN LINE D)

F_y	50	ksi

Sizing (From 30%)

Level 2 Beams/Girders	W14x132
Roof Level Beams/Girders	W14x90
Columns	W12x40
Bracing	W10x33

Vertical Loading

Roof		
Roof Live Load, W_{RLL}	0.6	klf
Snow, W_s	0.38	klf
Wind Uplift, W_{WU}	-0.28	klf
Roof Dead Load=self weight of beam		
Girder Reactions on Interior Columns	71.46	kips
Girder Reactions on Exterior Columns	35.73	kips
Level 2		
Floor Dead Load, W_{DL}	1.12	klf
Floor Live Load, W_{FIL}	1.2	klf
Girder Reactions on Interior Columns	156.44	kips
Girder Reactions on Exterior Columns	78.22	kips

Lateral Loading

Seismic Force, Roof Level	4.31	kips
Seismic Force, Level 2	16.82	kips
Wind Force, Roof Level	6.27	kips
Wind Force, Level 2	12.54	kips

Determine Notional Loads

Roof

Roof Live Load	0.6	klf
Snow Load	0.38	klf
Wind Uplift	-0.28	klf
Dead Load (Beam Self Weight)	0.09	klf
$W_{u\text{-}roof}$	1.07	klf
Girder Reactions	107.19	kips

Level 2

Floor System Load	1.12	klf	
Beam Self weight	0.132	klf	
Dead Load	1.25	klf	
Live Load	1.20	klf	
$W_{u\text{-}2}$	3.42	klf	
Girder Reactions	234.66	kips	
Y_R	222.75	kips	
Y_2	369.36	kips	
α	1		
N_R	0.446	kips	Eq C2-2
N_2	0.739	kips	Eq C2-1

Eccentricity

Column Web Thickness, t_w	0.295	in
Connection Distance (Assumed)	2.5	in
e	2.6475	**in**
Say 3" eccentricity		

Run Computer model in SAP2000

Design of Bottom Columns (Members 3 & 5)

First Order Analysis Forces	Top		Bottom	
Ultimate Axial Load, P_{unt}	295.93	kips	295.93	kips
Ultimate Moment, M_{unt}	13.37	kip*ft	0	kip*ft

Unbraced Length, L_b	15	ft			
Braced Frame Bay Width, W_b	36	ft	Plan Length, L	108	ft
Moment Frame Bay Width, W_m	24	ft	Plan Width, W	72	ft

Section Used *W12x50*

Section Properties (Table 1-1)

Plastic Section Modulus strong axis, Z_x	71.9	in^3	S_x	64.2	in^3
Moment of Inertia, I_x	391	in^4	r_{ts}	2.25	in
r_x	5.18	in	h_o	11.6	in
A_g	14.6	in2	J	1.71	in^4
b_f	8.08	in	c_w	1880	in^6
t_f	0.64	in			
h	9.25	in			
t_w	0.37	in			

Beam Properties

Full plastic yielding unbraced length, L_p	6.92	ft	Table 3-2
Elastic/Inelastic LTB unbraced length, L_r	23.8	ft	Table 3-2
ϕBF	5.98	kips	Table 3-2
ϕM_{px}	270	kip*ft	Table 3-2

	Flexure		Compression
Slenderness Characteristics	λ_p	λ_r	λ_r
Flanges	9.15	24.08	13.49
	NONCOMPACT		NONSLENDER
Web	90.55	137.27	35.88
	COMPACT		NONSLENDER

Axial Capacity

K	1		K=1 for columns in DAM
Kl	15	ft	
P_c	355	kips	Table 4-1

Flexure Capacity

Zone?	ZONE 2			
M_{max}	13.37	kip*ft	M_{ult}	
M_A	4.79	kip*ft	SAP	
M_B	9.58	kip*ft	SAP	
M_C	14.37	kip*ft	SAP	
C_b	1.29			Eq F1-1
Zone 1, M_c	270	kip*ft		Eq F2-1
Zone 2, M_c	270.00	kip*ft		Eq F2-2
Zone 3, F_{cr}	84.73	ksi		Eq F2-4
Zone 3, M_c	453.32	kip*ft		Eq F2-3
M_c	270.00	kip*ft		

Determine τ_b

P_r	295.93	kips	P_{unt}
P_y	730.00	kips	A_g*F_y
α	1		
$\alpha P_r/P_y$	0.41		
τ_b	1.00		Eq C2-2a

Determine B1

P_{e1}	2763.25	kips	*Eq A-8-5*
Modification Coefficient, C_{mx}	0.60		*0.6-0.4*(M1/M2)*
B_1	1.00		*Eq A-8-3*

Summary

M_r	13.37	kip*ft	*Eq A-8-1*
M_c	270.00	kip*ft	*Referenced Above*
P_r	295.9	kips	*Eq A-8-2*
P_c	355	kips	*Referenced Above*

Combined Forces Ineraction Equation

P_r/P_c	0.83		
P_r/P_c>0.2	0.88		Eq H1-1a
P_r/P_c<0.2	0.47		Eq H1-1b
Design Check	0.88	**SECTION OK**	

Design of Top Columns (Members 4 & 6)

First Order Analysis Forces	Top		Bottom	
Ultimate Axial Load, P_{unt}	103.15	kips	103.15	kips
Ultimate Moment, M_{unt}	12.57	kip*ft	0	kip*ft

Unbraced Length, L_b	15	ft			
Braced Frame Bay Width, W_b	36	ft	Plan Length, L	108	ft
Moment Frame Bay Width, W_m	24	ft	Plan Width, W	72	ft

Section Used: *W12x40*

Section Properties (Table 1-1)

Plastic Section Modulus strong axis, Z_x	57	in^3	S_x	51.5	in^3
Moment of Inertia, I_x	307	in^4	r_{ts}	2.21	in
r_x	5.13	in	h_o	11.4	in
A_g	11.7	in2	J	0.906	in^4
b_f	8.01	in	c_w	1440	in^6
t_f	0.515	in			
h	9.25	in			
t_w	0.295	in			

Beam Properties

Full plastic yielding unbraced length, L_p	6.85	ft	Table 3-2
Elastic/Inelastic LTB unbraced length, L_r	21.1	ft	Table 3-2
ϕBF	5.54	kips	Table 3-2
ϕM_{px}	214	kip*ft	Table 3-2

	Flexure		Compression
Slenderness Characteristics	λ_p	λ_r	λ_r
Flanges	9.15	24.08	13.49
	NONCOMPACT		SLENDER
Web	90.55	137.27	35.88
	COMPACT		NONSLENDER

Axial Capacity

K	1		K=1 for columns in DAM
Kl	15	ft	
P_c	281	kips	Table 4-1

Flexure Capacity

Zone?	ZONE 2			
M_{max}	12.57	kip*ft	M_{ult}	
M_A	14.37	kip*ft	SAP	
M_B	9.58	kip*ft	SAP	
M_C	4.79	kip*ft	SAP	
C_b	1.24			Eq F1-1
Zone 1, M_c	214	kip*ft		Eq F2-1
Zone 2, M_c	219.14	kip*ft		Eq F2-2
Zone 3, F_{cr}	71.46	ksi		Eq F2-4
Zone 3, M_c	306.68	kip*ft		Eq F2-3
M_c	219.14	kip*ft		

Determine τ_b

P_r	103.15	kips	P_{unt}
P_y	585.00	kips	A_g*F_y
α	1		
$\alpha P_r/P_y$	0.18		
τ_b	1.00		Eq C2-2a

Determine B1

P_{e1}	2169.61	kips	*Eq A-8-5*
Modification Coefficient, C_{mx}	0.60		*0.6-0.4*(M1/M2)*
B_1	1.00		*Eq A-8-3*

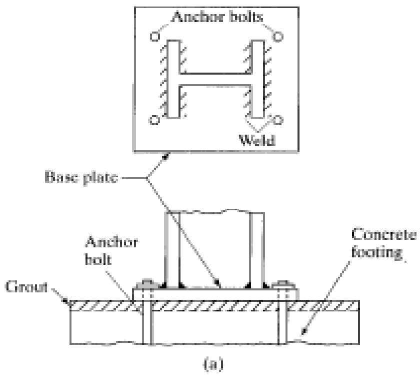
Summary

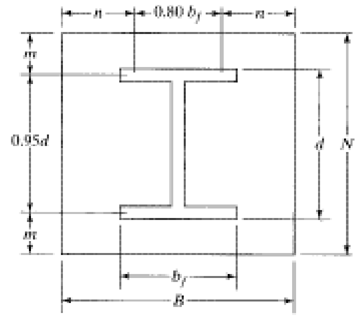
M_r	12.57	kip*ft	*Eq A-8-1*
M_c	219.14	kip*ft	*Referenced Above*
P_r	103.2	kips	*Eq A-8-2*
P_c	281	kips	*Referenced Above*

Combined Forces Ineraction Equation

P_r/P_c	0.37		
P_r/P_c>0.2	0.42		Eq H1-1a
P_r/P_c<0.2	0.24		Eq H1-1b
Design Check	0.42	**SECTION OK**	

CONNECTIONS





Member Ref:	23
Frame:	Moment
Floor:	First
Member:	Column
Ref. 2:	1-M

ASSUMPTIONS:

Footing dimensions	B=	9	ft
	L=	9	ft
	A=	81	ft^2

**Loads*

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
Shear Modulus:	G =	11200	ksi
Yield Strength:	$F_{y\,=}$	36	ksi
Concrete Compressive	f'c =	3	ksi

2. LOADS:

Dead Load	DL=	200	kip
Live Load	LL=	300	kip
Factors	ϕ_t=	0.9	
	ϕ_r=	0.75	
Compression	ϕ_c=	0.65	

LRFD

1) Demand:

Load	Pu =	295	kip

1. PREVIOUS MEMBER GEOMETRIC INFORMATION:

Demand

Previous Selection:		**W10X68**	
Plastic Modulus	Z =	85.3	in^3
Flange Width	bf =	10.1	in
Depth:	d =	10.4	in

2. NEW MEMBER GEOMETRIC INFORMATION:

Base Plate:				
Width	Bw =	12	in	15 *Project Information*
Length	Nl=	12	in	Least volume
Area	A =	144	in^2	
Thickness	t =	1.00	in	
Minimal Area Check:	Amin =	89.0		
	A>Amin ?	OK		
Ratio Check		31.18		
	V(A2/A1)=	OK		
Given Column Used:	m=	1.06	in	
	n=	1.96	in	
	n'=	2.56	in	
	l =	2.56	in	
Optimization	m~n			
	D =	0.90		
	N =	10.33		
	B =	8.61		
	Acheck=	88.99	in^2	
Check	bf.d =	105.04	in^2	
	Acheck>bf.df ?	YES		

Concrete Bearing Strength Check

$\phi_{c.}$ P=	477.36	k
$\phi_{c.}$ P > Pu ?	YES	

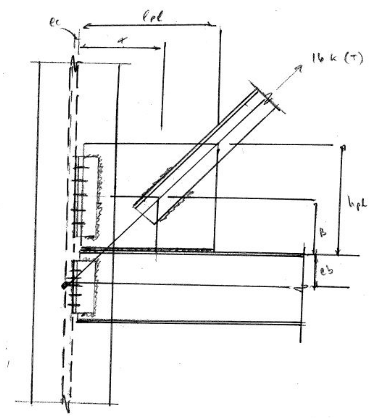
USE	**1**	**12.00**	**12**	in		**ANSWER**

Prepared by: Matthew Laskey Date: 11/29/2014
Checked by: Ana Gouveia

This calculation is representative of all second floor bracing conncections. This is due to the symmetrical values and the loading characteristics. The connection is also over designed, so any irregularities will be ok by inspection The tensile forces acting on the connection from the brace below. Will also reduce the forces on the connectior

Connection Information:

Grid Line:	2
Column Line:	A
Elevation:	2nd Floor



Story ht	15	ft
Run	36	ft
Angle from vert	67.38	deg

P(t/c)	16	k
E	29000	ksi

Column			Beam			Brace		
Section	W12x50		Section	W14x30		section	L6x4x.5	
dbm	12.2	in	dbm	13.8	in	wbr	6	in
Fy	50	ksi	Fy	50	ksi	Fy	46	ksi
tf	0.64	in	k	0.785	in	Weld1 (br to gpl)	3	/16 in
tw	0.37	in	tw	0.27	in	lw=lamin	6	in

Gusset plate		
hpl	18	in
lpl	12	in
tpl	0.375	in
fy	36	ksi
Weld2 (gpl to bm)	6	/16 in
L1 (whit sec)	12.40	in

Note: 1) highest edge of brace set at 2" above top of beam
2) left most edge of brace set at 1" from edge of connection angle

			Angle				Fy	Bolts	
			size			Length	Fu		
Connection angles	Desc	n rows	osl / gage	leg2 / weld	Thick	in	ksi	diam	spec
Gusset pl to column	dbl angle	5	4	3	0.5	15	36	0.75	A325
			2.81	3			58		
Beam to column	dbl angle	5	4	3	0.5	15	36	0.75	A325
			2.81	3			58		

Phi 0.75

1. Brace to gusset connection

lamin load 1.077586 in > 6 ? USE 6 in

Use lamin= 6 in

2. calc min plate size

Check lplmin (min length to fit brace)

lplmin=(leg2-.5"setback)+1"clear+(wbrcos(theta))+(lamin)sin(theta)

lplmin=	11.35	in	>	12	USE lpl stated
Use lplmin	12.00	=		12	in

Check hplmin (min height to fit brace)
hplmin= 2" clear + (wbrsin(theta))+(laminCos(theta))

hplmin=	9.85	in	>	18
Use hplmir	18.00	in		

3. Whitmore section check 0.449219

Lwmin= 12.9282 in

calc tension yielding on whit sec (assumes all Lw in Gpl)

Ra	130.8981	k	>	16 ?	Plate OK

calc compr buckling on whit sec of Gpl (assume all Lw in Gpl)

K	0.5				
L1	12.40				
r	0.108253				
KL/r	57.27279	check if Kl/r<>25 eq for Ra buclking			
Lambda c	0.642319				
Fcr	30.29056				
Ra buckling	110.1382	k	>	16 ?	Plate OK

Geometry and force parameters

tan(theta)=	2.399984	
eb=dbm/2	6.9	in
ec=dcol/2	0.185	in
Beta= (((ng-1)+3")/2)+	9	in

alphaideal=ebtan(theta)-ec+betatan(theta)

alphaideal	37.97	in

aphaactual= lp/2+.5"setback

alphaactua	6.5	in
Difference	-31.47	in

4. Forces

r=sqrt((alpahi+ec)^2+(beta+eb)^2)

r= 41.33977 in

At gsset to col

Vnc=(beta/r)Pn	3.483329	k
Hnc=(ec/r)Pn	0.071602	k

At gusset to beam

Vnb= (eb/r)Pn	2.670552	k
Hnb= (alpha ideal/r)Pn	14.69761	k

At beam to col

H= Hnc+Hnb	14.76922	k
V=Vnb+Vnc	6.153881	k

5. Gusset to column

Check bolts:

tensile force per bolt	rnt=Hc/2n	0.00716	k/bolt	<	19.9	allowable	Bolts OK
Shear force per bolt	rnv=Vnc/2	0.348333	k/bolt	<	7.38	(slip critical)	
				Bolts OK			
fv=rnv/Abolt		0.788464	ksi				

Check bearing strength at bolt holes

rbrg=phi*2.4*dbolt*tl*Fu	29.3625 k/bolt	>	0.348332885	Bearing strength OK

6. Check Angle

Prying action	b=g-tl/2	2.56	in
	a=osl-g	1.19	in
	b'=b-d/2	2.185	in
	a'=a+d/2	1.565	in
	p=b'/a'	1.396166	
	d'=d+1/16	0.8125	in
	p=L/n	3	in
	small delta=1-(d'/p)	0.729167	in

Beta = (1/p)*(B/T)-1) 3.581236 if Beta>1 set alpa'= 1

treq=sqrt((4.44*rnt*b')/(p*Fu*(1+deltaalpha')))

treq= 0.022273 in < 0.5 Angle thickness OK

7. Check Weld of Angle to Gusset Plate

fillet welds on 3 sides of both angles

Pnc=sqrt(Hnc^2+Vnc^2)

Pnc= 3.484065

Theta= arctan(Hnc/Vnc)

theta= 34.13 deg

Length of Dbl Angle= 15 in

kl= leg2-1/2" setback = 2.5 in

k=kl/l= 0.167

From Table 8-8 w/ theta=30 Find x by interpolation.

x= 0.022

xl= 0.33 in

al= leg2-xl 2.67 in

a=al/l= 0.178

Find C by interpolation:			0.1	0.167	0.2
C=	2.69	0.15	2.43	2.7784	2.95
		0.178		2.69	
ϕ=	0.75	0.2	2.29	2.625	2.79

ϕRn= 181.575 k

ϕRn> Pnc= 3.484065 Weld OK

8. Check strength of angles

Check shear yielding due to Vnc

ϕ= 1

Ag 1 angle= 7.5 in2

ϕRn= ϕ.6Fy2Ag= 324 k > Vnc?= 3.483328849 OK (J4-3)

Check Shear Rupture

ϕ= 0.75

Anv= 5.46875

ϕRn= ϕ.6Fu2Anv= 285.4688 k > Vnc?= 3.483328849 OK (J4-4)

Check Block Shear Rupture

ϕRn= ϕ[.6FuAnv + UbsFuAnt < .6FyAgv+UbsFuAnt]*2

ϕ= 0.75

Ubs= 1 Tension stress uniform

	.6FuAnv + UbsFuAnt=		507.5		
	.6FyAgv+UbsFuAnt=		479.1875		
ϕRn=	718.7813 k	>Vnc?=	3.483329 OK		(J4-5)

Check Column Flange

tflange= 0.64 > treq= 0.022273 OK

9. Gusset to Beam Design

Hnb=	14.69761 k	[From Above]
Vnb=	2.670552 k	[From Above]
Mb=Hbeb=	101.4135 kin	

Sx=(hpl-1)^2/3 96.33333 in^2 Weld treated as a line.

Check gusset stress

fv=Hb/lpltpl= 3.266137 < ϕ.6Fy?= 21.6 OK

fa= 0.773649 ksi <ϕFy= 36 OK

User note on pg 16.1-126: When required stress is less than 30% of available stress, combinec effects of stress need not be considered.

Weld Load

fr= 0.629346 k/in

For ductility multiply by 1.4 = 0.881085 k/in

Weld Capacity AISC Part 8

D=	0.375 in		
ϕ=	0.75		
θ=	71.10		
△u=	0.024192	< .17D=	0.6375
△i=	0.024192		
△m=	0.019848		
p=	1.218859		
f(p)=	0.993585		
Weld Cap=	12.11488 k/in	> 1.4fr?=	0.881 OK

Check beam web yielding @ gusset plate

fb= 1.286635 ksi <ϕFybm?= 37.5 OK

10. Beam to Column

Hnc=	14.76922	[From Above]
Vnc=	6.153881	[From Above]
n=	5	

Check Bolts

Abolt= 0.441786 in2

Tensile force per bolt

rnt= 1.476922 k/bolt < 19.9 OK

Shear force per bolt

rnv= 0.615388 k/bolt < 7.38 OK

fv= 1.392954 ksi

Tension-shear interaction

ft= 114.3534 < 90 Use 90

ϕrn= 29.82059 k >rnt? OK

Check Angle

Prying
Because using same sized angle, all values same as section 6.
treq= 0.022273 in

check weld of angles to column
fillet welds on 3 sides of both angles
Pnc= 16 k
θ= 30.00

Length of Dbl Angle= 15 in
kl= leg2-1/2" setback = 2.5 in
k=kl/l= 0.167
From Table 8-8 w/ theta=30 Find x by interpolation.
x= 0.022
xl= 0.33 in
al= leg2-xl 14.67 in
a=al/l= 0.978

Find C by interpolation:				0.1	0.167	0.2
C=	2.69		0.15	2.43	2.7784	2.95
C1=	1		0.178		2.69	
ϕ=	0.75		0.2	2.29	2.625	2.79
ϕRn=	181.575 k					

ϕRn> Pnc= 16 Weld OK

Check Strength of angles
Check shear yielding due to Vnc
ϕ= 1
Ag 1 angle= 7.5 in2
ϕRn= ϕ.6Fy2Ag= 324 k > Vnc?= 6.153880967 OK (J4-3)

Check Shear Rupture
ϕ= 0.75
Anv= 5.46875
ϕRn= ϕ.6Fu2Anv= 285.4688 k > Vnc?= 6.153880967 OK (J4-4)

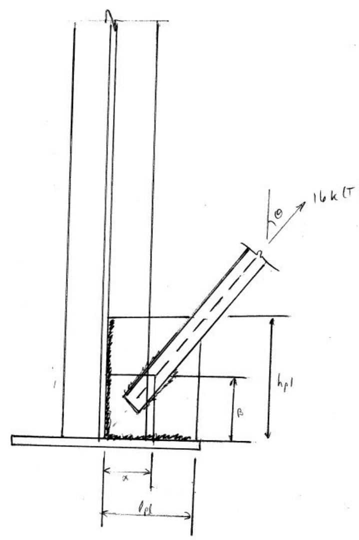
Check Block Shear Rupture
ϕRn= ϕ[.6FuAnv + UbsFuAnt < .6FyAgv+UbsFuAnt]*2
ϕ= 0.75
Ubs= 1 Tension stress uniform

.6FuAnv + UbsFuAnt= 507.5
.6FyAgv+UbsFuAnt= 479.1875
ϕRn= 718.7813 k >Vnc?= 6.153881 OK (J4-5)

Check Column Flange
tflange= 0.64 > treq= 0.022273 OK

Prepared by:	Matthew Laskey	Date:	11/29/2014
Checked by:	Ana Gouveia		

This calculation is representative of all first floor bracing conncections. This is due to the symmetrical values and the loading characteristics. The connection is also over designed, so any irregularities will be ok by inspection.



Connection Information:

Grid Line:	2
Column Line:	A
Elevation:	1st Floor

Story ht	15	ft
Run	36	ft
Angle from vert	67.38	deg

P(t/c)	16	k
E	29000	ksi

Column		
Section	W12x50	
dbm	12.2	in
Fy	50	ksi
tw	0.37	in
tbplate	0.75	in
wbplate	16	in

Brace		
section	L6x4x.5	
wbr	6	in
Fy	46	ksi
Weld1 (br to gpl)	3	/16 in
lw=lamin	8	in

Gusset plate		
hpl	18	in
lpl	12	in
tpl	0.375	in
fy	36	ksi
Weld2 (gpl to bm)	6	/16 in
L1 (whit sec)	12.40	in

Note: 1) highest edge of brace set at 2" above top of beam
2) left most edge of brace set at 1" from edge of connection angle

Phi 0.75

1. Brace to gusset connection

lamin load	1.077586	in >	6 ?	USE 6 in
Use lamin=	6	in		
U=	0.75			
Lamin=	8	in		

2. calc min plate size

Check lplmin (min length to fit brace)

lplmin=(leg2-.5"setback)+1"clear+(wbrcos(theta))+(lamin)sin(theta)

lplmin=	11.69	in	>	12	USE lpl stated	
Use lplmin	12.00	=		12	in	

Check hplmin (min height to fit brace)

hplmin= 2" clear + (wbrsin(theta))+(laminCos(theta))

hplmin=	10.62	in	>	18
Use hplmir	18.00	in		

3. Whitmore section check 0.449219

Lwmin= 15.2376 in

calc tension yielding on whit sec (assumes all Lw in Gpl)

Ra	154.2807	k	>	16	?	Plate OK

calc compr buckling on whit sec of Gpl (assume all Lw in Gpl)

K	0.5					
L1	12.40					
r	0.108253					
KL/r	57.27279					
Lambda c	0.642319					
Fcr	30.29056					
Ra buckling	129.8125	k	>	16	?	Plate OK

Geometry and force parameters

tan(theta)=	2.40	
eb=dbm/2	0.75	in
ec=dcol/2	0.375	in
Beta=	1.242722	in
alphaactua	4.41	in
Beta actl=	9	
alphaactua	4.41	in
Difference	7.76	in

4. Forces

r=sqrt((alpahi+ec)^2+(beta+eb)^2)

r= 10.85978 in

At gsset to col

Vnc=(beta/r)Pn	13.25994	k
Hnc=(ec/r)Pn	0.552497	k

At gusset to base plate

Vnb= (eb/r)Pn	1.104995	k
Hnb= (alpha ideal/r)Pn	6.493686	k

5. Gusset to column Design

Hnc=	0.552497	k	[From Above]
Vnc=	13.25994	k	[From Above]

Sx=(hpl-1)^2/3 96.33333 in^2 Weld treated as a line.

Check weld strength of vertical weld

fx=fa=	0.06074	k/in			
fy=fv=	0.389998	k/in			
fr=	0.3947	k/in			
For ductility, multiply by 1.4=		0.55258	k/in		
Weld Capacity	AISC Part 8				
D=	0.375	in			
ϕ=	0.75				
θ=	9.86				
△u=	0.067608	< .17D=	0.6375		
△i=	0.067608				
△m=	0.035516				
p=	1.903577				
f(p)=	0.733282				
Weld Cap=	8.94	k/in	> 1.4fr?=	0.553	OK

6. Gusset to Base Plate

Hnb=	6.493686	k	[From Above]
Vnb=	1.104995	k	[From Above]

Check weld strength of horizontal weld

lwh=	6.815	in			
fx=fv=	0.48	k/in			
fy=fa=	0.08	k/in			
fr=	0.48	k/in			
for ductility, multiply by 1.4=		0.677	k/in		
Weld Capacity	AISC Part 8				
D=	0.375	in			
ϕ=	0.75				
θ=	9.66				
△u=	0.068	< .17D=	0.6375		
△i=	0.068				
△m=	0.036				
p=	1.909				
f(p)=	0.727998				
Weld Cap=	8.88	k/in	> 1.4fr?=	0.677	OK

7. Gusset plate stresses

a. Normal load

normal load = larger of fxtop or fybott

fxtop=	0.06	
fybott=	0.08	
Normal load=	0.08	k/in

Plate stress= normalload* 2welds* 1in long/ tpl*1in long

plate stress=	0.432378	ksi	< ϕFy?=	32.4	OK

b. Shear Load

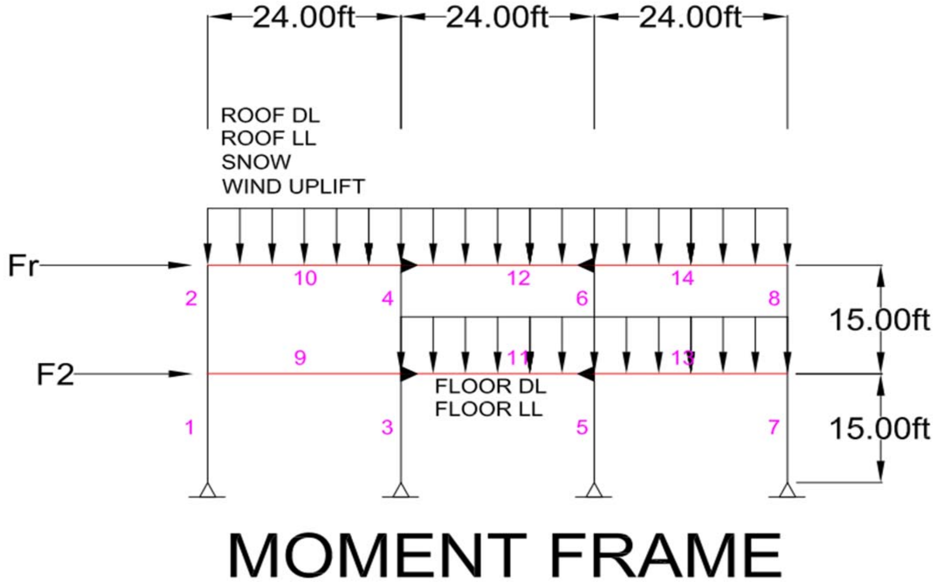
Shear load larger of fytop or fxbott

fytop	0.389998	
fxbott	0.48	
shear load	0.48	k/in

Plate stress= shearload*2welds*1in long / tpl*1in long

Plate stress=	2.540939	ksi	< ϕ.6Fy?=	19.44	OK

Moment Frame Design



Sizing (From 30%)

Members 10 & 14	W18x40
Member 12	W12x19
Members 9 & 13	W24x68
Member 11	W18x35
Exterior Columns	W8x40
Interior Columns	W8x40

Braced Frame Bay Width, W_b	36	ft	Plan Length, L	108	ft	Plan Area	7776 ft^2
Moment Frame Bay Width, W_m	24	ft	Plan Width, W	72	ft		

Vertical Loading

Roof

Roof Live Load, W_{RLL}	0.9	klf
Snow, W_s	0.57	klf
Wind Uplift, W_{WU}	-0.42	klf
Roof Dead Load=self weight of beam		

Level 2

Floor Dead Load, W_{DL}	1.67	klf
Floor Live Load, W_{FlL}	1.8	klf
Girder Reactions on Interior Columns	0	kips
Girder Reactions on Exterior Columns	0	kips

Lateral Loading

Seismic Force, Roof Level	3.23	kips
Seismic Force, Level 2	12.62	kips
Wind Force, Roof Level	9.61	kips
Wind Force, Level 2	19.22	kips

Determine Notional Loads

Roof

Roof Live Load	0.9	klf
Snow Load	0.57	klf
Wind Uplift	-0.42	klf
Dead Load (Beam Self Weight)	0.04	klf
$W_{u\text{-}roof}$	1.49	klf

Level 2

Floor System Load	1.67	klf
Beam Self weight	0.068	klf
Dead Load	1.74	klf
Live Load	1.80	klf
$W_{u\text{-}2}$	4.99	klf

Y_R	107.14	kips	
Y_2	358.99	kips	
α	1		
N_R	0.214	kips	Eq C2-2
N_2	0.718	kips	Eq C2-1
F_y	50	ksi	

Run Computer model in SAP2000

Design of Columns (Data from Members 3 & 5)

First order analysis forces

	top		bottom						
Ultimate Axial Load, P_{unt}	57.15	kips	57.15	kips	SAP	Δ_h/L	0.003		
Ultimate Axial Load, P_{ult}	29.35	kips	29.35	kips	SAP	H	30.215	kips	SAP
Ultimate Moment, M_{untx}	31.08	kip*ft	0	kip*ft	SAP				
Ultimate Moment, M_{ultx}	158.84	kip*ft	0	kip*ft	SAP				
Unbraced Length, L_{bx}	15	ft							
			Effective Length Factor, K_y			1			

P_{story} & P_{mf}

Roof Level Uniform Pressure	0.08	ksf
Level 2 Uniform Pressure	0.27	ksf
$P_{story\text{-}r}$	622.08	kips
$P_{story\text{-}2}$	2111.96	kips
P_{story}	2734.04	kips
Moment Frame Column Tributary Area	432.00	ft^2
P_{mf}	607.56	kips

TRIAL SECTION W10x68

Section Properties (Table 1-1)

Plastic Section Modulus strong axis, Z_x	85.3	in^3	S_x	26.4	in^3
Moment of Inertia, I_x	394	in^4	r_{ts}	2.59	in
r_x	4.44	in	h_o	9.63	in
r_y	2.59	in	J	3.56	in^4
A_g	19.9	in2			
b_f	10.1	in			
t_f	0.77	in			
h	7.5	in			
t_w	0.47	in			

Beam Properties

Full plastic yielding unbraced length, L_p	9.15	ft	Table 3-2
Elastic/Inelastic LTB unbraced length, L_r	40.6	ft	Table 3-2
ϕBF	3.85	kips	Table 3-2
ϕM_{px}	320	kip*ft	Table 3-2

	Flexure		Compression
Slenderness Characteristics	λ_p	λ_r	λ_r
Flanges	9.15	24.08	13.49
	NONCOMPACT		NONSLENDER
Web	90.55	137.27	35.88
	COMPACT		NONSLENDER

Axial Capacity

K	1		K=1 for columns in DAM	
Kl	15	ft		
KL/r	69.50		4.71*sqrt(E/Fy)	113.4318
F_e	59.26	ksi		
F_{cr}	35.12	ksi	E3-2	
F_{cr}	51.97	ksi	E3-3	
F_{cr}	35.12	ksi		
ϕ	0.90			
P_c	629.06	kips	Table 4-1	

Flexure Capacity

Zone?	ZONE 2			
M_{max}	158.84	kip*ft	M_{ult}	
M_A	23.54	kip*ft	SAP	
M_B	54.31	kip*ft	SAP	
M_C	81.47	kip*ft	SAP	
C_b	2.14			Eq F1-1
Zone 1, M_c	320	kip*ft		Eq F2-1
Zone 2, M_c	320.00	kip*ft		Eq F2-2
Zone 3, F_{cr}	317.14	ksi		Eq F2-4
Zone 3, M_c	697.72	kip*ft		Eq F2-3
M_c	320.00	kip*ft		

Determine τ_b

P_r	86.50	kips	$P_{nt}+P_{lt}$
P_y	995.00	kips	A_g*F_y
α	1		
$\alpha P_r/P_y$	0.09		
τ_b	1.00		Eq C2-2a

Determine B1

P_{e1}	2784.45	kips	*Eq A-8-5*
Modification Coefficient, C_{mx}	0.60		*Eq A-8-4*
B_1	1.00		*Eq A-8-3*

Determine B2

R_M	0.97		*Eq A-8-8*
P_{estory}	8762.35	kips	*Eq A-8-7*
B_2	1.45		*Eq A-8-6*
M_r	261.96	kip*ft	*Eq A-8-1*
M_c	320.00	kip*ft	*Referenced Above*
P_r	99.8	kips	*Eq A-8-2*
P_c	629.06	kips	*Referenced Above*

Combined Forces Ineraction Equation

P_r/P_c	0.16		
$P_r/P_c>0.2$	0.89		Eq H1-1a
$P_r/P_c<0.2$	0.90		Eq H1-1b
Design Check	0.90	**SECTION OK**	

Design of Level 2 Beam (Member 11)

First order analysis forces

Ultimate Axial Load, P_{unt}	11.62	kips	SAP	Δ_h/L	0.003		
Ultimate Axial Load, P_{ult}	5.69	kips	SAP	H	30.215	kips	SAP
Ultimate Moment, M_{untx}	74.27	kip*ft	SAP				
Ultimate Moment, M_{ultx}	112.88	kip*ft	SAP				
Unbraced Length, L_{bx}	24	ft					
			Effective Length Factor, K_y		1		

P_{story} & P_{mf}

Roof Level Uniform Pressure	0.08	ksf
Level 2 Uniform Pressure	0.27	ksf
$P_{story-r}$	622.08	kips
$P_{story-2}$	2111.96	kips
P_{story}	2734.04	kips
Moment Frame Column Tributary Area	432.00	ft^2
P_{mf}	607.56	kips

TRIAL SECTION W12x53

Section Properties (Table 1-1)

Plastic Section Modulus strong axis, Z_x	77.9	in^3	S_x	70.6	in^3
Moment of Inertia, I_x	425	in^4	r_{ts}	2.79	in
r_x	5.23	in	h_o	11.5	in
r_y	2.48	in	J	1.58	in^4
A_g	15.6	in2			
b_f	10	in			
t_f	0.575	in			
h	9.25	in			
t_w	0.345	in			

Beam Properties

Full plastic yielding unbraced length, L_p	8.76	ft	Table 3-2
Elastic/Inelastic LTB unbraced length, L_r	28.2	ft	Table 3-2
ϕBF	5.5	kips	Table 3-2
ϕM_{px}	292	kip*ft	Table 3-2

Slenderness Characteristics	Flexure λ_p	Flexure λ_r	Compression λ_r
Flanges	9.15	24.08	13.49
	NONCOMPACT		SLENDER
Web	90.55	137.27	35.88
	COMPACT		NONSLENDER

Axial Capacity

K	1		K=1 for columns in DAM	
Kl	24	ft		
KL/r	116.13		4.71*sqrt(E/Fy)	113.4318
F_e	21.22	ksi		
F_{cr}	18.65	ksi	E3-2	
F_{cr}	18.61	ksi	E3-3	
F_{cr}	18.61	ksi		
ϕ	0.90			
P_c	261.33	kips		

Flexure Capacity

Zone?	ZONE 2			
C_b	1.14		Table 3-1	
Zone 1, M_c	292	kip*ft		Eq F2-1
Zone 2, M_c	249.06	kip*ft		Eq F2-2
Zone 3, F_{cr}	49.54	ksi		Eq F2-4
Zone 3, M_c	291.47	kip*ft		Eq F2-3
M_c	249.06	kip*ft		

Determine τ_b

P_r	17.31	kips	$P_{nt}+P_{lt}$
P_y	780.00	kips	A_g*F_y
α	1		
$\alpha P_r/P_y$	0.02		
τ_b	1.00		Eq C2-2a

Determine B1

P_{e1}	1173.25	kips	*Eq A-8-5*
Modification Coefficient, C_{mx}	0.99		*Eq A-8-4*
B_1	1.01		*Eq A-8-3*

Determine B2

R_M	0.97		*Eq A-8-8*
P_{estory}	8762.35	kips	*Eq A-8-7*
B_2	1.45		*Eq A-8-6*
M_r	239.01	kip*ft	*Eq A-8-1*
M_c	249.06	kip*ft	*Referenced Above*
P_r	19.9	kips	*Eq A-8-2*
P_c	261.33	kips	*Referenced Above*

Combined Forces Ineraction Equation

P_r/P_c	0.08		
P_r/P_c>0.2	0.93		Eq H1-1a
P_r/P_c<0.2	1.00		Eq H1-1b
Design Check	0.998	**SECTION OK**	

Design of Roof Level Beam (Member 12)

First order analysis forces

Ultimate Axial Load, P_{unt}	7.29	kips	SAP	Δ_h/L	0.003		
Ultimate Axial Load, P_{ult}	2.35	kips	SAP	H	30.215	kips	SAP
Ultimate Moment, M_{untx}	14.41	kip*ft	SAP				
Ultimate Moment, M_{ultx}	43.08	kip*ft	SAP				
Unbraced Length, L_{bx}	24	ft					
			Effective Length Factor, K_y		1		

P_{story} & P_{mf}

Roof Level Uniform Pressure	0.08	ksf
Level 2 Uniform Pressure	0.27	ksf
$P_{story-r}$	622.08	kips
$P_{story-2}$	2111.96	kips
P_{story}	2734.04	kips
Moment Frame Column Tributary Area	432.00	ft^2
P_{mf}	607.56	kips

TRIAL SECTION W10x39

Section Properties (Table 1-1)

Plastic Section Modulus strong axis, Z_x	46.8	in^3	S_x	42.1	in^3
Moment of Inertia, I_x	209	in^4	r_{ts}	2.24	in
r_x	4.27	in	h_o	9.39	in
r_y	1.98	in	J	0.976	in^4
A_g	11.5	in2			
b_f	7.99	in			
t_f	0.53	in			
h	7.5	in			
t_w	0.315	in			

Beam Properties

Full plastic yielding unbraced length, L_p	6.99	ft	Table 3-2
Elastic/Inelastic LTB unbraced length, L_r	24.2	ft	Table 3-2
ϕBF	3.78	kips	Table 3-2
ϕM_{px}	176	kip*ft	Table 3-2

	Flexure		Compression
Slenderness Characteristics	λ_p	λ_r	λ_r
Flanges	9.15	24.08	13.49
	NONCOMPACT		SLENDER
Web	90.55	137.27	35.88
	COMPACT		NONSLENDER

Axial Capacity

K	1		K=1 for columns in DAM	
Kl	24	ft		
KL/r	145.45		4.71*sqrt(E/Fy)	113.4318
F_e	13.53	ksi		
F_{cr}	10.64	ksi	E3-2	
F_{cr}	11.86	ksi	E3-3	
F_{cr}	11.86	ksi		
ϕ	0.90			
P_c	122.80	kips		

Flexure Capacity

Zone?	ZONE 2		
C_b	1.14		Table 3-1
Zone 1, M_c	176	kip*ft	Eq F2-1
Zone 2, M_c	136.34	kip*ft	Eq F2-2
Zone 3, F_{cr}	40.37	ksi	Eq F2-4
Zone 3, M_c	141.64	kip*ft	Eq F2-3
M_c	136.34	kip*ft	

Determine τ_b

P_r	9.64	kips	P_{nt}+P_{lt}
P_y	575.00	kips	A_g*F_y
α	1		
$\alpha P_r/P_y$	0.02		
τ_b	1.00		Eq C2-2a

Determine B1

P_{e1}	576.96	kips	*Eq A-8-5*
Modification Coefficient, C_{mx}	0.99		*Eq A-8-4*
B_1	1.01		*Eq A-8-3*

Determine B2

R_M	0.97		*Eq A-8-8*
P_{estory}	8762.35	kips	*Eq A-8-7*
B_2	1.45		*Eq A-8-6*
M_r	77.18	kip*ft	*Eq A-8-1*
M_c	136.34	kip*ft	*Referenced Above*
P_r	10.7	kips	*Eq A-8-2*
P_c	122.80	kips	*Referenced Above*

Combined Forces Ineraction Equation

P_r/P_c	0.09		
P_r/P_c>0.2	0.59		Eq H1-1a
P_r/P_c<0.2	0.61		Eq H1-1b
Design Check	0.61	**SECTION OK**	

Moment Frame Connection Design-Level 2

Given:

Girder	W12x53		**Column**	W10x68	
depth, d	12.1	in	depth, d	10.4	in
t_f	0.575	in	t_f	0.77	in
b_f	10	in	b_f	10.1	in
t_w	0.345	in	t_w	0.47	in
			k	1.27	in

Flange Plates

t	3/4	in	F_y	36	ksi	F_u	58	ksi
w	10	in	<0.15*bf (column)?	CHECK FLANGE LOCAL BENDING			J10.1	
unbraced length, L_p	3	in						
# of rows	2		Gage	7	in	Spacing	3	in
A325N, bolt dia.	7/8	in	Hole	std		# bolts	10	per flange
						Conn. Length	12	in
						Edge Distance	2	in

Flange-Column Weld

Electrode Strength, F_u	70	ksi
Flange CJP Weld:		
Length	10	in
Size	1/2	in

Double Angle Connection

t	1/4	in	weld size	1/4	in	
# bolts/row, n	4		Weld Lengt	11.5	in	(L)
A325N, bolt dia.	3/4	in	Hole	std		

Beam End Forces (LRFD)

Shear, V	78.22	kips
Moment, M	239.01	kip*ft

Determine:

Connection adequacy and need for column stiffeners

Solution

Determine Flange and Web Shear Forces

Flange Force, T=C	237.03	kips	M/d
Web Shear Force	78.22	kips	Given

Tensile Strength of Flange Plate

a. Tensile Yield, P_n

Gross Area, A_g	7 1/2	in^2	
Nom. Tensile Yield, P_n	270	kips	Eq D2-1
Φ	0.9		
ΦP_n	243	kips	
	OK		

b. Tensile Rupture, P_n

Effective Area, A_e	6.00	in^2
Nom. Tensile Rupt., P_n	348.00	kips
Φ	0.75	
ΦP_n	261.00	kips
	OK	

Flange Plate Block Shear

A_{nt}	3.00	in^2
A_{nv}	28.50	in^2
A_{gv}	42.00	in^2
x=y for WT6x26.5	1.02	in
U	1.00	

$0.6*F_u*A_{nv}+U*F_u*A_{nt}$	$0.6*F_y*A_{gv}+U*F_u*A_{nt}$	Eq J4-5
1165.8 kips	1081 kips	

R_n	1081.20	kips
Φ	0.75	
ΦR_n	810.90	kips
	OK	

Flange Plate Bolt Bearing

Find Minimum clear distance, L_c

Interior L_c	2.00	in
Edge L_c	1.50	in
Min L_c	1.50	in

Tearout		Deformation			
78.30	kips/bolt	91.35	kips/bolt	Eq J3-6A	

Nominal Strength, R_n	78.30	kips/bolt
Nominal Plate Bearing Strength, P_n	783	kips
Φ	0.75	
ΦP_n	587.25	kips
	OK	

Bolt Shear

Bolt Area, A_b	0.60	in^2	
A325N, F_{nv}	54	ksi	Table J3.2
R_n	32.47	kips/bolt	
Bolt Shear Strength	324.71	kips	
Φ	0.75		
ΦP_n	243.53		
	OK		

Flange Plate Compressive Strength

K	0.65		Table C-A-7.1
r_y	0.22	in	t/sqrt(12)
KL_p/r	9.01		Eq J-4-6 Applies
Effective Plate width, l_w	10	in	
Effective gross area, A_{ge}	7.5	in^2	
P_n	270	kips	Nominal Compressive Strength, Eq J4-6
F_{cr}	32.3	ksi	Table 4-22
Φ	0.9		
ΦP_n	243		
	OK		

Evaluate Double-angle connection

Strength of welds	79.9	kips		Table 10-2 (Case II)
	OK			
Bolt/Angle Strength	101	kips		Table 10-1
	OK			
Bolt bearing on web	351	k/in	121.10 kips	Table 10-1
	OK			

Check Column with concentrated forces

Flange local bending

R_n	133.40	kips	J10-1
Φ	0.9		
ΦR_n	120.06	kips	
	OK		

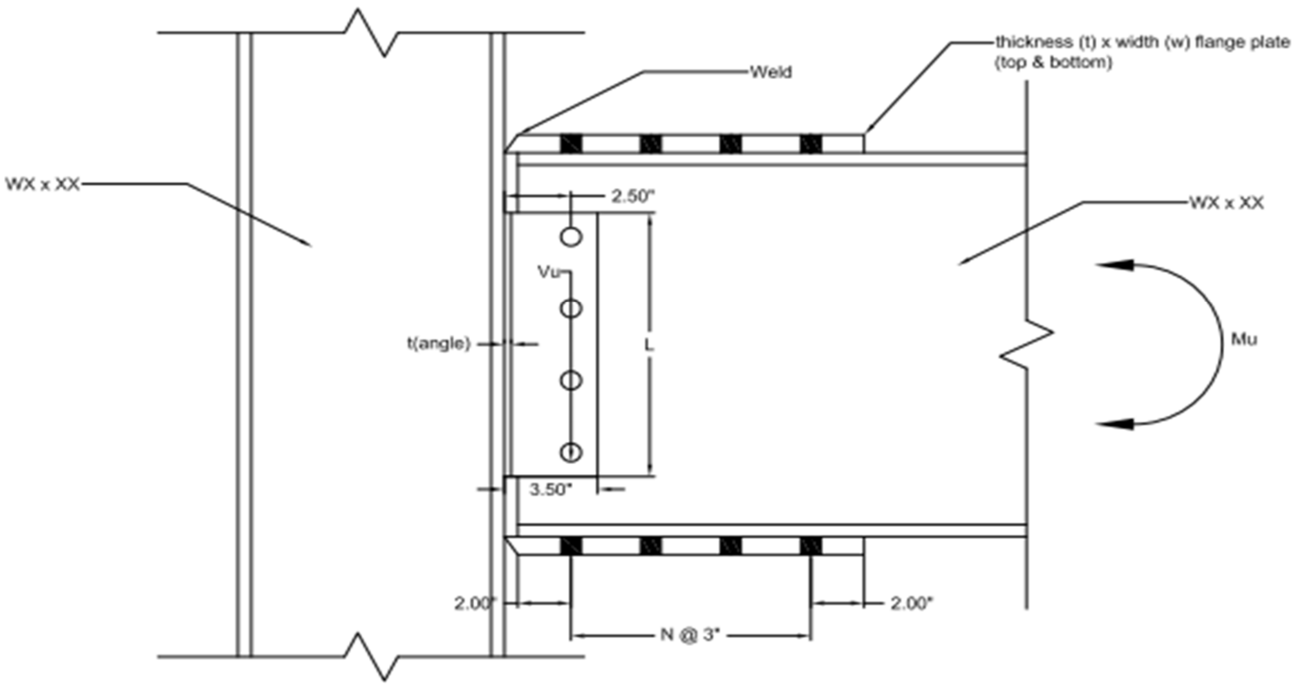
Web Local Yielding

R_n	104.48	kips	J10-3
Φ	1		
ΦR_n	104.48	kips	
	OK		

Web Crippling Parameters

R_n	326.50		J10-4
Φ	0.75		
ΦR_n	244.87		
	OK		

Summary


thickness (t) x width (w) flange plate
(top & bottom)
Weld
WX x XX
2.50"
WX x XX
Vu
t(angle)
L
Mu
3.50"
2.00"
2.00"
N @ 3"

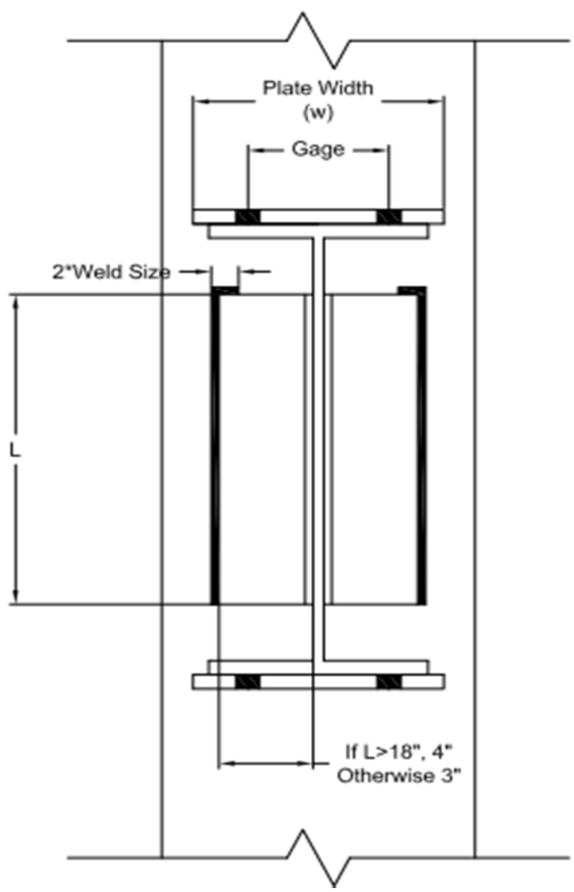


Plate Width
(w)
Gage
2*Weld Size
L
If L>18", 4"
Otherwise 3"

Girder:	W12x53	M_u	239.01 kip*ft
Column:	W10x68	V_u	78.22 kips

Flange Plate:

Thickness, t	3/4	in	F_y	36	ksi	F_u	58 ksi
width, w	10	in					
# of rows	2						
Hole	std						
A325N, bolt dia.	7/8	in					
# bolts	10	per flange					
Gage	7	in					

Flange-Column Weld:

Electrode Strength, F_u 70 ksi

Flange CJP Weld:

Length	10	in
Size	1/2	in

Double Angle Connection:

Weld to column, bolt to girder

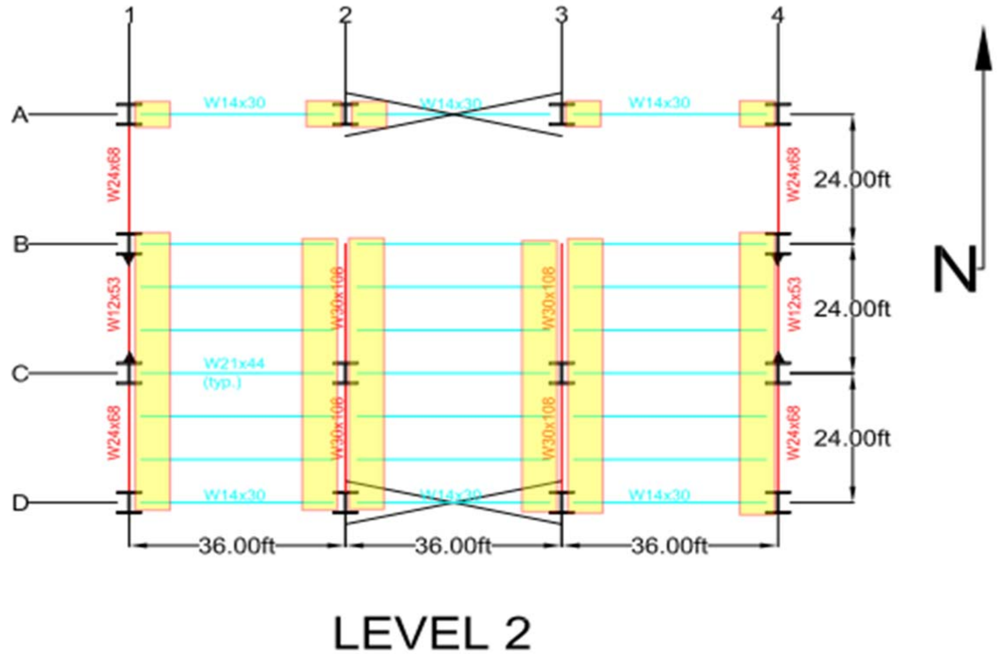
Angle properties:

t	1/4	in
L	11.5	in
Leg Connect. to Column	3	in
Leg connect. To girder	3.5	in
weld size	1/4	in
Hole	std	
# bolts/row, n	4	(Bolts spaced at 3" O.C.)
A325N, bolt dia.	3/4	in

Simple Shear Connection Design - Level 2

BEAM CONNECTIONS

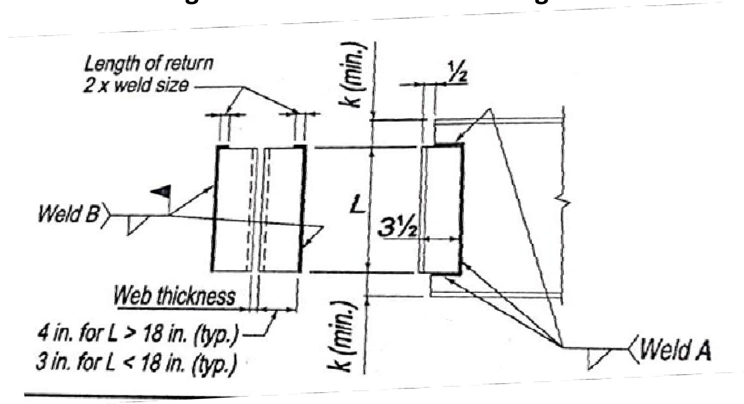
Connections highlighted below:



P_u	39.11	kips	Shear reaction

Use all welded double angle connections

Table 10-3: Available Weld Strength of All-Welded Double Angle Connections



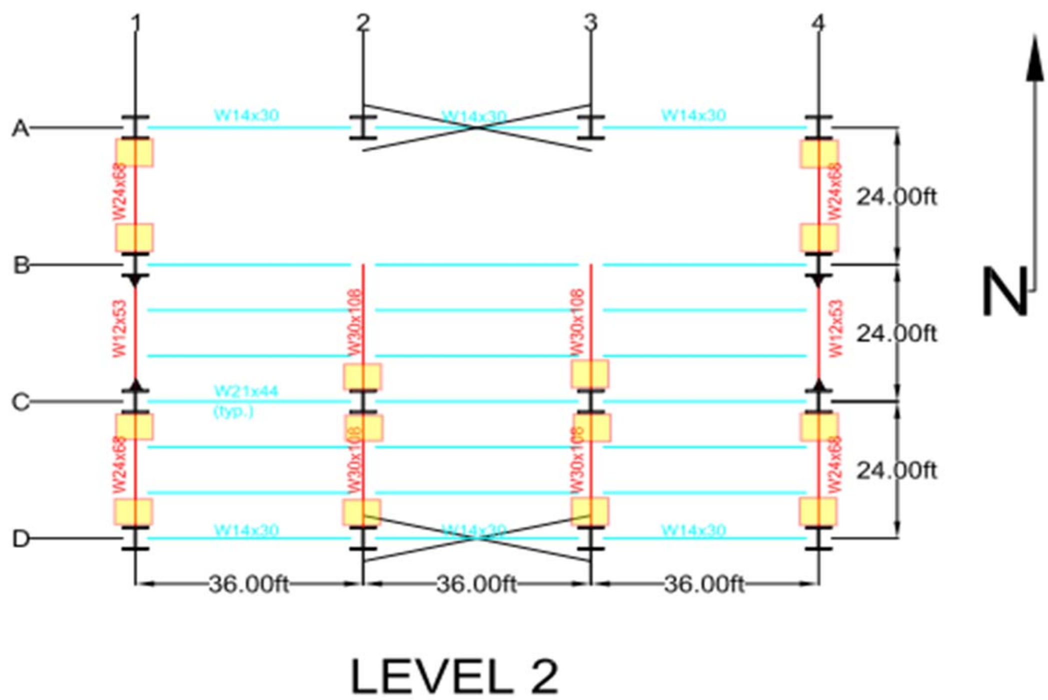
Welds with the following specifications meet requirements:

L	6	in	
Weld B	5/16	in	
Angle Width on Supporting element	3	in	
Weld A	3/16	in	
Angle Thickness	0.25	in	Weld size+(1/16)

Use (2) 3.5x3x1/4 angles

GIRDER CONNECTIONS

Connections highlighted below:

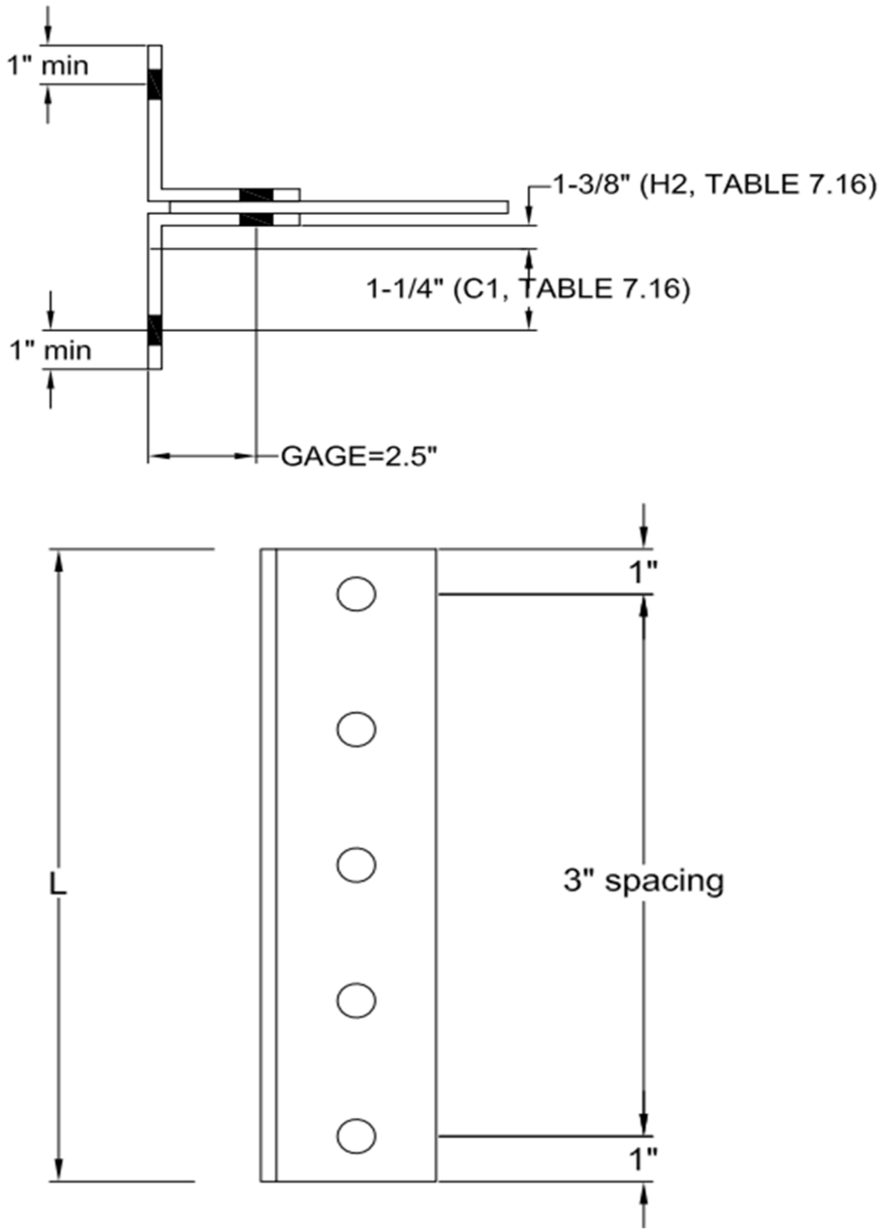


P_u 156.44 kips

Use all bolted double angle connection
Table 10-1: All Bolted Double-Angle Connections

Following bolt specifications meet requirements:
5 rows, 3/4" bolts
Group A, threads included, STD hole
3/8" angle thickness

Size angle:



Number of spaces, N	4		
Girder t_w	0.545	in	
Angle thickness, t	3/8	in	
H_2	1 3/8	in	Table 7.16
C_1	1 1/4	in	Table 7.16
Gage	2.5	in	Assumed
Min. edge distance	1	in	Assumed

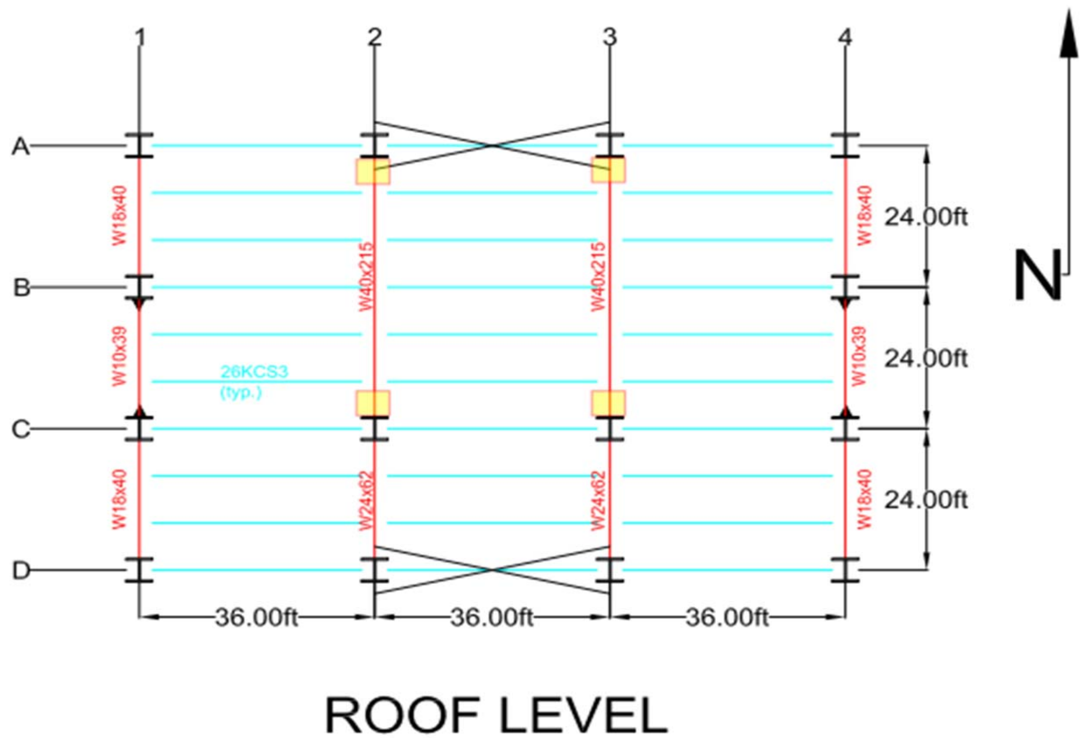
Leg Length bolted to girder:	3.5	in	(Gage+1")
Use	3.5	in	
Leg length bolted to column:	4.00	in	($t+H_2+C_1+1"$)
Use	4	in	
Angle Length	14		

Use (2) 4 x 3.5 x 3/8 x 1'-2" angles

Simple Shear Connection Design - Roof Level

GIRDER CONNECTIONS

Connections highlighted below:

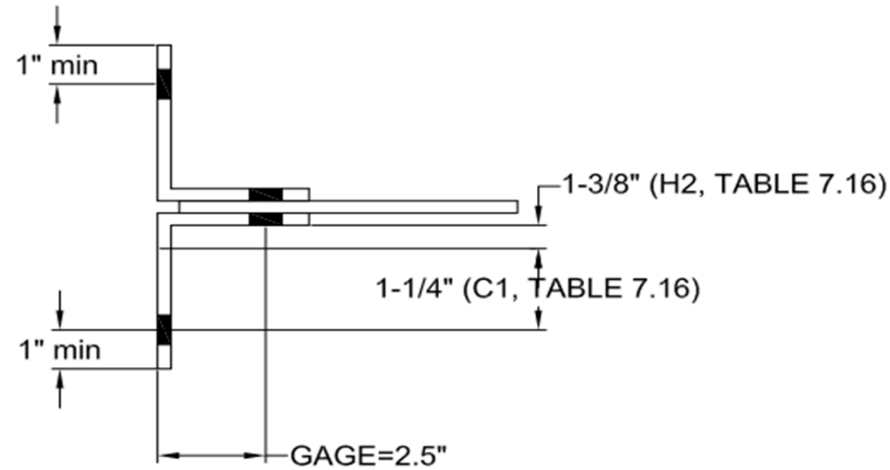


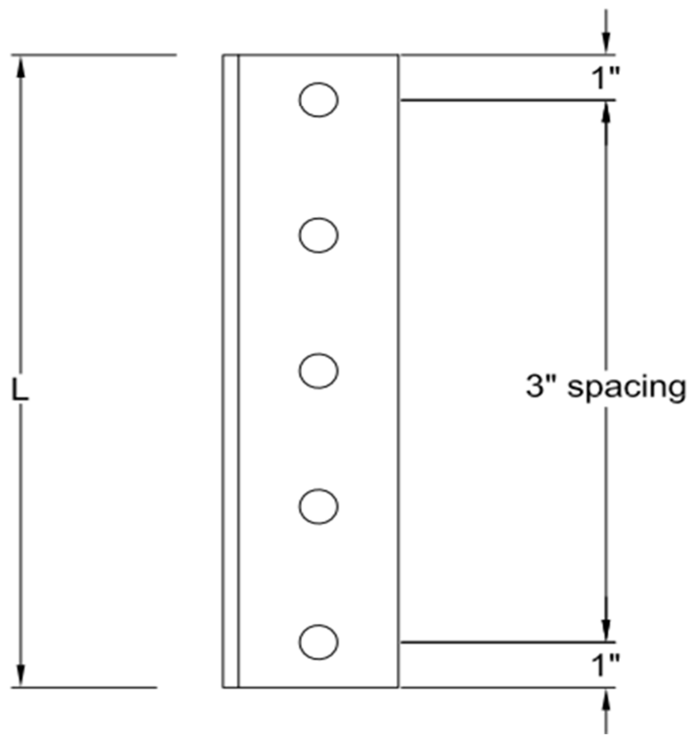
P_u 295.97 kips

Use all bolted double angle connection
Table 10-1: All Bolted Double-Angle Connections

Following bolt specifications meet requirements:
9 rows, 3/4" bolts
Group A, threads included, STD hole
3/8" angle thickness

Size angle:



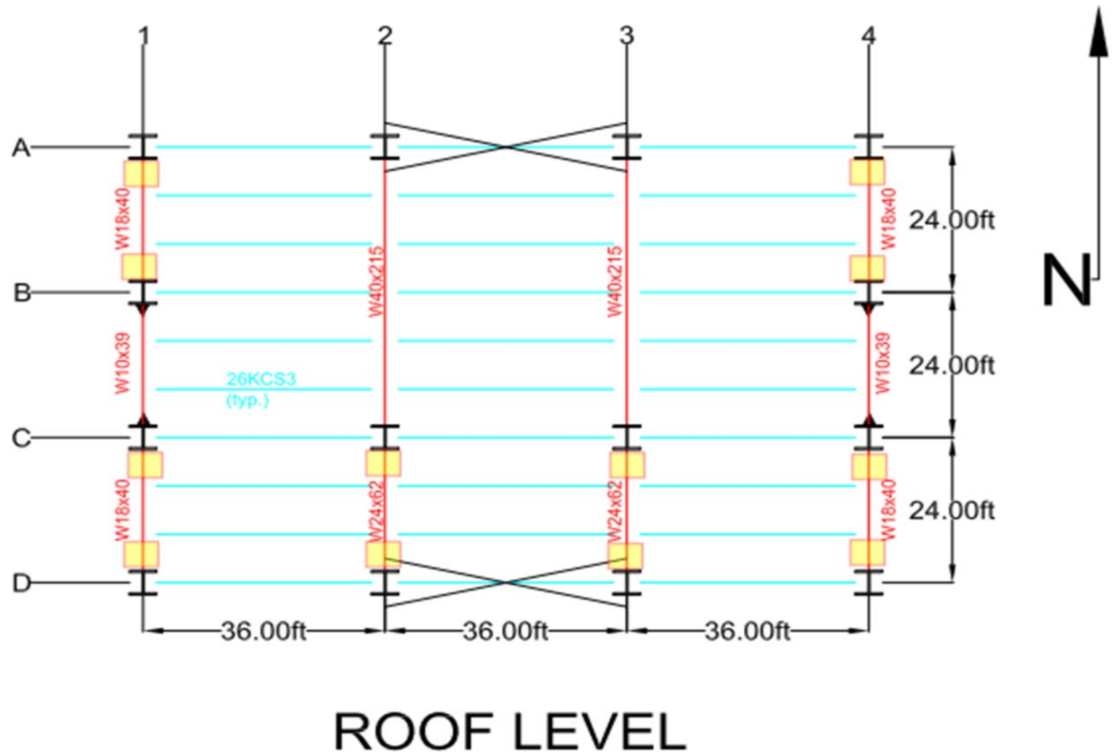


Number of spaces, N	8		
Girder t_w	0.65	in	
Angle thickness, t	3/8	in	
H_2	1 3/8	in	Table 7.16
C_1	1 1/4	in	Table 7.16
Gage	2.5	in	Assumed
Min. edge distance	1	in	Assumed

Leg Length bolted to girder:	3.5	in	(Gage+1")
Use	3.5	in	
Leg length bolted to column:	4.00	in	($t+H_2+C_1+1$")
Use	4	in	
Angle Length	26		

Use (2) 4 x 3.5 x 3/8 x 2'-2" angles

Connections highlighted below:



P_u 71.46 kips

Use all bolted double angle connection

Table 10-1: All Bolted Double-Angle Connections

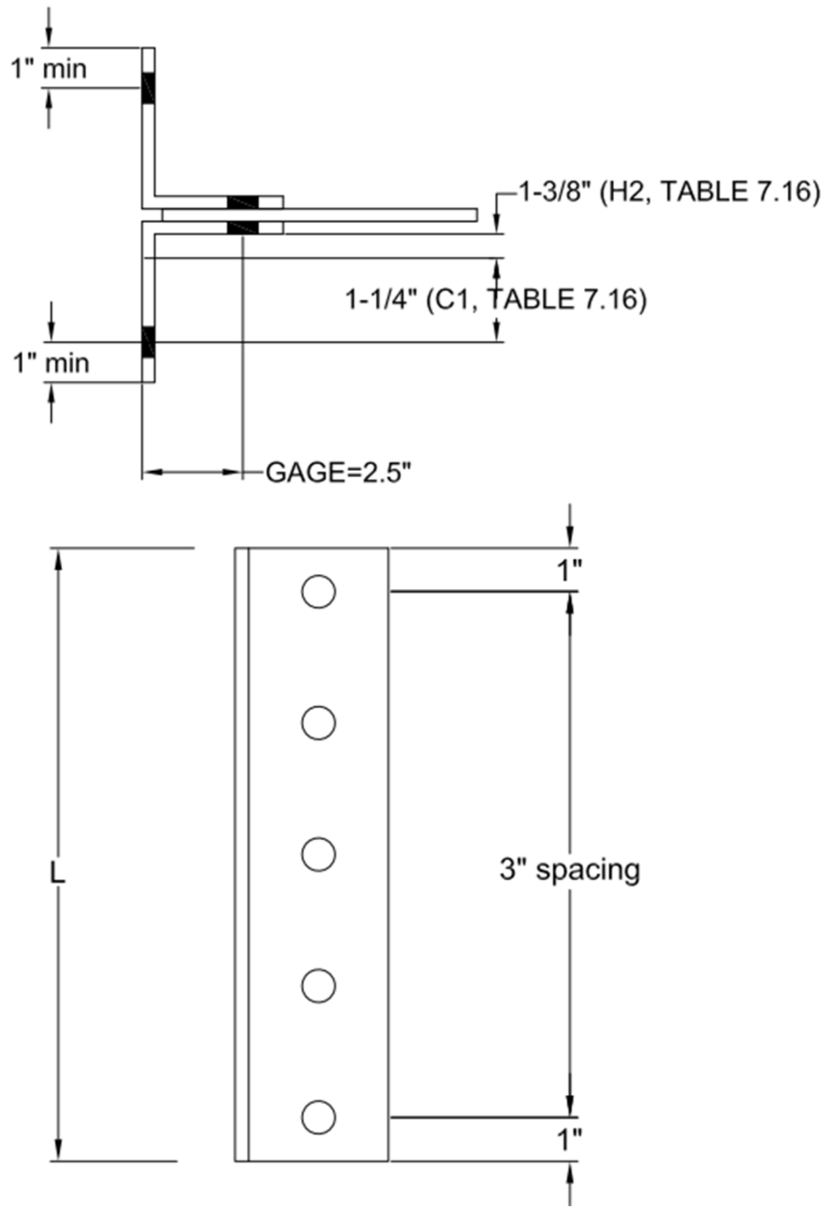
Following bolt specifications meet requirements:

4 rows, 3/4" bolts

Group A, threads included, STD hole

1/4" angle thickness

Size angle:



Number of spaces, N	3		
Girder t_w	0.43	in	
Angle thickness, t	1/4	in	
H_2	1 3/8	in	Table 7.16
C_1	1 1/4	in	Table 7.16
Gage	2.5	in	Assumed
Min. edge distance	1	in	Assumed

Leg Length bolted to girder:	3.5	in	(Gage+1")
Use	3.5	in	
Leg length bolted to column:	3.88	in	($t+H_2+C_1+1"$)
Use	4	in	
Angle Length	11		

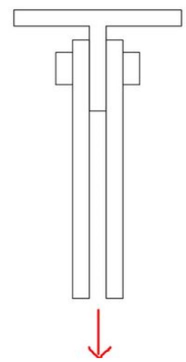
Use (2) 4 x 3.5 x 1/4 x 11" angles

Prepared by:	Matthew Laskey	Date:	11/29/2014
Checked by:	Ana Gouveia		

Hanger Tension MemberDesign
Loading Referenced from "Hanger Connection Design"

The tension member will consist of two narrow steel plates bolted together at the WT member. The only forces acting on this member is tension.

Pu=	117.33	k
Pu=	58.665	k/tension member



Steel Plates			WT12x96 Member		
tpl=	1	in	d=	12.7	in
wpl=	14	in	k=	1.96	in
Fy=	36	ksi	d-k=	10.74	in
Fu=	58	ksi			

Plates

Allowable tensile yielding (AISC 16.D.2)

ϕ=	0.9	
Ag=	14	in2
Pn=	504	k
ϕPn=	453.6	k/member

Allowable Tensile rupture

ϕ=	0.75	
An=	11.375	in2
U=	1	Table D3.1
Ae=	11.375	in2
Pn=	659.75	k
ϕPn=	494.8125	k/member

Block Shear of Plates

ϕ=	0.75	
Ubs=	1	Uniform tension stress

Rn=.6FuAnv+UbsFuAnt<.6FyAgv+UbsFuAnt

.6FuAnv+UbsFuAnt=	659.75	k
.6FyAgv+UbsFuAnt=	659.75	k

ϕRn=	395.85	k/member

Check

yield	453.6	k/member	>	58.665	OK
Rupture	494.8125	k/member	>	58.665	OK
Block	395.85	k/member	>	58.665	OK

Bolts

Using A325N 3/4" diameter bolts

n=	3		
dia=	0.75	in	
Ab=	0.44	in2	
Fnt=	90	ksi	Table J3.2
Fnv=	54	ksi	Table J3.3

Tensile Strength of Bolts AISC J3.6

ϕ=	0.75	
Rnt=	39.76	k
ϕRnt=	29.82	k/bolt

Shear Strength of Bolts (double Shear) AISC J3.6

ϕ=	0.75	
Rnv=	23.86	k
ϕRnv=	17.89	k/bolt

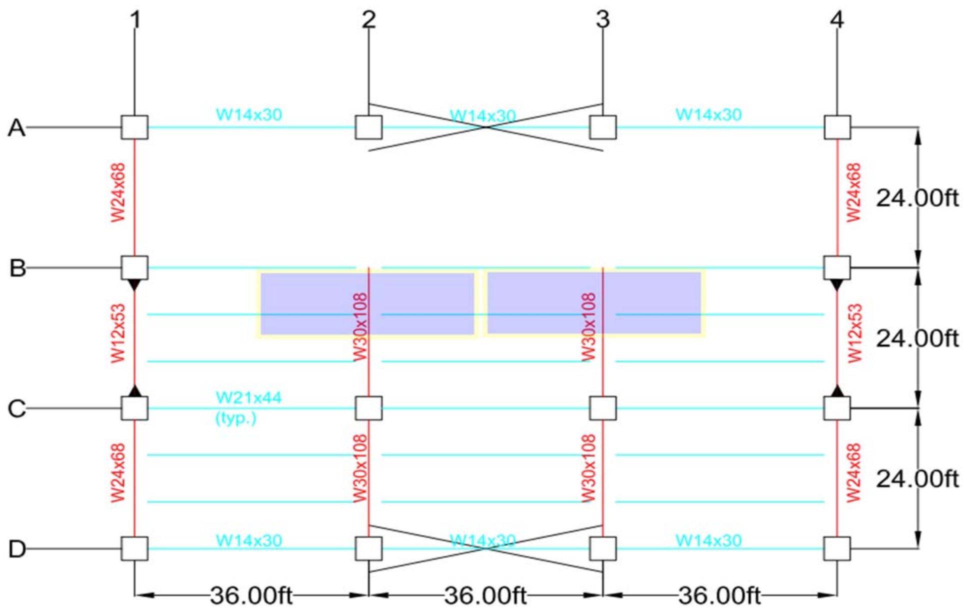
Check shear force per bolt

rnv=Pu/2n	9.78	<	17.89	OK

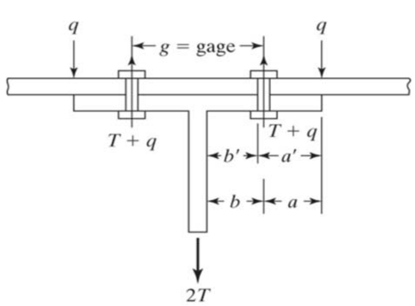
Check Tension force per bolt

rnt=Pu/n	19.56	<	29.82	OK

Hanger Connection Design



Dead Load, DL	93	psf
Live Load, LL	100	psf
Tributary width	36	ft
Tributary Length	12	ft
Tributary Area	432	ft^2
W_u	271.6	psf
P_u	117.33	kips/hanger



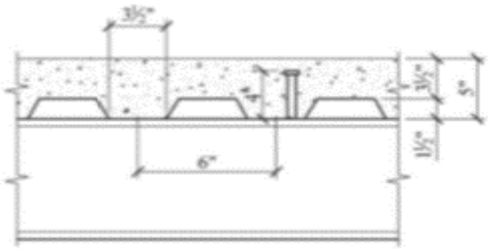
Specify WT: ***WT12x96***

Hanger will bear on a W30 girder

ϕ	0.9		
Ω	1.67		
F_u	58	ksi	Table 2-4
b_f	13	in	Table 1-8
t_f	1.46	in	Table 1-8
t_w	0.81	in	Table 1-8
bolt diameter, d_b	3/4	in	
hole diameter, d_h	7/8	in	+1/8
Edge distance	2	in	
Gage	8.13	in	b_f-(2*Edge distance)-(2*d_h/2)
a	2.44	in	
a'	2.00	in	
b	3.66	in	
b'	3.22	in	
S	8.13	in	
P	8.13	in	
T	58.67	kips	
t_{min}	1.33	in	
t_f>t_{min}?	YES		

1. Given:

Unshored Construction:
Steel beams must support all loads prior to .75f'c



Floor Slab/Deck Details

Steel Beam Properties:

Section:	W	**W21X44**		E	29000	ksi
	d	**20.7**	in	F_y	50	ksi
	b_f	**6.5**	in	r_{ts}	**2.81**	in
	t_f	**0.45**	in	h_0	**11.6**	in
	t_w	**0.35**	in	I_{xw}	**843**	in^4
	A	**13**	in^2	I_{yw}	**20.7**	in^4
	S_x	**81.6**	in^3	J_w	**0.77**	in^4
	Z	**95.4**	in^3	r_{xw}	**8.06**	in
	Zy	**10.2**	in	ryw	**1.26**	in
	S_y	**6.37**	in^3			

Beam/Slab Properties:

Beam Yield Stress, Fy	50	ksi					
Beam Spacing	9	ft					
Beam Span Length	36	ft	Bay Width	24	ft		
Deck height, hr	1.5	in	rib spacing	6	in	OK	
Overall Slab thickness, Ycon	5	in	rib width	3.5	in		
Deck orientation	Perpendicular						
Thickness of Concrete, tc	4.25	in	tc = Yc - hr/2				*I3.2c*
effective slab width	108	in	beff = S1/2 + S2/2				*13.1a*

Concrete Properties:

Compressive Strength, f'c	4	ksi					
Unit Weight of Concrete	145	pcf					
Elastic Modulus, Ec	3492	ksi					*Eq. I8-1*

Load Transfer Device

Max Headed stud diameter	3/4	in	Stud Used	3/4	in	OK	*I8.2d*
Min Length	3	in	length	4	in	OK	
ASTM A108, Tensile Strength, Fu	65.00	ksi	Stud position	weak			
Minimum longitudinal spacing	4.5	in	Rp	0.6			*I8.2d*
Maximum longitudinal spacing	36	in	Rg	1			*I8.2d*
Minimum transverse spacing	3	in	clearance	1.5	in		*I8.2d*

Loads

Superimposed Floor Live Load	150	psf	LRFD, ϕ	0.9	
Superimposed Dead Load	171.5	psf			
Construction Live Load (staging)	20.00	psf			
Metal Deck	2	psf			
Beam Self Wt	44	psf	from beam used above		
Ceilling and utilities	10	psf			
Partitions	20	psf			8100

Method/Solution

1. Design Loads on Beam — **Service Loads (plf)**

1,1 Construction Phase	D	L	TOTAL	
				Assumptions:
Live Load		180		Value given * beam spacing (twidth)
Dead Loads:				
Metal Deck	18			
Beam Self Weight	44			
Wet Concrete	462			Calculated value based on beam spacing
Total with concrete	524	180	704	Concrete Thickness
Total without concrete	62	180	242	

Total Uniform Construction Design Load, wc	**Factored Loads (plf)**	**TOTAL**

LRFD, with concrete, wc-c	629	288	917
LRFD, without concrete, wc-wc	74	288	362

1,2 Superimpose Loads (post Construction)	D	L	TOTAL
Superimpose Floor Live Load		1543.5	
Superimpose Dead Loads:	288		
Total with concrete	288	1543.5	1832

Total Uniform Construction Design Load, wc	**Factored Loads (plf)**		
LRFD, with concrete, wc-c	346	2470	2815

2, Determine Required Beam Strength

2.1. Construction Phase

Live Moment, Mrcll	46.656	k-ft	total live load
Total Moment, Mrc	148.6	k-ft	with concrete
Total Shear, Vrc	16.5	kips	with concrete
Total Moment, Mr+rc	58.7	k-ft	Prior to fastening deck (no conc)

2,2 Superimposed (post-construction)

Moment, Mrs	456.06	k-ft
Shear, Vrs	50.67	kips

3. Select Feasible shapes assuming 100% composite action and superimposed loads

3,1 PNA is located in slab, estimate a= ,4 x tc	1.7	in
3,2 Calculate Y2 = Ycon - a/2	4.15	in
3,3 Select Feasible shapes		

Required Superimposed Moment	456.06	k-ft

4,0, Check Construction Phase Required Strength/Capacity **Cb** 1

4.1 Construction w/ DL + LL (before bracing top flange with unfastened deck)

Check Elastic Lateral Torsional Capacity and temporary bracing requirements

Required Moment

Mr-nc	58.7	k-ft

Feasible Shapes	Lb	Sx	rts (in)	ho	J	Lb/rts	Eq. F2-4 Fcr (ksi)	Mn (k-ft)	$\Phi_b M_n$ (k-ft)	Check Mr-nc/ΦbMn	Is Temp. Bracing Req'd?
W18X35	36	57.6	1.42	15.5	0.506	304.2	6.98	33.5	30.1	1.95	Yes
W18X35	30	57.6	1.51	17.3	0.506	238.4	9.08	43.6	39.2	1.50	No

4.2 Construction w/ DL + LL (with fastened deck) + Assume Fully Braced Top Flange

Required Moment

Mrc	148.6	k-ft

Construction Dead Load Deflection

Superimposed Load, wu	0.704	kips
Md=	114.08	k-ft

Feasible Shapes	$\Phi_b M_{px}$ (k-ft)	Ix	$\Phi_b M_n$ (k-in)	Mrc/ΦbMn	$\Phi_b V_n$ (k)	Vrc/ΦbVn	Check Δcdl (in)	Camber? (D>,75 in)	
W18X35	125	510	361.7	0.41	94.5	0.17	1.80	YES	Table 3-2
W18X35	166	510	470.7	0.32	106	0.16	1.80	YES	Table 3-2

5. Check Adequacy of Composite Sections

5.1 Flexure & Shear

Feasible Shapes	As	d	a	Y2	Mn (k-ft)	$\Phi_b M_n$ (k-in)	Mrs/ΦbMn	< 1,0 ?	$\Phi_b V_n$ (k)	Vrs/ΦbVn	< 1,0 ?
W18X35	10.3	17.7	1.40	4.30	515	463.5	0.98	OK	94.5	0.54	OK
W18X35	10.3	17.7	1.40	4.30	515	463.5	0.98	OK	106	0.48	OK

5.2 Deflection/Serviceability C1= 161 (l/360) = 1.2 in (l/240) = 1.8 in

[Fig. 3-2 AISC]

Feasible Shapes	Y2	Y1	Table 3-20 I_{LB}	Camber (in)	D (l/360) = Camber - Δcdl (in)	Δsdl (in)	Δsll (in)	Check Δsll (in) < (l/360)	Δtl (in)	Check Δtl (in) < (l/240)
W18X35	4.30	0	1055	1	-0.20	0.36	1.20	OK	1.500	OK
W18X35	4.30	0	875	0	0.00	0.43	2.30	NG	2.730	NG

6.0 Design Steel Anchor Studs Area of Steel Anchor, Asa **Asa =** 0.44 in^2 ea.

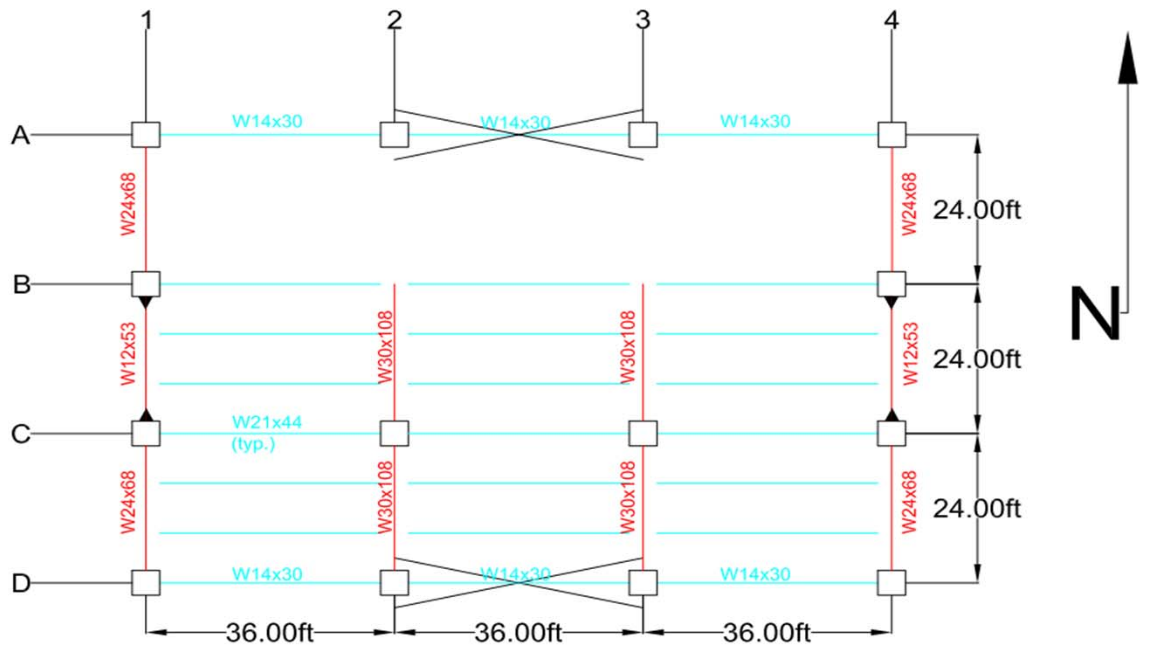
6.1 Strength of Steel Headed Anchors	**Qn1**	26.1		*Eq. I8-1*
	Qn2	17.2		*Eq. I8-1*
	Qn	17.2	Control	

		W18X35	**W18X35**
Total Required Shear Transfer, SQn= Fy.As		515	515
Number Steel Headed Anchors Required, Ns		29.9	29.9
	USE:	30	30
Distance between max and zero moment = L/2		216	216
Required Stud Spacing		7.2	7.2
	CHECK:	OK	**OK**
	TOTAL ANCHORS REQUIRED:	60	**60**

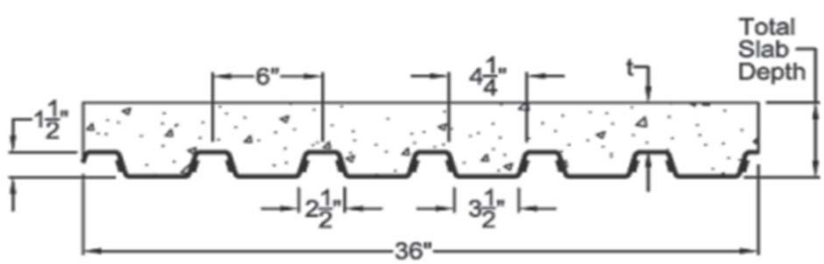
RESULTS	**W18X35**	
CAMBER	1	in
# STUDS	60	studs
STUD Φ	0.75	in
STUD LENGTH	4	in
SPACING	EQUAL	
TEMPORARY BRACING		Yes
Δsll (in)	1.20	in

Floor System Vibration Calculation

Interior Beam/Slab System:



1.5 VLR



Find:

1) Fundamental natural frequency of the interior beam, girder, and combined panels modes of the second floor.

2) Adequacy of the floor system for vibration due to walking excitation.

Beam/Slab Properties

Beam Yield Stress, F_y	50	ksi	
Elastic Modulus, E_s	29000	ksi	
Beam Spacing	8	ft	
Beam Span Length	36	ft	
Deck Height, h_r	1.5	in	
Overall Slab Thickness, Y_{con}	5.5	in	
Deck Orientation on Beam	Perp.		
Concrete Thickness, t_c	4	in	
Rib Spacing	6	in	
Rib Width	3.5	in	

Concrete Properties

Conc. Comp. Strength, f'_c	4	ksi	
Unit weight of conc., w_c	145	pcf	
Elastic Modulus, E_c	3492	ksi	(wc^1.5*sqrt(f'c))
Dynamic Elastic Modulus	4714	ksi	(1.35*Ec)
Modular ratio, n	6.15		(Es/1.35Ec)

Loads

Floor Live Load	100	psf	
Dead Load (No Conc.)	18	psf	
Concrete Dead Load	33.23	psf	$(Y_{con}/2)/12*w_c$
Deck Dead Load	2.5	psf	Vulcraft Manual
W_{total}	153.73	psf	Total uniform load

Dynamic Parameters

Excitation force, P_o	65	lbs	Table 4.1
Damping Ratio, β	0.05	lbs	Table 4.1

Solution:

Determine $I_{transformed}$ of Beams and Girders

	Girder		Beam			
	W30x108		W21x44			
Span, L	24	ft	36	ft		
Tributary Width	36	ft	8	ft		
Self Weight	108	plf	44	plf		
I_x	4470	in^4	843	in^4		
A	31.7	in^2	13	in^2		
d	29.8	in	20.7	in		
effective slab depth, d_e	4.75	in	4	in		
Effective w of conc., b_e	115.2	in	96	in		
Transformed Conc., A_{TC}	88.95	in^2	62.42	in^2	b_e*d_e/n	
dist. To centroid, y'	2.38	in	2	in	$d_e/2$	(from top of slab)
Area of Steel, A_s	31.7	in^2	13	in^2		
dist to centroid, y'	20.4	in	15.85	in	$Y_{con}+d/2$	(from top of slab)
Dist to Trans, N.A., y''	7.11	in	4.39	in		(from top of slab)
I_x of transformed conc., I_{TC}	1028.85	in^4	512	in^4		
$A_{TC}(y'-y'')^2$	1995.04	in^4	355.73	in^4		
$A_S(y'-y'')^2$	5598.30	in^4	1708.15	in^4		
I_x	4470	in^4	843	in^4		
I_x (transformed)	13092.20	in^4	3418.88	in^4		

Determine effective panel width

	Girder		Beam			
Coefficient, C	1.8		2			
D_s	17.42	in^4/ft	10.40	in^4/ft		16
$D_{j/g}$	363.67	in^4/ft	427.36	in^4/ft		
Effective Panel width, B	**44.98**	**ft**	**28.44**	**ft**	4.3a or b	

Determine max deflection, Δ

	Girder		**Beam**		**Combined**	
Uniform load, w	5642.25	plf	1273.83	plf		
L_g/B_j	0.84	<1?	YES			
max deflection, Δ (ft)	0.094	ft	0.410	ft	0.50	ft
max deflection, Δ (in)	1.12	in	4.92	in	6.04	in

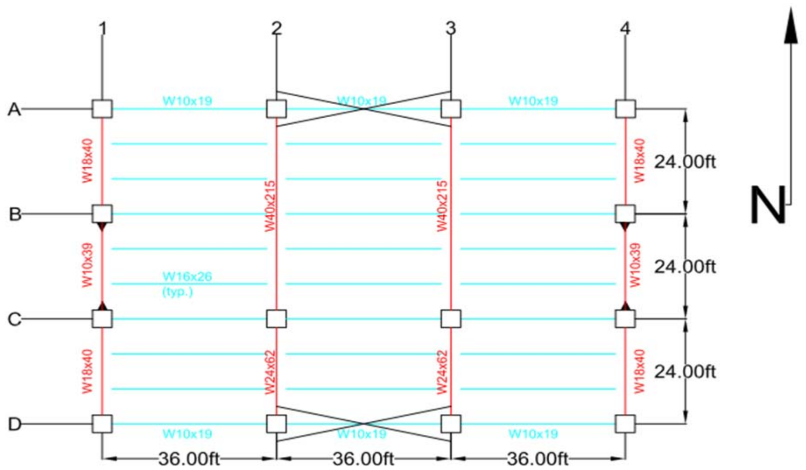
Determine Fundamental Frequency of each panel, f_n

	Girder		**Beam**		**Combined**	
f_n	3.34	Hz	1.59	Hz	4.93	Hz

Determine peak acceleration ratio, a/g

	Girder		**Beam**		**Combined**	
Total uniform floor load	153.73	psf	153.73	psf		
Girder/beam self weight	108	plf	44	plf		
Self Weight uniform load	0.125	psf	0.153	psf		
w	153.85	psf	153.88	psf		
W_j & W_g	166083.02	lbs	157551.89	lbs		
Comb. Equiv panel weight	30888.9841	lbs	128249.5719	lbs	159138.6	lbs
peak acceleration ratio, a/g	0.24%		0.47%		0.15%	
Peak accel<accel limit	0.50%	OK	0.50%	OK	0.50%	OK

Roof Deck Specification (Vulcraft)



Beam Spacing, S	8	ft
Beam Length, L	36	ft

1) Determine suitable metal roof deck to accommodate construction and permenant loads

Construction Loads

Construction Live Load	20	psf	(Assumed)
Metal Deck Self Weight	1.6	psf	(Assumed)
W_a	16.6	psf	ASD LC3

Superimposed Loads

Roof Live Load	30	psf	
Dead Roof	20	psf	
Snow	31.5	psf	
W_a	43.625	psf	ASD LC3

Vulcraft Steel Roof and Floor Deck:
3 Span, B18 meets capacity (83/71 psf)

2) Determine fastening details for roof diaphragm to accommodate the design lateral loads in each direction
By inspection, wind force control over seismic

Wind Force, F_{roof}	6.27	kips	(per frame)
Linear Load, W_{roof}	87.08	plf	
Linear Load (Allowable)	52.25	plf	$0.6*W_{roof}$

Vulcraft B18 table:

36/3 fastener layout
Support Fasteners: 5/8" puddle welds
Side Lap Fasteners (2) #10 tek screws
Shear Strength 278 plf

Shear Strength>Linear Load YES

3) Determine maximum diaphragm deflection in each orthogonal direction

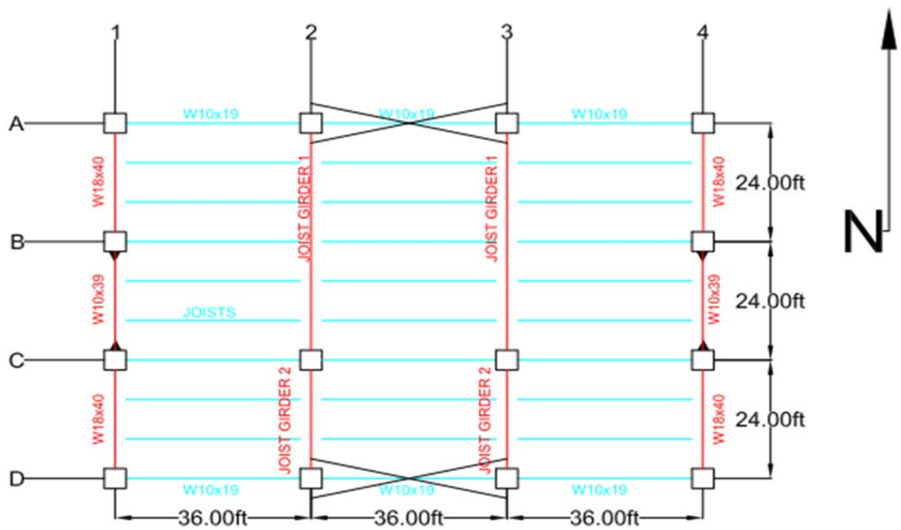
Vertical Diaphragm Deflection

w	43.625	psf
L	8	ft
E	2950000	psi
I	0.289	in^4/ft
Δ	0.001446	in

Lateral Diaphragm Deflection

w	0.09	klf
D_B	1084	
K_1	0.573	
K_2	1398	
Span	8	ft
G'	24.03	k/in
L	108	ft
B	72	ft
Δ_{CL}	0.073	in

Roof Joist Specification (Vulcraft)



Superimposed Loads		
Roof Live Load	30	psf
Dead Roof	20	psf
Snow	31.5	psf

1) Design open web joist and girder system for the roof

Joists			
Joist Spacing, S	8	ft	
Joist Length, L	36	ft	
P_U	74.4	psf	(LRFD LC 3)
W_U	595.20	plf	
W_U	0.0041	pli	
Moment, M_u	96.42	kip*in	
Shear, V_u	10,713.60	lbs	

Use KCS joist:

26KCS3 meets capacity

M_c	1662	kip*in
$>M_u$?	YES	
V_c	12600	lbs
$>V_u$?	YES	

APPENDIX B: DRAWINGS

Appendix B: Drawings

S-1. General Notes
S-2. First Floor Framing Plan
S-3. Roof Framing Plan
S-4. Elevations
S-5. Connection Details

XREFs: [CDMS_2436] Images: []
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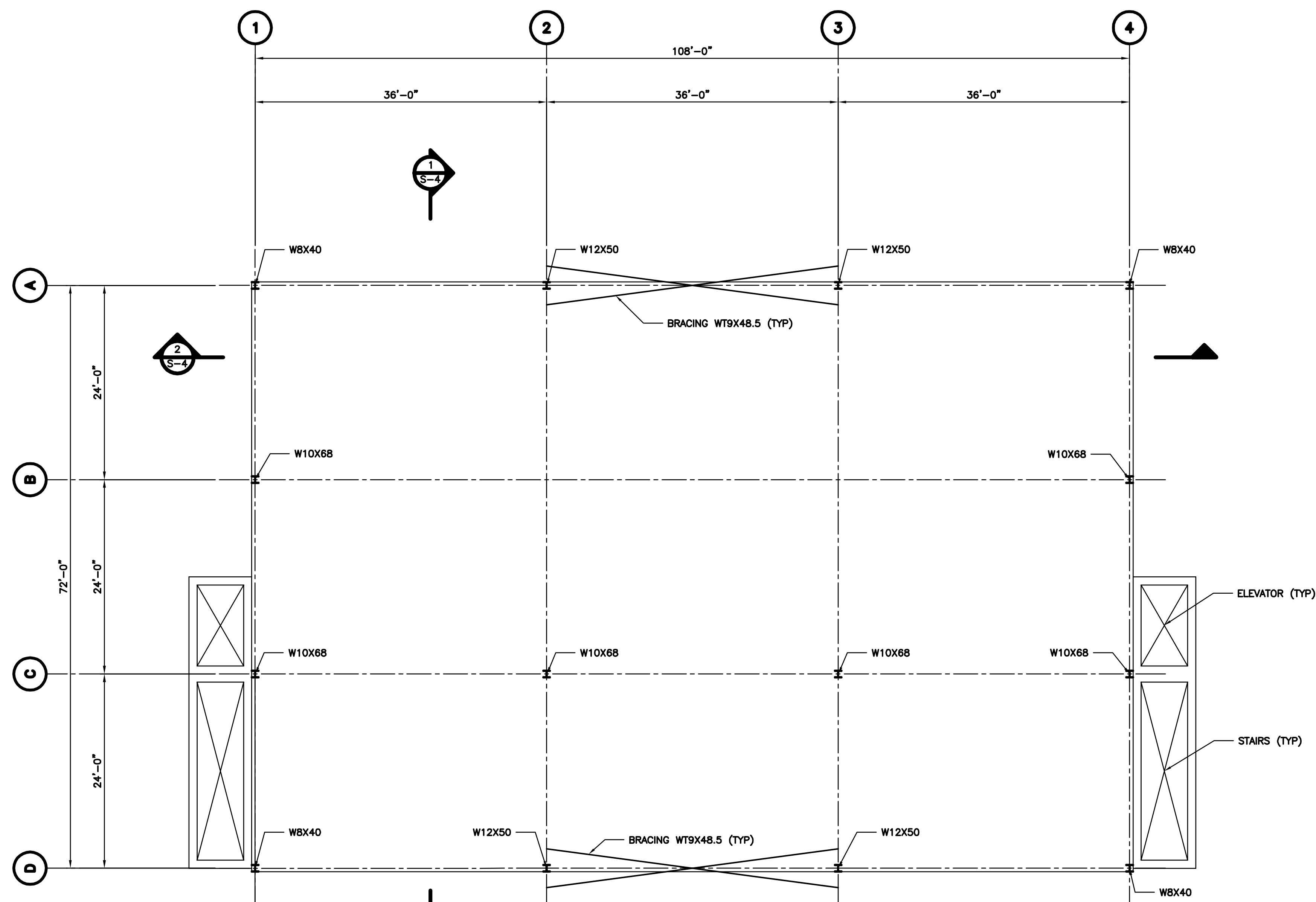

FIRST FLOOR
PLAN
1/8" = 1'-0"

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: Z.ZAVALIANOS
DRAWN BY: M.LASKEY
SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: NOV 2014

14.551 STEEL DESIGN
STEEL DESIGN CLASS PROJECT

PRELIMINARY DESIGN
FIRST FLOOR STEEL PLANS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S001PD
SHEET NO.
S-1

30% SUBMITTAL - NOT FOR CONSTRUCTION

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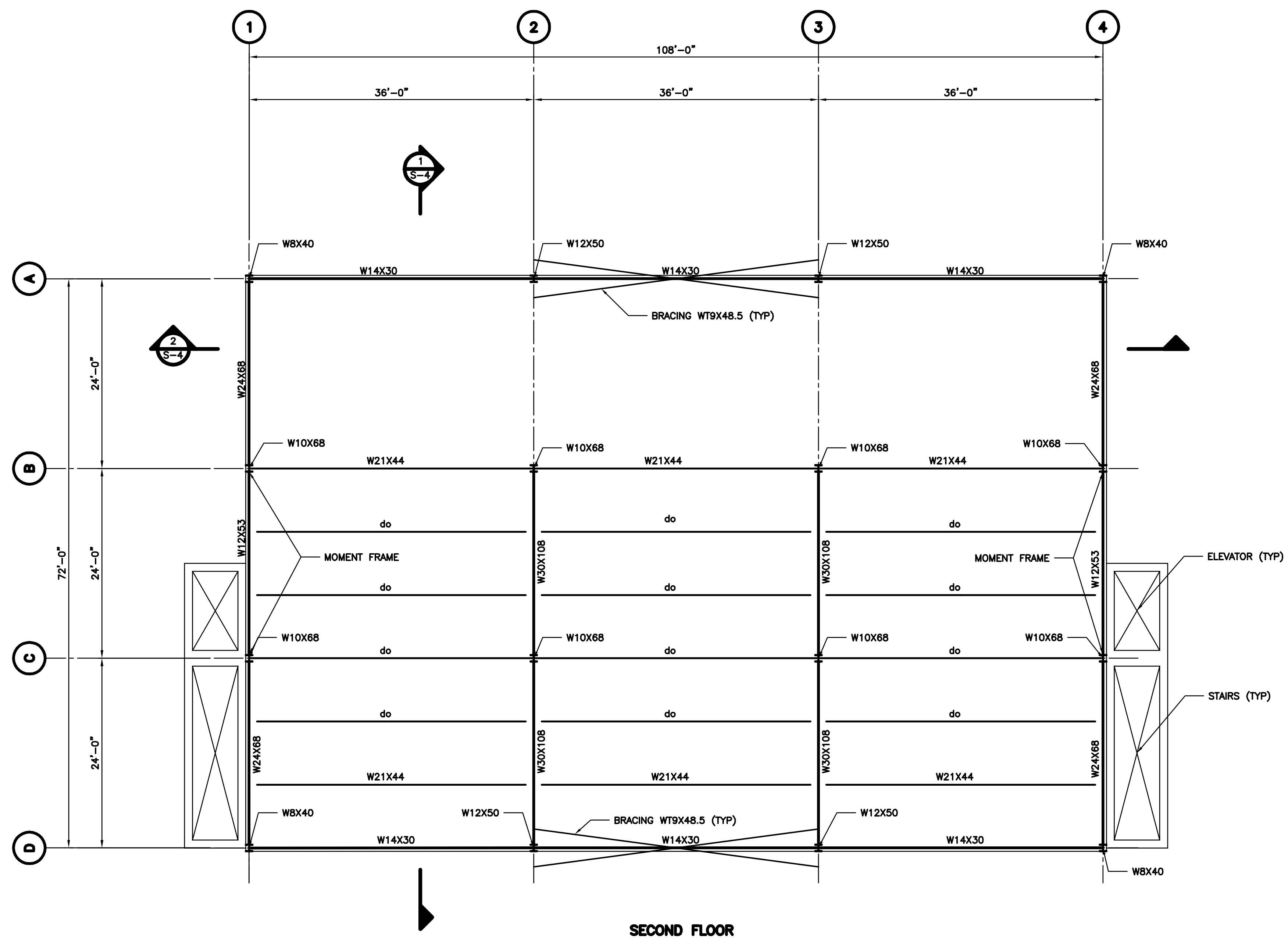

SECOND FLOOR
PLAN
1/8" = 1'-0"

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: Z.ZAVALIANOS
DRAWN BY: M.LASKEY
SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: NOV 2014

14.551 STEEL DESIGN

STEEL DESIGN CLASS PROJECT

PRELIMINARY DESIGN
SECOND FLOOR STEEL PLANS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S002PD
SHEET NO.
S-2

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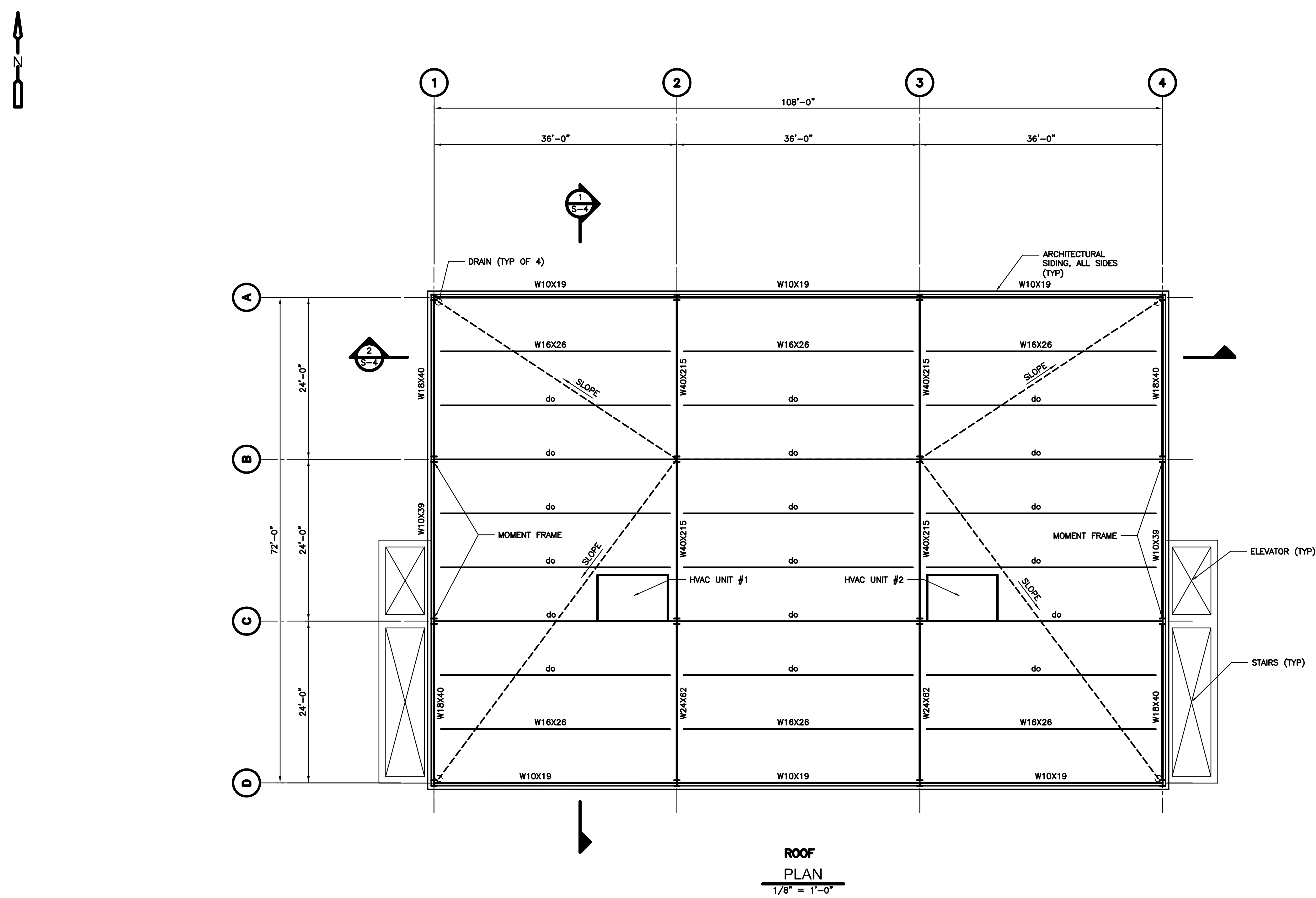

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: Z.ZAVALIANOS
DRAWN BY: M.LASKEY
SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: NOV 2014

14.551 STEEL DESIGN

STEEL DESIGN CLASS PROJECT

PRELIMINARY DESIGN
ROOF STEEL PLANS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S003PD
SHEET NO.
S-3

30% SUBMITTAL - NOT FOR CONSTRUCTION

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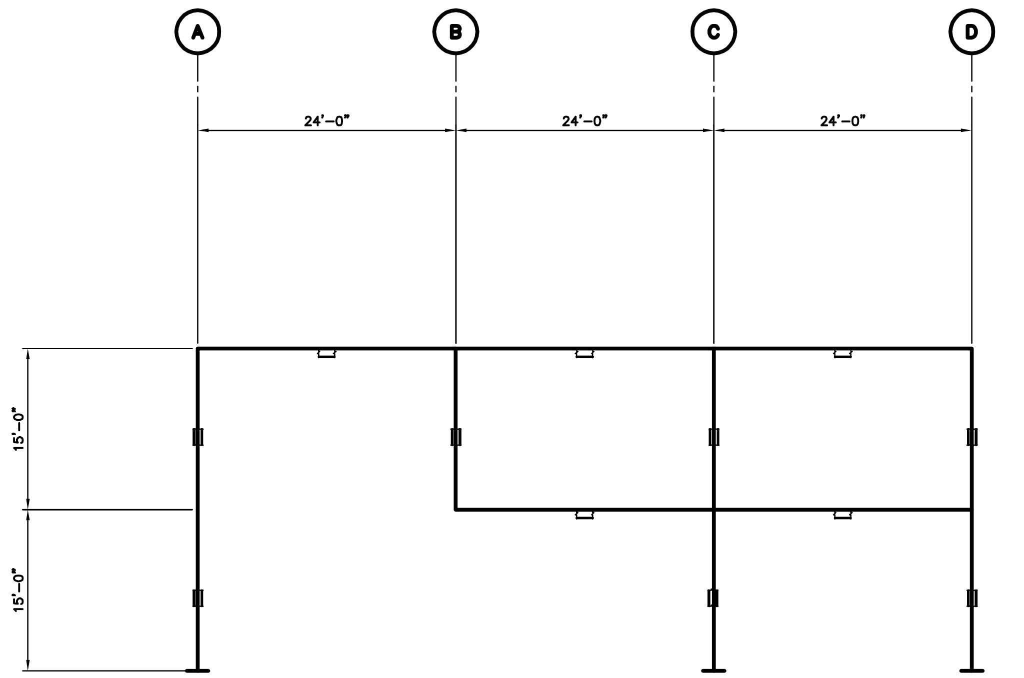
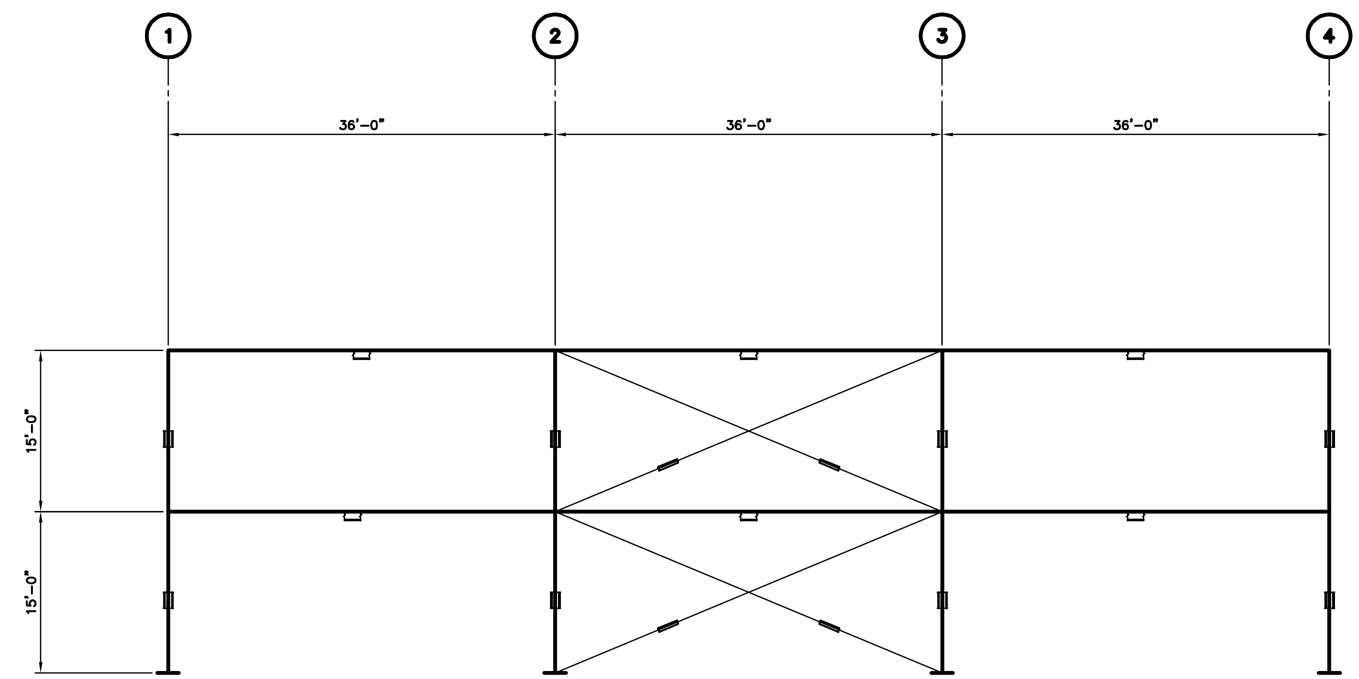


ELEVATION ALONG GRID 2
SECTION 1/–
1/8" = 1'-0"



ELEVATION ALONG GRID A
SECTION 2/–
1/8" = 1'-0"

REV. NO.	DATE	DRWN	CHKD	REMARKS

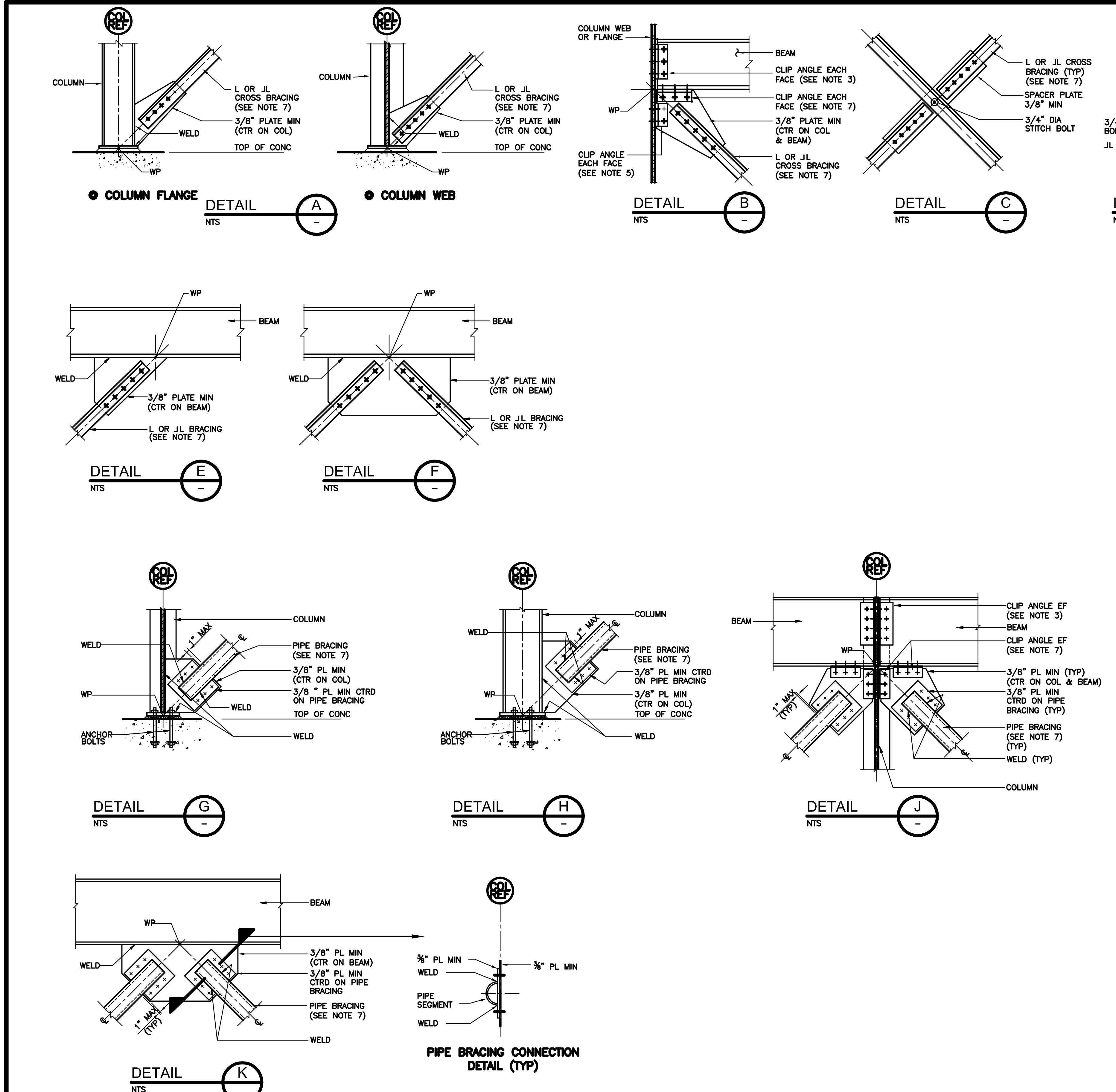
DESIGNED BY: Z.ZAVALIANOS
DRAWN BY: M.LASKEY
SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: NOV 2014

14.551 STEEL DESIGN
STEEL DESIGN CLASS PROJECT

PRELIMINARY DESIGN
STEEL ELEVATIONS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S004PD
SHEET NO.
S-4

30% SUBMITTAL - NOT FOR CONSTRUCTION



STRUCTURAL STEEL NOTES:

1. STRUCTURAL STEEL SHALL BE FABRICATED IN ACCORDANCE WITH THE THIRTEENTH EDITION OF THE AISC MANUAL OF STEEL CONSTRUCTION, PROVISIONS FOR ALLOWABLE STRENGTH DESIGN.

2. MATERIAL FOR STRUCTURAL STEEL SHALL CONFORM TO THE FOLLOWING UON:
STRUCTURAL STEEL WIDE FLANGE SHAPES ASTM A992
STRUCTURAL STEEL HSS TUBE ASTM A500, GRADE B
ALL OTHER STRUCTURAL STEEL SHAPES, PLATES, AND BARS ASTM A36

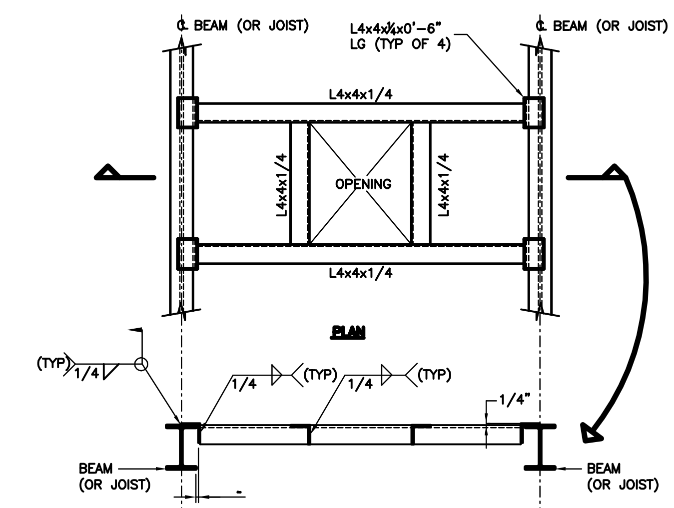
3. ALL BEAM CONNECTIONS SHALL BE DESIGNED FOR ONE HALF OF THE MAXIMUM TOTAL UNIFORM LOAD TABULATED IN TABLE 3-6, PART 3 OF THE AISC MANUAL, OR FOR DESIGN REACTION SHOWN, PLUS AXIAL LOAD (#K T/C) WHERE INDICATED ON THE DRAWINGS, UON. ALL CLIP ANGLES SHALL BE ⅜" MINIMUM THICKNESS.

4. ALL BOLTED CONNECTIONS SHALL BE SLIP-CRITICAL UON.

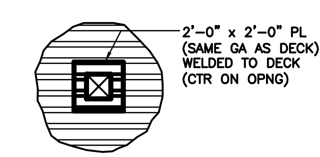
5. FIELD CONNECTIONS SHALL BE BY ASTM A-325 HIGH STRENGTH BOLTS, UON. BOLTS SHALL BE 3/4" DIAMETER IN 13/16" DIAMETER HOLES, UON. WHERE CONNECTIONS ARE TO EXISTING MEMBERS, HOLES SHALL BE FIELD DRILLED.

6. WELDING SHALL BE PERFORMED BY CERTIFIED WELDERS IN ACCORDANCE WITH AWS D1.1"STRUCTURAL WELDING CODE-STEEL". WELDING FILLER METAL SHALL BE AWS A5.1 E70XX ELECTRODES. SURFACES TO BE WELDED SHALL BE WIRE BRUSHED CLEAN BEFORE WELDING. THE AREA TO BE FIELD WELDED SHALL BE CLEANED FREE OF PAINT, DIRT, RUST, GREASE, AND OTHER FOREIGN MATTER PRIOR TO WELDING.

7. BRACING CONNECTIONS SHALL BE DESIGNED FOR AXIAL LOADS SHOWN ON DRAWINGS. FOR VERTICAL BRACING, THE HORIZONTAL COMPONENT OF THE LOAD SHALL BE DEVELOPED IN THE CONNECTION TO THE BEAM OR COLUMN BASE PLATE AND THE VERTICAL COMPONENT IN CONNECTION TO THE COLUMN. WHEN BRACING IS CONNECTED TO ONE MEMBER ONLY, THE CONNECTION SHALL DEVELOP THE FULL LOAD. BOLTED CONNECTIONS SHALL HAVE A MINIMUM OF 2 BOLTS, UON.



STEEL ROOF DECK OPENING 12" AND LARGER (MAX 48")



NOTE: FOR OPNGS < 6" IN BOTH DIRECTIONS, NO ADDITIONAL FRAMING IS REQUIRED.

STEEL ROOF DECK OPENING 6" AND LARGER BUT LESS THAN 12"

ROOF OPENING FRAMING
NTS

NOTE: SEE ROOF FRAMING PLAN FOR OPENING SIZES & LOCATIONS.

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: Z.ZAVALIANOS
DRAWN BY: M.LASKEY
SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: NOV 2014

14.551 STEEL DESIGN
STEEL DESIGN CLASS PROJECT

PRELIMINARY DESIGN
STANDARD STEEL DETAILS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S005PD
SHEET NO. S-5

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