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## Course Description

Session 8: Building Envelope and Bracing Design  
April 3, 2017

Lesson 8 is the final lesson and the final design of the 50-ton overhead crane building design example is presented. The final roof and wall design is presented with a discussion relative to the design of standing seam roofs and to membrane roofs. Advantages and disadvantages of standing seam roofs are presented. Design concerns for metal roof deck and open web joists relative to mechanical membrane roofs subjected to wind uplift forces are presented. Proper specifications for open web joists subjected to gravity, uplift and roof ponding are discussed. Design procedures for cold-formed girts are presented along with wind column design. Lateral bracing calculations for the Ordinary Moment Frame (OMF) columns and beams are performed and discussed. The lesson concludes with suggested general notes for the installation of the runway beams.





## Learning Objectives

- Describe the advantages and disadvantages using a standing seam roof.
- List the key considerations to properly designing open web joists for roof structures.
- Describe the design procedure for rake beam design in an end wall.
- List the requirements for lateral bracing of an Ordinary Moment Frame (OMF).



### Design of Industrial Buildings

Session 8: Building Envelope and Bracing Design  
April 3, 2017



Presented by  
James M. Fisher, PE, PhD  
Emeritus Vice President  
Computerized Structural Design

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## AISC Night School 13

### Design of Industrial Buildings Lesson 8



Presenter:  
James Fisher



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## Design of an Industrial Crane Building

### Lesson 8

#### Roof Design

- Mechanically Fastened Membrane Roofs
- Standing Seam Roof Considerations
- Joist Design
  - Gravity loads, uplift loads, ponding

#### Wall Design

- Girt Design (Endwalls and Sidewalls)
- Rake Beam Design
- Wind Column Design

#### OMF Lateral Bracing Design

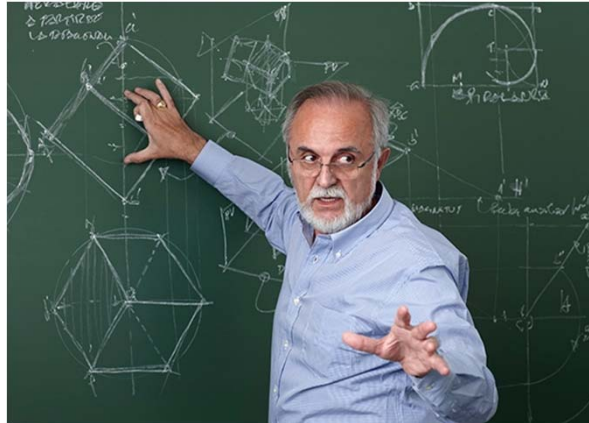


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## Review of Design Criteria

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## Project Description

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- 50 ton, top running crane, Class D
- Quantity: 1 per aisle
- Hook height: 45 ft
- Roof type: Standing Seam on Joists
- Wall type: R- panel with continuous Zees
- Automatic Sprinkler System



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## Codes and Standards

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- Building Code: IBC 2012
- Minimum Design Loads For Buildings And Other Structures (ASCE 7-10)
- Building Department Contact: John Smith
- Date: January 4, 2016
- Local Ordinances: None
- Wind Speed: 115 mph
- Wind Exposure: C
- Building Category II



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## Local Code Requirements

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- Ground Snow Load: 15 psf
- Frost Depth: 24 in.



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## Loads

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- ROOF DEAD LOAD
  - Roofing (SSR) 2.0 psf
  - Insulation 1.0 psf
  - Roof Bracing 1.0 psf
  - Joists 3.0 psf
  - Joist Girders 3.0 psf
  - Columns 6.0 psf
  - MEP Allowance 3.0 psf
  - **Total 19.0 psf**
- WALL DEAD LOAD **3.0 psf**  
(Includes Girts)



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## Loads

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- ROOF LIVE LOADS
  - 20.0 psf (reduceable)
- SNOW LOADS
  - Ground Snow Load ( $P_g$ ): 15.0 psf
  - Importance Factor,  $I = 1.0$
  - Terrain Factor:  $C$
  - Thermal Factor,  $C_t$ : 1.0
  - Exposure Factor,  $C_e$ , partially exposed: 1.0
  - Flat Roof Snow Load:  $P_f = 0.7(P_g)(I)(C_e) = 10.5 \text{ psf} < \text{Use } 20 \text{ psf}$
  - Building Category: II



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## Loads

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### • SEISMIC LOADS

- Spectral Acceleration,  $S_s$ : 1.054g
- Spectral Acceleration,  $S_1$ : 0.400g
- Occupancy Category: II
- Site Class: D
  - Soil shear wave velocity,  $\bar{V}_s$ : 800 ft/sec
  - Standard penetration resistance,  $\bar{N}$ : 15 blows
  - Soil undrained shear strength,  $\bar{\tau}_u$ : 1500 psf
- Importance Factor,  $I$ : 1.0
  
- Seismic Design Category: D



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## Loads

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### WIND LOADS

- Occ. II Risk category, Table 1.5-1
- $V = 115$  Basic wind speed (3 second gust), mph, Fig. 26.5-1A
- $Exp = C$  Exposure category, Section 26.7
- $K_d = 0.85$  Wind directionality factor, Section 26.6 & Table 26.6-1
- $K_{zt} = 1$  Topographic factor, Section 26.6 & Fig. 26.8-1
- $Encl.$  = Enclosure classification, Section 26.10
- $R = 1$  Large volume buildings reduction factor, Section 26.11.1.1
- $G = 0.85$  Gust factor, Section 26.9



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## Completed Design

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- Final frame design
- Roof horizontal bracing
- Sidewall vertical bracing
- Endwall vertical bracing

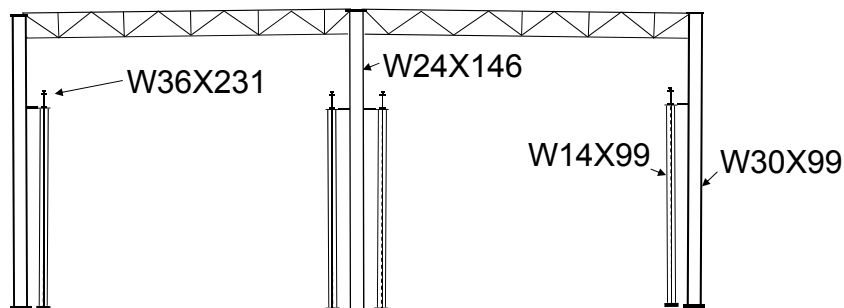


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## Final Frame Design

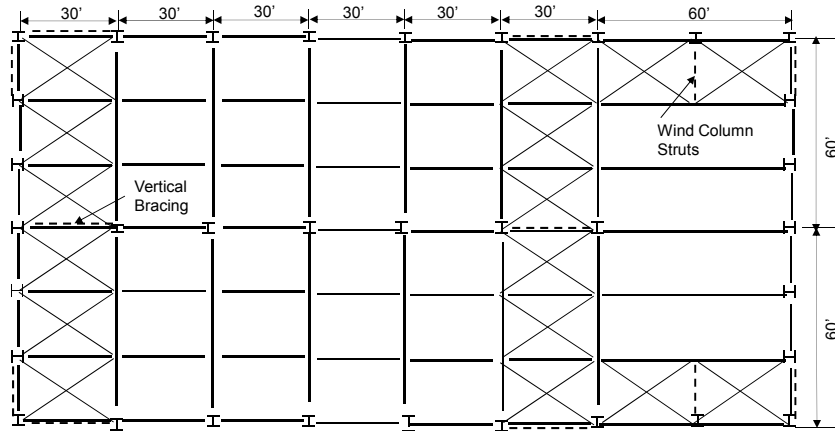
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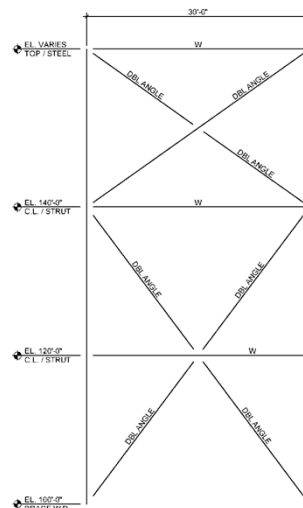
## Roof Horizontal Bracing



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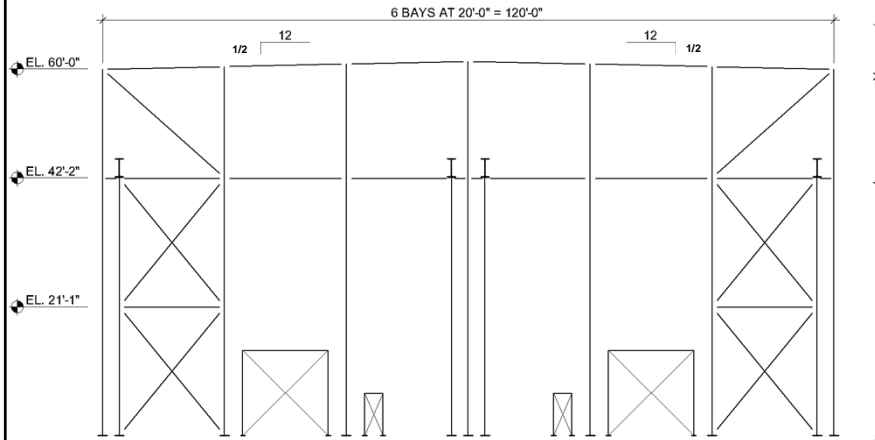
## Sidewall Vertical Bracing



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## End Wall Vertical Bracing



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## Roof Design

- Mechanically Fastened Membrane Roofs
- Standing Seam Roofs



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## Mechanically Fastened Membrane Roofs

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## Usage

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- In the 1960s, single-ply membrane roof systems were first introduced into the U.S. roofing market and by the late 1970s, the **seam-fastened, mechanically-attached** method of installation was first introduced.
- With this installation method, the single-ply membrane sheet is **mechanically-attached along its outer edges** into the roof deck, which results in a larger tributary uplift load per fastener and fasteners being placed in linear, non-uniform loading configurations of the roof deck and underlying supporting structure



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## Usage

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- When first introduced, membrane sheet widths in seam-fastened single-ply membrane roof systems typically were **five-foot-wide**, resulting in rows of mechanical fasteners in rows spaced at five feet on center.
- Since the early-2000s, single-ply membrane sheet widths have gotten wider, with **10-foot-wide sheets** now being commonplace -- resulting in rows of mechanical fasteners in rows spaced at 10 feet on center.
- In some cases, 12-foot wide sheets are being used and speculation is that **20-foot sheets** may be in the future.



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## Usage

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- The seam-fastened, mechanically-attached method of installation has overtaken taken adhered methods of application used traditionally.

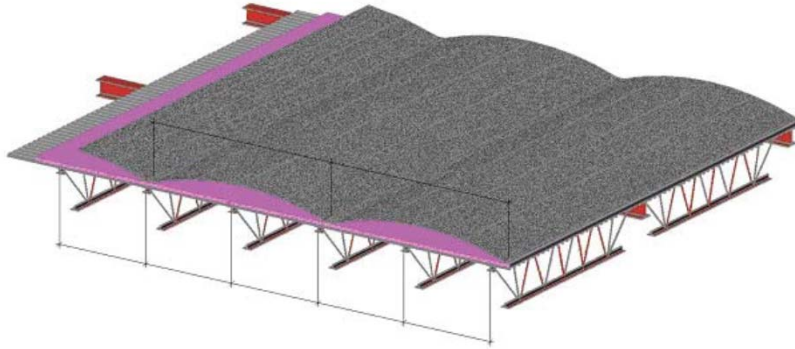


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## Deck and Joist Loading

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## The Problem

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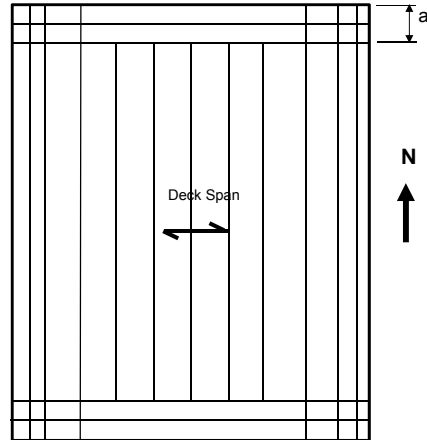
From the National Research Council of Canada



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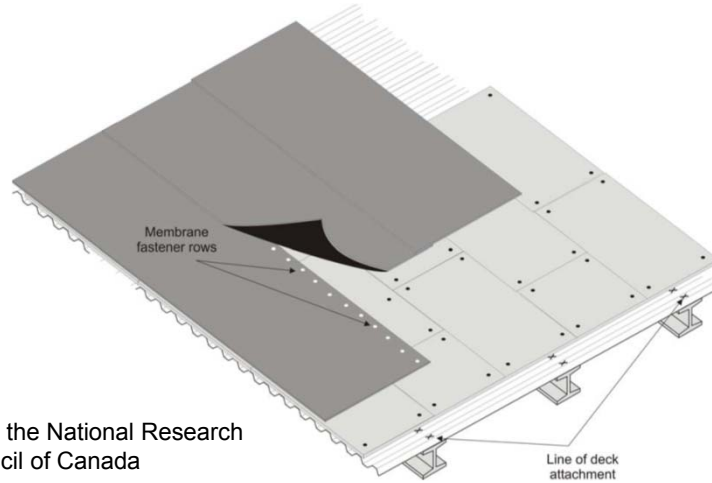
## Typical Layout



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## Fasteners Perpendicular to Deck Flutes



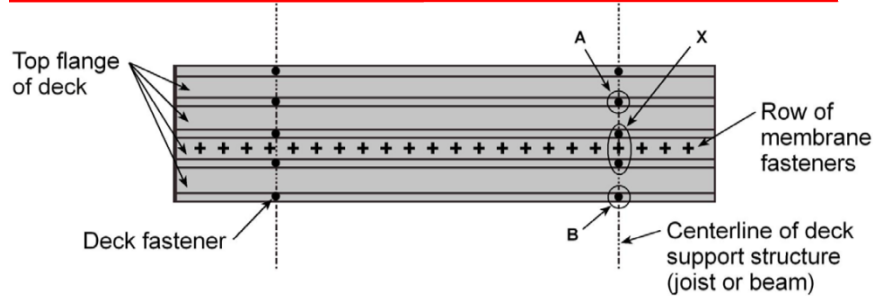
From the National Research  
Council of Canada



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## Fasteners Parallel to Deck Flutes



In this case, the 2 X deck fasteners must resist essentially 100% of the uplift load applied by the membrane fasteners because the steel deck transmits very little of the load to the adjacent deck fasteners A and B

From the National Research Council of Canada



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## Calculations for Deck Strength

Zone	Moment Line Load, $M_r$ (kip-inches)	$M_r / \phi M_p$ or $M_r / \phi M_n$	Moment Uniform Load, $M_r$ (kip-inches)	$M_r / \phi M_p$ or $M_r / \phi M_n$
Interior	4.85 <sup>1</sup>	0.95	1.24 <sup>2</sup>	0.23
East-West Perimeter	4.32 <sup>1</sup>	0.81	2.31 <sup>2</sup>	0.43
North-South Perimeter	11.9 <sup>2</sup>	<b>2.22</b>	2.22 <sup>2</sup>	0.41
Corner	11.55 <sup>2</sup>	<b>2.16</b>	3.48 <sup>2</sup>	0.65

1. Positive moment controls, 2. Negative moment controls  
 $\phi M_p = 5.088$  kip-in.,  $\phi M_n = 5.358$  kip-in.



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## Deck and Joist Design Case Study

- In order to comply with the IBC, the deck in the along the North-South Perimeter and the Corner Zones would have to be 16 gage.
- Joist Design:
  - Analysis indicates that the joists 6 ft. and 12 ft. from the building edges are not overloaded for the given geometry; however, the joists in the interior zone are significantly overloaded.
  - Design joists using ASD.
  - Joist Span = 33.3 ft., spacing = 6 ft.
  - Live Load = 20 psf, Dead Load = 6 psf



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## Joist Design Case Study

The unfactored uplift wind load = 33.3 psf

Uniform Net Uplift:  $0.6D + 0.6W = [(0.6)(6\text{psf}) - (0.6)(33.3\text{psf})](6\text{ft}) = 98 \text{ plf}$

Concentrated Net Uplift:  $(0.6)(6\text{psf})(6\text{ft.}) - (0.6)(33.3\text{psf})(12\text{ft}) = 218 \text{ plf}$

### Joist Weight (Normal Case):

Uniform Case Gravity Load =  $(6\text{psf})(6\text{ft}) + (20\text{psf})(6\text{ft}) = 156 \text{ plf}$

Use 18K156/120, Net Uplift = 98 plf

**Weight/joist = 215 lbs**

### Joist Weight (12 ft Membrane Case):

Concentrated Uplift Load Case: Net Uplift = 218 plf

Use 18K156/120, Net Uplift = 218 plf

**Weight/joist = 260 lbs**

**21% Increase.**



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## Paper in Structure Magazine – March 2017

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### Are your roof members overstressed?

By: James M. Fisher, Ph.D., PE, Dist. M. ASCE , and  
Thomas Spoto, Ph.D., PE, SE, F.ASCE



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## Roof Design

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- Standing Seam Roofs
  - Spans
  - Slope
  - Diaphragm
  - Anchorage
  - Snow drift areas



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## Standing Seam Roofs

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- Spans: 5 ft
- Slope:  $\frac{1}{4}$  in./ft minimum  
–  $\frac{1}{2}$  in./ft used
- Diaphragm: Limited
- Anchorage: One end
- Snow drift areas: Add secondary members



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## Standing Seam Roofs

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- **Advantages**
  - The system is flexible and can accommodate the building movements associated with the overhead crane.
  - The roof can accommodate thermal movements without slotting the roofing at screw locations. This is particularly important when placing a metal roof on open web steel joists. Joist seats are stiff laterally, unlike cold-formed C or Z purlins, and do not roll as the roof attempts to expand and contract due to thermal loading.



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## Standing Seam Roofs

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- **Disadvantages**

- First cost is greater than conventional roofs.
- Limited diaphragm strength and stiffness.  
Diaphragm strength cannot be relied upon to transfer horizontal roof forces to the vertical load resisting system. Other horizontal bracing systems must be used.



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## Roof Design Calculations

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- Joists
- Girts
- Rake Beams
- Wind Columns



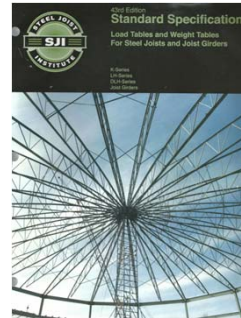
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## Joist Design

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- Gravity Loads
- Ponding
- Seat Depths
- Wind Uplift



Free download from [steeljoist.org](http://steeljoist.org)



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## Joist Design- Gravity Loads

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### 30 ft joists

Dead Load = 10 psf, Live Load = 20 psf,  $w = (10 \text{ psf} + 20 \text{ psf})(5 \text{ ft}) = 150 \text{ plf}$   
Try 18K3: 203 plf/123 plf

### 60 ft joists

Try 30K11: 231 plf/109 plf



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## Joist Calculations- Ponding

The structure is free draining to the eave; however, check to determine if the water can drain off the roof based on the deflection of the joists and Joist Girders. The eave is assumed not to deflect. Ignore joist and Joist Girder camber. Critical location is the first joist up from the eave.

30 ft joists (18K3) 203 plf/123 plf

$$I = 26.767(w_{LL})(L^3)(10^{-6}) \text{ in.}^4 \quad (\text{From SJI Catalog})$$

$$L = 30 \text{ ft} - 0.33 \text{ ft} = 29.67 \text{ ft}$$

$$I = (26.767)(123 \text{ plf})(29.67 \text{ ft})^3(10^{-6}) = 86 \text{ in.}^4$$

$$\Delta = \frac{(1.15)(5)w_{TL}L^4}{384EI} = \frac{(1.15)(5)(0.150)(29.67)^4(1728)}{(384)(29000)(86)} = 1.21 \text{ in.}$$

$$\text{Roof slope} = (0.50 \text{ in./ft})(5 \text{ ft}) = 2.50 \text{ in.}$$

1.21 in.  $\leq$  2.50 Positive slope exists o.k., not considering the Joist Girder deflection.



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## Joist Calculations- Ponding

- Determine the deflection of the Joist Girder 5 ft. upslope from the eave.
- The deflection at any point, x, along the Joist Girder can be determine using the following equation:

$$\Delta = \frac{wx}{24EI} (L_g^3 - 2L_g x^2 + x^3) \text{ From AISC Manual Table 3-23}$$

$$w = (10 \text{ psf} + 20 \text{ psf})(30 \text{ ft})/1000 = 0.90 \text{ kips/ft, } x = 5 \text{ ft, } L_g = 60 \text{ ft}$$

$$\text{From Lesson 4: } I_g = 13,167 \text{ in.}^4$$

$$\begin{aligned} \Delta &= \frac{wx}{24EI} (L_g^3 - 2L_g x^2 + x^3) \\ &= \frac{(0.90)(5)(1728)}{(24)(29000)(13,167)} [(60)^3 - 2(60)(5)^2 + (5)^3] = 0.18 \text{ in.} \end{aligned}$$

$$\text{Total joist deflection} = 1.21 \text{ in.} + 0.18 \text{ in.} = 1.39 \text{ in.} < 2.50 \text{ in. o.k.}$$



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## Joist Calculations- Ponding

60 ft joists (30K11) 231 plf/109 plf

$$I = 26.767(w_{LL})(L^3)(10^{-6}) \text{ in.}^4$$

$$L = 60 \text{ ft} - 0.33 \text{ ft} = 59.67 \text{ ft}$$

$$I = (26.767)(109 \text{ plf})(59.67 \text{ ft})^3(10^{-6}) = 620 \text{ in.}^4$$

$$\Delta = \frac{(1.15)(5)w_{TL}L^4}{384EI} = \frac{(1.15)(5)(0.150)(59.67)^4(1728)}{(384)(29000)(620)} = 2.74 \text{ in.} > 2.50 \text{ in. n.g}$$

Joist Girder  $\Delta$ ,  $w = (30 \text{ psf})(45 \text{ ft}) / 1000 = 1.35 \text{ kips/ft}$

$$\Delta = (0.18)(1.35/0.9) = 0.27 \text{ in.}$$

Joist  $I_{req'd}$  using  $\Delta = 2.50 - 0.27 = 2.23 \text{ in.}$ ,  $I_{req'd} = 760 \text{ in.}^4$

Solving for  $w_{LL} = 134 \text{ plf}$

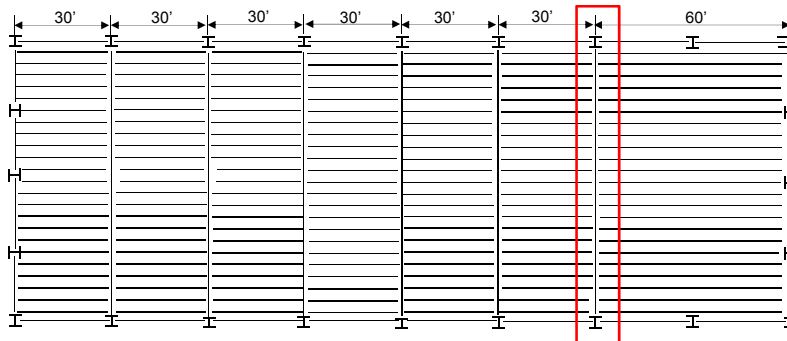
Use 32LH07, 271 plf / 140 plf,  $I_{furnished} = 796 \text{ in.}^4$



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## Roof Plan-Seat Depths



Typical 30 ft. joist: 18K3  
 Typical 60 ft. joist: 32LH07



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# Joist Wind Roof Suctions

Using Spreadsheet:

Part 1: Low-Rise Buildings  
 Gable Roof Pressures - Figure 30.4-2A (  $h < 60$  feet &  $\theta \leq 7^\circ$  )

Location	Tributary Area, ft. <sup>2</sup>							
	300		10		50		50	
	GCp	PSF	GCp	PSF	GCp	PSF	GCp	PSF
roof pressure - 1, 2, 3	0.20	12.4	0.30	15.7	0.23	13.4	0.23	13.4
roof suction - field - 1	-0.90	<b>-35.3</b>	-1.00	-38.6	-0.93	-36.3	-0.93	-36.3
roof suction - edge - 2	-1.10	-41.9	-1.80	-64.8	-1.31	-48.8	-1.31	-48.8
roof suction - corner - 3	-1.10	-41.9	-2.80	-97.5	-1.61	-58.6	-1.61	-58.6

Tributary area for joists =  $(1/3)(30)(30) = 300 \text{ ft}^2$



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# Girder Roof Suctions

Using Spreadsheet:

Wind Loads For A Specific Height, h (Used for Components & Cladding)

Height	Kz	q	WW	LW	Total	Side	Roof WL			Int.
			WL	WL	WL	WL	0 to h	h to 2h	> 2h	WL
0-15	0.85	24.43	16.6							
<b>60</b>	1.14	32.71	22.2	13.9	36.1	-19.5	<b>-25.0</b>	-13.9	-8.3	<b>5.9</b>



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## Joist Calculations

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### Uplift requirements (Gross)

$$\text{Joists} = (0.6)(35.3 \text{ psf}) = 21.2 \text{ psf}$$

$$\text{Girders} = (0.6)(25 \text{ psf} + 5.9 \text{ psf}) = 18.5 \text{ psf}$$

### Uplift requirements (Net)

$$\text{Joists} = 21.2 \text{ psf} - (0.6)(7 \text{ psf}) = 17.0 \text{ psf}$$

$$\text{Girders} = 18.5 \text{ psf} - (0.6)(10 \text{ psf}) = 12.5 \text{ psf}$$

#### ROOF DEAD LOADS

Roofing (SSR)	2.0 psf
Insulation	1.0 psf
Roof Bracing	1.0 psf
Joists	3.0 psf
Joist Girders	3.0 psf
Columns	6.0 psf
MEP Allowance	3.0 psf
<b>Total</b>	<b>19.0 psf</b>



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## Girt Design

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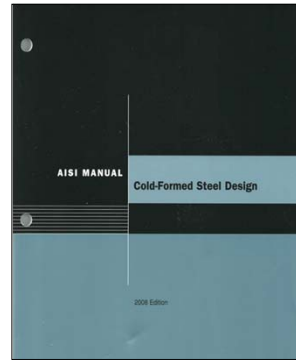
- Cold-Formed Girts
  - Pressure and Suction
  - Maximum spans
  - Simple vs continuous span
  - Sag rods
  - OSHA requirements



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## Reference Material



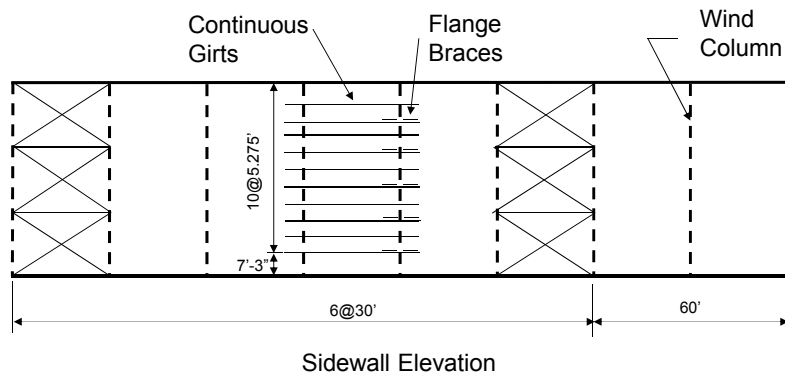
1. Download all the AISI new standards from the link ([www.AISIStandards.org](http://www.AISIStandards.org))
2. Order the AISI Cold-Formed Steel Design Manual from the AISI online store or via the link (<https://shop.steel.org/p/312/cold-formed-steel-design-manual-2013-edition-electronic-version-includes-aisi-s1-12-specification-and-commentary>).



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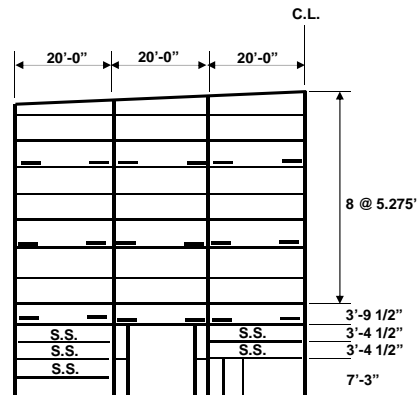
## Girt Design- Sidewalls



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## Girt Design- Endwalls



Endwall



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## Girt Design

- Pressure and Suction Loads from IBC
- For pressure, the girts are laterally braced by the paneling.  $M_n = S_e F_y$   
 $\phi = 0.9, \Omega = 1.67$
- For suction, the strength of the girts is established using "R" values from the AISI Specification.  $M_n = R S_e F_y$   
 $\phi = 0.9, \Omega = 1.67$
- Sag rods are often used when the span exceeds 30 ft.



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## Girt Design- AISI Specification

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TABLE D6.1.2.1  
Single Span C- or Z- Section R Values

Depth Range, in. (mm)	Profile	R
$d \leq 6.5$ (165)	C or Z	0.70
$6.5$ (165) $< d \leq 8.5$ (216)	C or Z	0.65
$8.5$ (216) $< d \leq 11.5$ (292)	Z	0.50
$8.5$ (216) $< d \leq 11.5$ (292)	C	0.40



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## Girt Design- Windloads

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Sidewall values:

- Tributary Area =  $[(5.275 \text{ ft} + 7.25 \text{ ft})/2](30 \text{ ft}) = 188 \text{ sq. ft}$
- Need not be  $< (1/3)(30)(30) = 300 \text{ sq.ft}$

Endwall values:

- Tributary Area (Typical girt) =  $(5.275)(20 \text{ ft}) = 106 \text{ sq. ft}$
- Need not be  $< (1/3)(20)(20) = 133 \text{ sq.ft}$

Conservatively use endwall values for all girts.



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## Girt Endwall Pressures

Using Spreadsheet:

SECONDARY FRAMING:  $p = qh[(GCp)-(GCpi)]$

qh = 32.71 psf	roof pitch	0.25	x/12
GCpi = 0.18 internal coefficient	roof slope	1.19	degrees
	reduction factor	0.9	

Wall Pressures - Figure 30.4-1 ( h ≤ 60 feet )

Location	Tributary Area, ft <sup>2</sup>					
	133		10			
	GCp	P	GCp	P	GCp	P
wall pressure - 4 & 5	0.72	29.5	0.90	35.3		
wall suction - field - 4	-0.81	-32.4	-0.99	-38.3		
wall suction - edge - 5	-0.90	-35.4	-1.26	-47.1		



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## Girt Design

From AISI Specification D6.1.1:  $\Omega = 1.67$ ,  $R = 0.5$  for s.s. girt

Endwall:  $w = (5.275)(0.6)(35.4 \text{ psf}) = 112 \text{ plf} = 0.112 \text{ klf}$

$M = (1/8)wL^2 = (1/8)(0.112 \text{ plf})(20)^2 = 5.60 \text{ kip-ft}$

$$S_{req'd} = \frac{\Omega M}{RF_y} = \frac{(1.67)(5.6 \text{ kip-ft})(12 \text{ in./ft})}{(0.5)(50 \text{ ksi})} = 4.49 \text{ in.}^3 \text{ o.k.}$$

Specify: 10Z3.25x105  $S_x = 5.69 \text{ in.}^3$  (From AISI Manual)

Note: depth = 10 in., 3.25 flange width, 0.105 thickness



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# Design of Rake Beam

Seismic Loads Control:

$P_r$  (from analysis) = 24.6 kips

$M_r = (1/8)wL^2 = (1/8)[(1+0.14S_{DS})(0.15 \text{ klf}) + 0.30 \text{ klf}](20)^2 = 23.3 \text{ kip-ft}$

$S_{DS} = 0.770g$  (from Lesson 5)

$w_D = (10 \text{ psf})(30 \text{ ft} / 2) / 1000 = 0.15 \text{ klf}$

$w_L = (20 \text{ psf})(30 \text{ ft} / 2) / 1000 = 0.30 \text{ klf}$

$L = 20 \text{ ft}$

**Try W10x33**

From AISC Manual Table 6-1:  $p = 3.8 \times 10^3$ ,  $b_x = 9.18 \times 10^3$

Ratio =  $[(0.0038)(24.6) + (0.00918)(23.3)] = 0.31 < 1.0$  o.k.

Use W10x33 (seismically compact)



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# Wind Column Design

## MWFRS WIND LOAD CALCULATIONS (ASCE 7-10)

Project: ABC Building Job No. \_\_\_\_\_  
 By: JMF  
 Date: 5/4/16

$qz = 0.00256Kz \cdot Kzt \cdot Kd \cdot V^2$  (Eq. 27.3-1)

- Occ. = **II** risk category, Table 1.5-1
- V = **115** basic wind speed (3-second gust), mph, Figure 26.5-1A
- Exp = **C** exposure category, Section 26.7
- Kd = **0.85** wind directionality factor, Section 26.6 & Table 26.6-1
- Kzt = **1** topographic factor, Section 26.8 & Figure 26.8-1
- End. = **E** enclosure classification, Section 26.10
- Rf = **1** large volume buildings reduction factor, Section 26.11.1.1
- G = **0.85** gust factor, Section 26.9
- zg = 900 atmospheric boundary layer, ft., Table 26.9-1
- $\alpha = 9.5$  3-sec gust speed power law exponent, Table 26.9-1

Pressure Coefficients per  
 Table 26.11-1 & 27.4-1

Height	Kz	qz	Wind Pressures, psf							
			WW WL	LW WL	Total WL	Side WL	Roof WL			Internal WL
			0.8	0.5		-0.7	0 to h	h to 2h	> 2h	
0-15	0.85	24.43	16.6	10.4	27.0	-14.5	-18.7	-10.4	-6.2	4.4
20	0.90	25.95	17.6	11.0	28.7	-15.4	-19.9	-11.0	-6.6	4.7
30	0.98	28.27	19.2	12.0	31.2	-16.8	-21.6	-12.0	-7.2	5.1
40	1.04	30.03	20.4	12.8	33.2	-17.9	-23.0	-12.8	-7.7	5.4
50	1.09	31.48	21.4	13.4	34.8	-18.7	-24.1	-13.4	-8.0	5.7
60	1.14	32.71	22.2	13.9	36.1	-19.5	-25.0	-13.9	-8.3	5.9



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## Wind Columns

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### Sidewall Wind Columns:

$$\text{Use } p = (0.6)(22.2 \text{ psf} + 5.9 \text{ psf}) = 16.9 \text{ psf}$$

$$\text{Pressure: } w = (16.9 \text{ psf})(30 \text{ ft})/1000 = 0.516 \text{ k/ft (Use for suction also)}$$

$$M = (1/8)wL^2 = (1/8)(0.516 \text{ k/ft})(60 \text{ ft})^2 = 232 \text{ kip-ft (Use Table 3-10)}$$

Lateral bracing at  $\approx 10$  ft centers, Try W18x55,  $I_x = 890 \text{ in.}^4$

$$\Delta = \frac{5wL^4}{384EI} = \frac{(5)(0.516 \text{ k / ft})(60 \text{ ft})^4(1728)}{(384)(29000 \text{ ksi})(890 \text{ in.}^4)} = 5.83 \text{ in.}$$

$$\text{Serviceability (10 year wind), } H/120 = (60 \text{ ft})(12)/120 = 6.0 \text{ in.} > 5.83 \text{ in.}$$

5.83 in. was for a 50 year wind.

$$\text{Reaction at the top of column: } (16.9 \text{ psf})(60 \text{ ft}/2)(30 \text{ ft})/1000 = 15.2 \text{ kips}$$



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## Wind Columns

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Endwall: Consider MWFRS

$$p = (16.9 \text{ psf})(20) = 338 \text{ plf} = 0.338 \text{ klf}$$

$$M = (1/8)wL^2 = (1/8)(0.338 \text{ k/ft})(60 \text{ ft})^2 = 152 \text{ kip-ft (Use ASIC Table 3-10)}$$

Use a W21x44,  $I_x = 843 \text{ in.}^4$

Deflection ok by inspection



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## Bracing of Sidewall Columns

- AISC 2010 Specification Appendix 6  
“Stability Bracing for Columns and Beams”
  - Beam columns
  - Nodal v. Relative Bracing
  - Beam Bracing
    - Nodal
    - Relative
    - Torsional

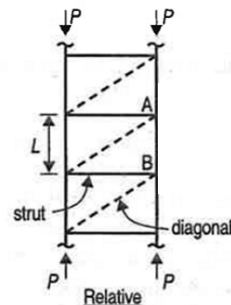


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## Relative Brace

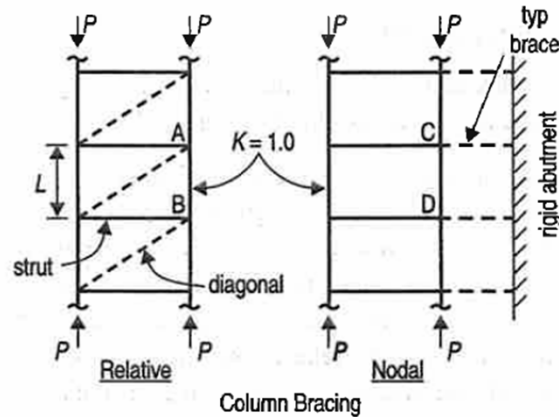
- A relative column brace system (such as diagonal bracing or shear walls) is attached to two locations along the length of the column that defines the unbraced length. The relative brace system shown consists of the diagonal and the strut that controls the movement at one end of the unbraced length, *A*, with respect to the other end of the unbraced length, *B*.



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## Relative and Nodal Braces

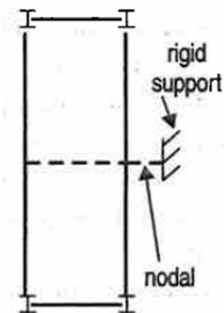


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## Nodal Brace

A nodal brace controls the movement only at the particular brace point, without direct interaction with adjacent braced points. The two nodal column braces at C and D that are attached to the rigid abutment define the unbraced length for which  $K=1.0$  can be used.

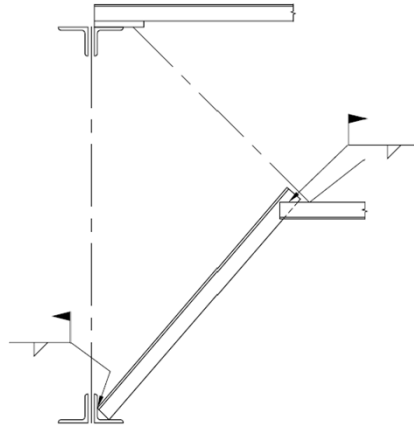


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## Girder Bracing

Typical girder bracing  
for stability and wind  
uplift

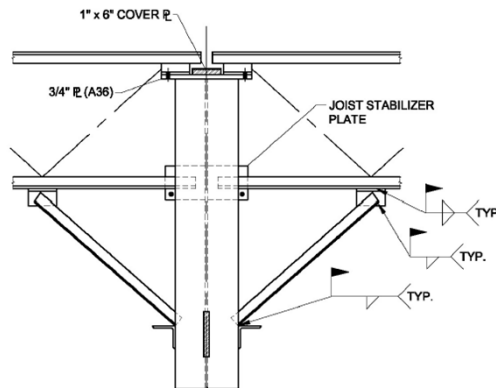


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## Girder Bracing

Typical girder bracing at plastic hinge locations



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## Bracing of Sidewall Columns

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W30x99

Use AISC Specifications Appendix 6

Two conditions exist for bracing the sidewall columns:

Condition 1: The column exterior flange is in compression, this braced directly by the girts and panels.

Condition 2: The column interior flange is in compression and braced using a flange brace from the girt.

The sidewall columns are functioning primarily as beams thus the beam equations will be used.



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## Bracing of Sidewall Columns

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Relative bracing equations are appropriate.

Strength:  $P_{br} = 0.008M_r C_d / h_o$

Stiffness:  $\beta_{br} = \Omega \frac{4M_r C_d}{L_b h_o} (ASD) = \frac{4M_r C_d}{\phi L_b h_o} (LRFD)$

$\Omega = 2.0, \phi = 0.75$

$h_o$  = distance between flange centroids, in.

$C_d = 1.0$  bending in single curvature

$L_b$  = laterally unbraced length, in.

$M_r$  = required flexural strength, kip-in.



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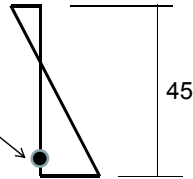
72

## Bracing of Sidewall Columns

### Condition 1: Exterior Flange Compression

W30x99

Brace located 7.25 ft  
 from column base



Moment at brace = 272 kip-ft

By inspection the stiffness will satisfy the stiffness equation.

$$P_{br} = 0.008M_r C_d / h_o = (0.008)(272 \text{ k-ft})(12)(1.0) / 29.0 \text{ in.} \\ = 0.90 \text{ kips}$$

Diaphragm strength: 900 lbs/30 ft = 30 lbs/ft

Typical R panel shear value = 140 lbs/ft ok



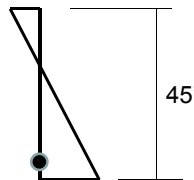
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## Bracing of Sidewall Columns

### Condition 2: Interior Flange Compression

Brace located 7.25 ft  
 from column base



Moment at brace = 235 kip-ft

Determine brace stiffness: End span condition, cont. girts

Use L2x2x3/16 for flange brace,  $A_{brace} = 0.722 \text{ in.}^2$

Z girt (10Z3.25x105),  $A = 1.88 \text{ in.}^2$ ,  $I_x = 28.4 \text{ in.}^4$



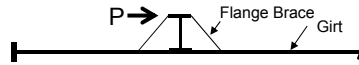
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## Bracing of Sidewall Columns

From analysis:

$$\Delta = 0.05 \text{ in.}, P = 1.0 \text{ kips}, \beta_{\text{furnished}} = 1.0/0.05 = 20 \text{ kips/in.}$$



$$\beta_{br} = \Omega \frac{4M_r C_d}{L_b h_o} (ASD) = 2 \frac{(4)(235 \text{ kip-ft})(1.0)}{(7.25 \text{ ft})(29.0 \text{ in.})} = 8.9 \text{ kips/in.}$$

8.9 kips/in. < 20 kips/in. o.k.

$$P_{br} = 0.008 M_r C_d / h_o = (0.008)(235 \text{ k-ft})(12)(1.0)/29.0 \text{ in.} \\ = 0.78 \text{ kips}$$



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## Bracing of Sidewall Columns

Determine Flange Brace Angle Strength:

$$\text{Length, } L = (30 \text{ in.})(1.414) = 42.4 \text{ in. (column depth} = 30 \text{ in.)}$$

$$r_x = 0.612 \text{ in.}^2$$

$$L/r_x = 42.4 \text{ in.} / 0.612 \text{ in.} = 69.0$$

Equivalent slenderness from AISC Section E5

$$KL/r_x = 72 + 0.75(L/r_x) = 72 + (0.75)(69) = 124$$

From Manual Table 4-22:  $F_{cr}/\Omega = 9.59 \text{ ksi}$

$$P_a = (9.59)(0.722) = 6.92 \text{ kips} > (1.414)(0.78) = 1.10 \text{ kips ok}$$



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# The End

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## End of Lesson 8 and Night School 13



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# Individual Webinar Registrants

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## PDH Certificates

Within 2 business days...

- You will receive an email on how to report attendance from: [registration@aisc.org](mailto:registration@aisc.org).
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!



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## Individual Webinar Registrants

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- Password: Same as AISC website password.



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## 8-Session Registrants

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**PDH Certificates**  
One certificate will be issued at the conclusion of  
all 8 sessions.



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## 8-Session Registrants

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### FINAL EXAM

The final exam will be issued on Tuesday, April 11.

The final exam must be submitted by Monday, April 24 at 8:00 AM EDT.



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## 8-Session Registrants

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### QUIZZES

Access to the quiz: Information for accessing the quiz will be emailed to you by Wednesday. It will contain a link to access the quiz. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG

Quiz and Attendance records: Posted Tuesday mornings. [www.aisc.org/nightschool](http://www.aisc.org/nightschool) - scroll down to Quiz and Attendance Records.

Reasons for quiz:

EEU – must take all quizzes and final to receive EEU

PDHS – If you watch a recorded session you must take quiz for PDHs.

REINFORCEMENT – Reinforce what you learned tonight. Get more out of the course.

NOTE: If you attend the live presentation, you do not have to take the quizzes to receive PDHs.



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## 8-Session Registrants

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### RECORDINGS

Access to the recording: Information for accessing the recording will be emailed to you by this Wednesday. The recording will be available for two weeks. For 8-session registrants only. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

PDHS – If you watch a recorded session you must take AND PASS the quiz for PDHs.



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## Night School Resources for 8-session package Registrants

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Find all your handouts, quizzes and quiz scores, recording access, and attendance information all in one place!



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# Night School Resources for 8-session package Registrants



## Night School Resources

Event	Date
<a href="#">NS 13 8-Session Package</a>	1/30/2017 7:00:00 PM



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# Night School Resources for 8-session package Registrants



## Night School 13: Design of Industrial Buildings

### 8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
NS13 - Design Criteria	1/30/2017 7:00:00 PM	<a href="#">Handouts</a>	<a href="#">View</a> Passcode: NS13DSN	Pass Score: 80	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	<a href="#">Handouts</a>	Available 02/08/2017 5pm EST	Available 02/08/2017 5pm EST	Pending
NS13 - Lateral Load Systems and Details	2/13/2017 7:00:00 PM	<a href="#">Handouts</a>	Available 02/15/2017 5pm EST	Available 02/15/2017 5pm EST	Pending
NS13 - Preliminary Design Procedures	2/27/2017 7:00:00 PM	<a href="#">Handouts</a>	Available 03/01/2017 5pm EST	Available 03/01/2017 5pm EST	Pending
NS13 - Crane Girder Design and Frame Analysis	3/6/2017 7:00:00 PM	<a href="#">Handouts</a>	Available 03/08/2017 5pm EST	Available 03/08/2017 5pm EST	Pending
NS13 - Frame Member and Connection Design	3/13/2017 7:00:00 PM	<a href="#">Handouts</a>	Available 03/15/2017 5pm EST	Available 03/15/2017 5pm EST	Pending
NS13 - Transfer Crane Girder & Longitudinal Bldg Bracing Dsn	3/27/2017 7:00:00 PM	<a href="#">Handouts</a>	Available 03/29/2017 5pm EST	Available 03/29/2017 5pm EST	Pending
NS13 - Building Envelope and Bracing Design	4/3/2017 7:00:00 PM	<a href="#">Handouts</a>	Available 04/05/2017 5pm EST	Available 04/05/2017 5pm EST	Pending
NS13 - Final Exam	4/10/2017 7:00:00 PM			Available 04/12/2017 5pm EST	



## **Night School Resources for 8-session package Registrants**

- Weekly “quiz and recording” email.
- Weekly updates of the master Quiz and Attendance record found at [www.aisc.org/nightschool](http://www.aisc.org/nightschool). Scroll down to Quiz and Attendance records.
  - Updated on Tuesday mornings.



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## **Night School Resources for 8-session package Registrants**

- Webinar connection information:
  - Found in your registration confirmation/receipt.
  - Reminder email sent out Monday mornings.
- Link to handouts also found here.



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# Thank You

Please give us your feedback!  
*Survey at conclusion of webinar.*

