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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.

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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).

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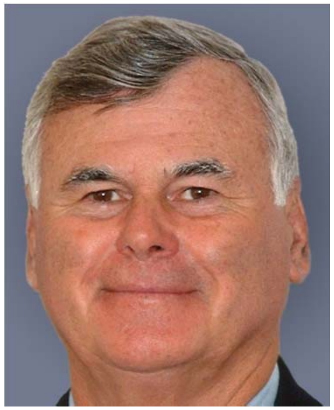
Design of Industrial Buildings

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



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

structural STEEL

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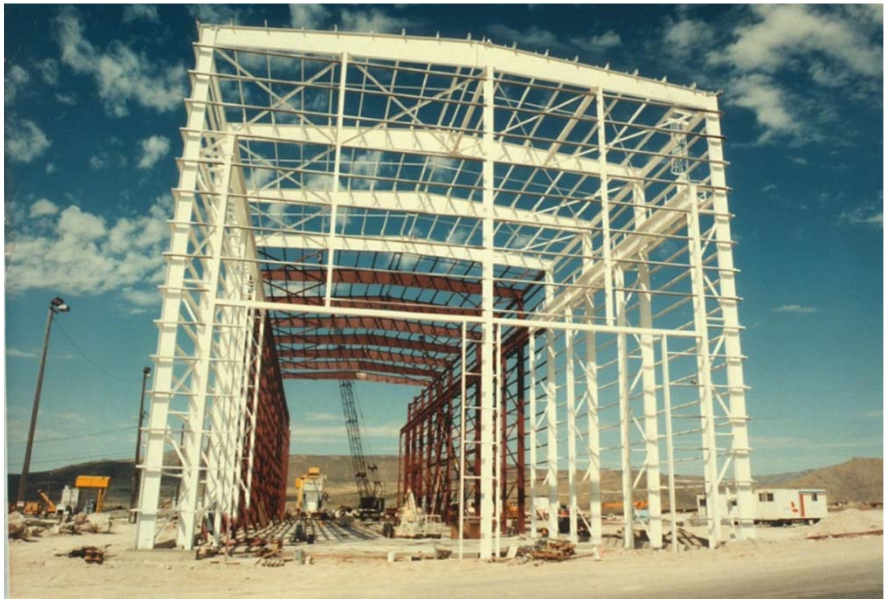
Design of Industrial Buildings
Lesson 8



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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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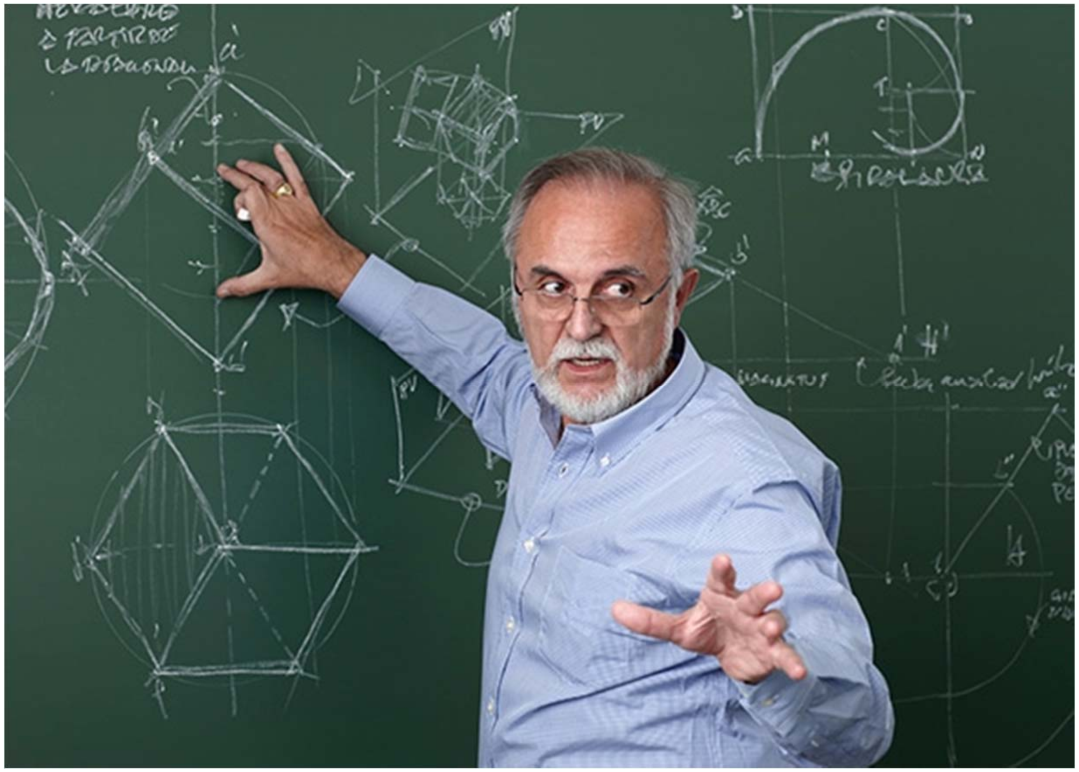
Review of Design Criteria



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Project Description

- 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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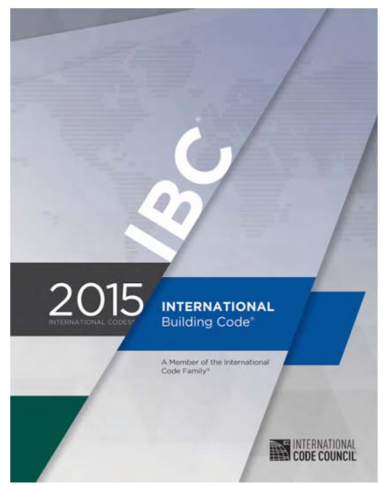
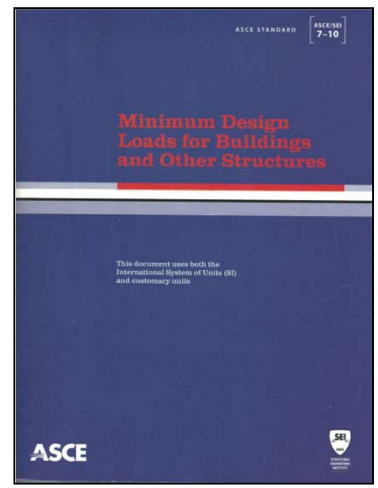
Codes and Standards

- 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

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

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Loads

- 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 (Includes Girts) **3.0 psf**

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Loads

- 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: $P_f = 0.7(P_g)(I)(C_e) = 10.5$ psf < Use 20 psf
  - Building Category: II

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Loads

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

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Loads

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

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

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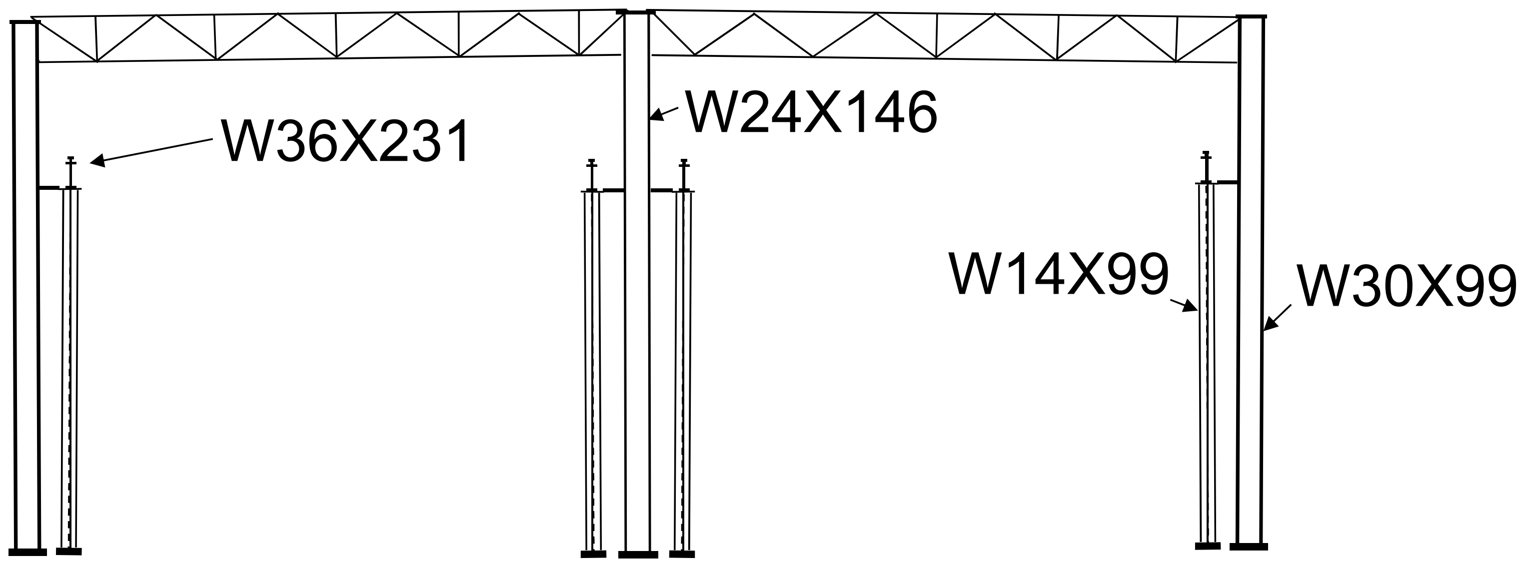
Final Frame Design



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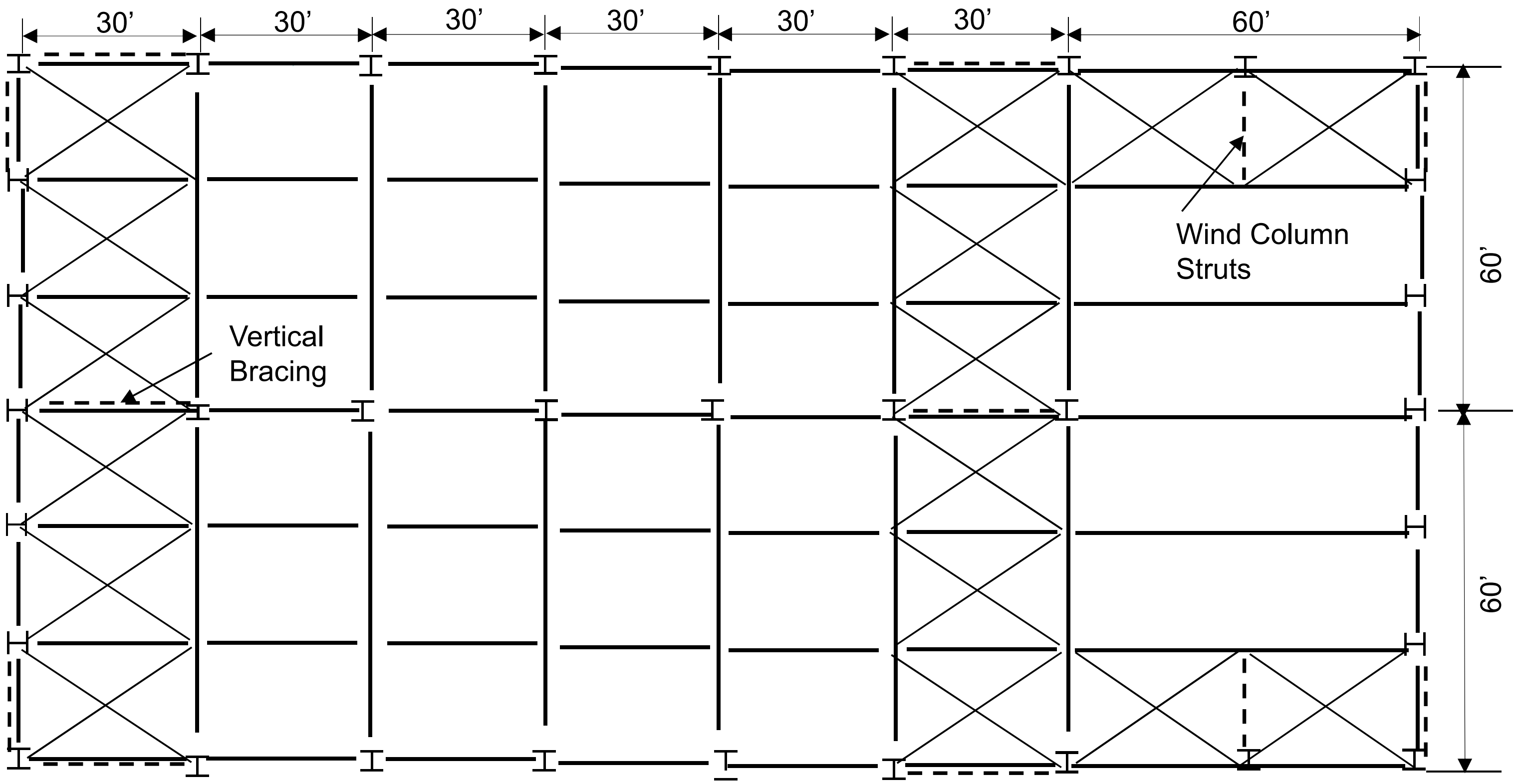
Roof Horizontal Bracing



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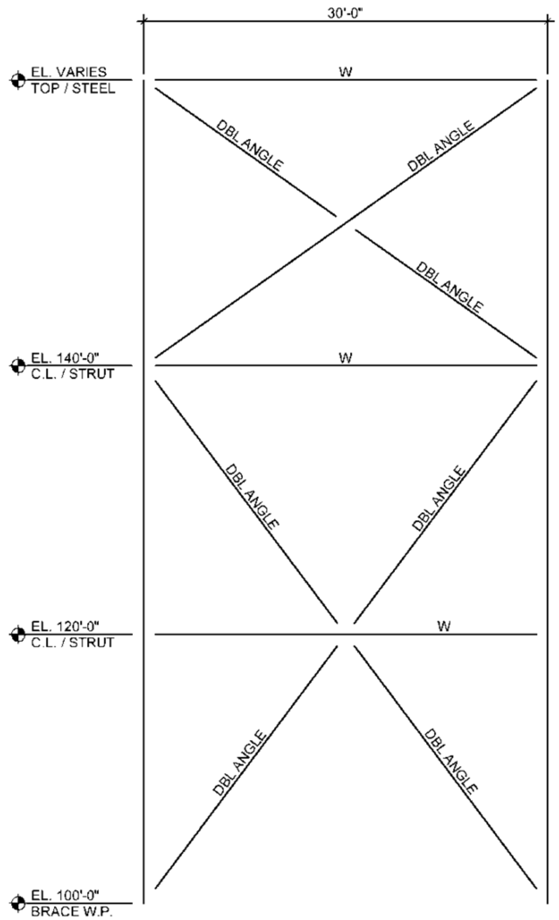
Sidewall Vertical Bracing



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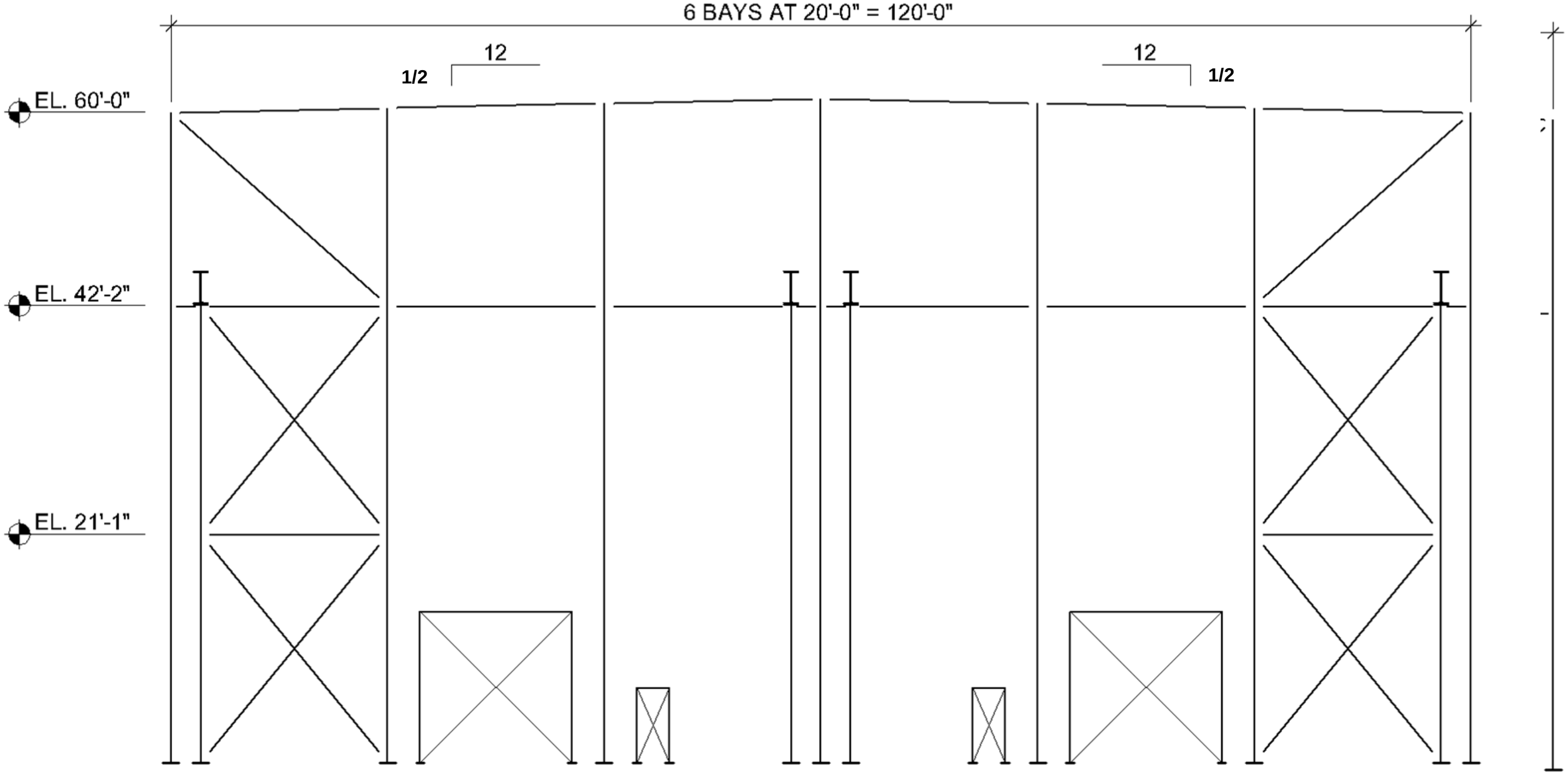
End Wall Vertical Bracing



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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

- 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

- When first introduced, membrane sheet widths in seam-fastened single-ply membrane roof systems typically were **five-feet-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

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

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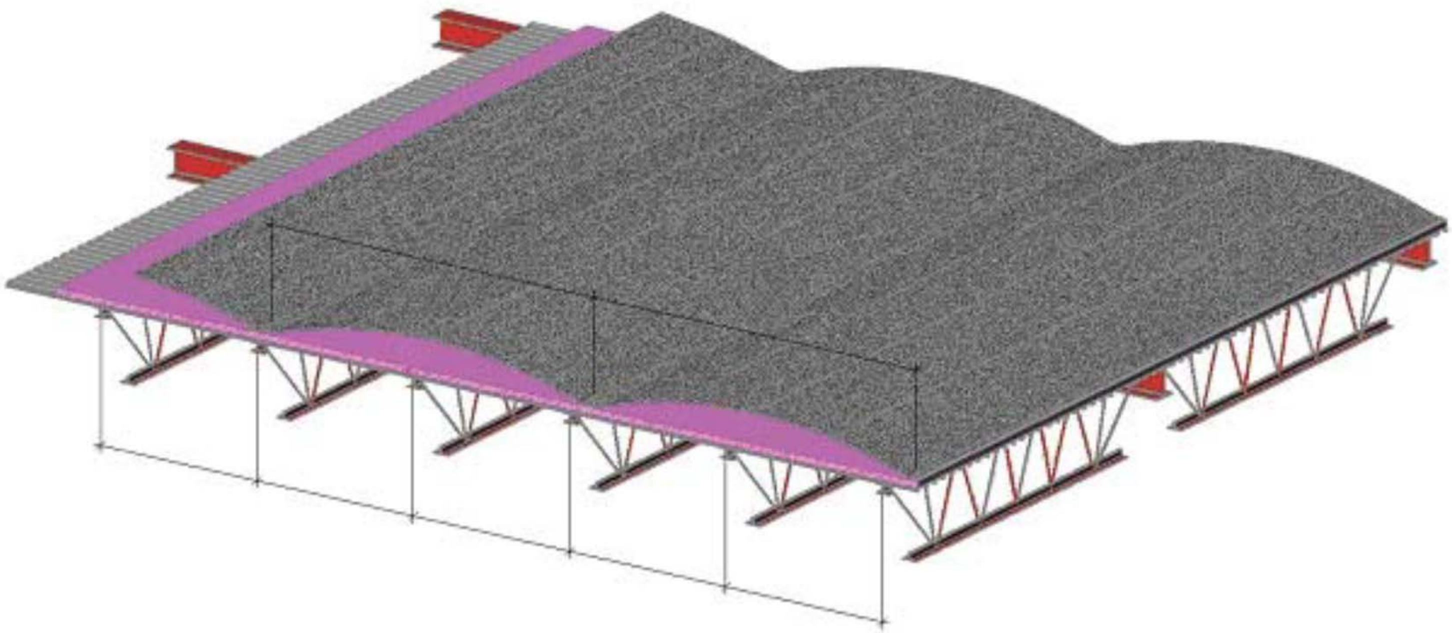
Deck and Joist Loading



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The Problem

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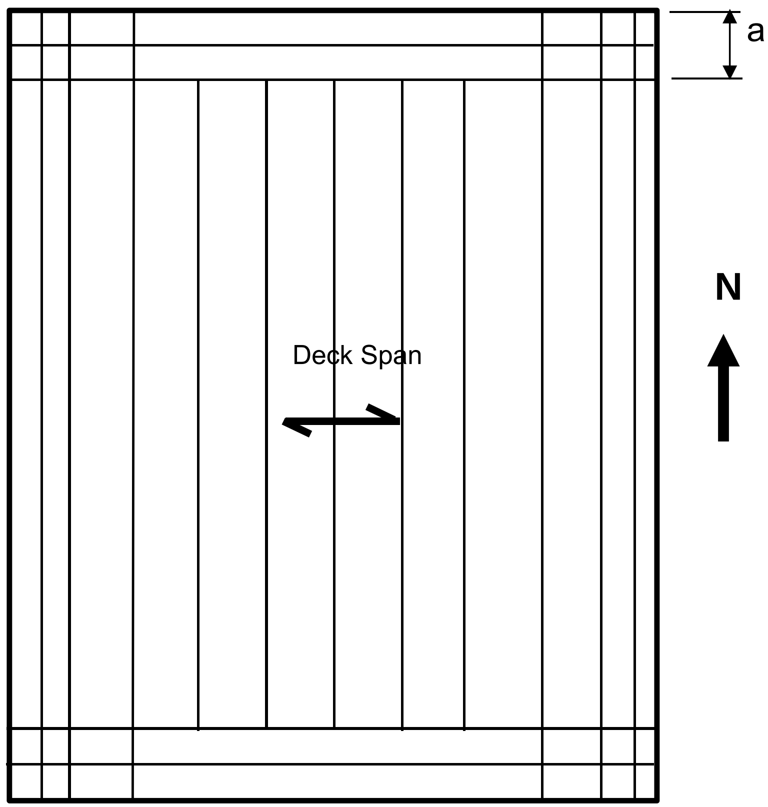
Typical Layout



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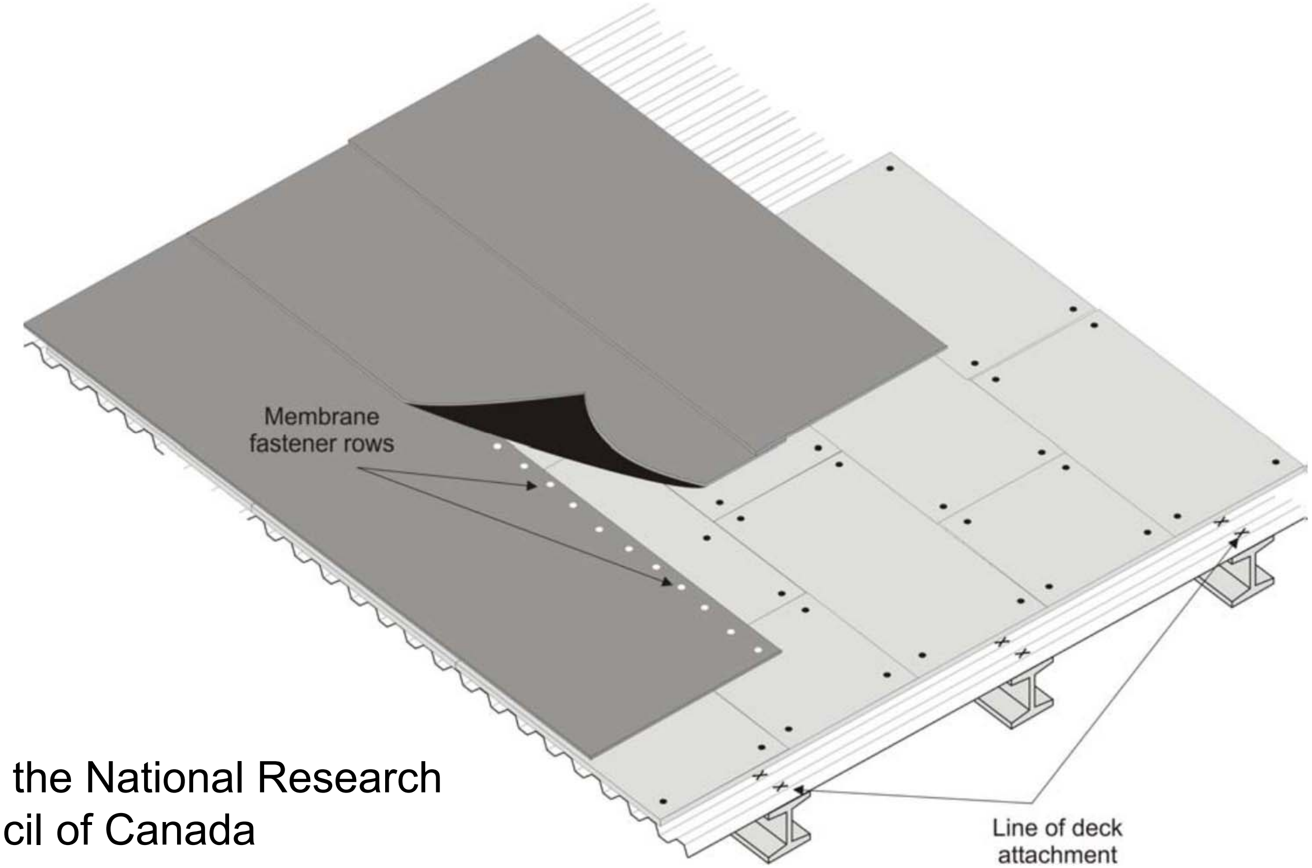
Fasteners Perpendicular to Deck Flutes



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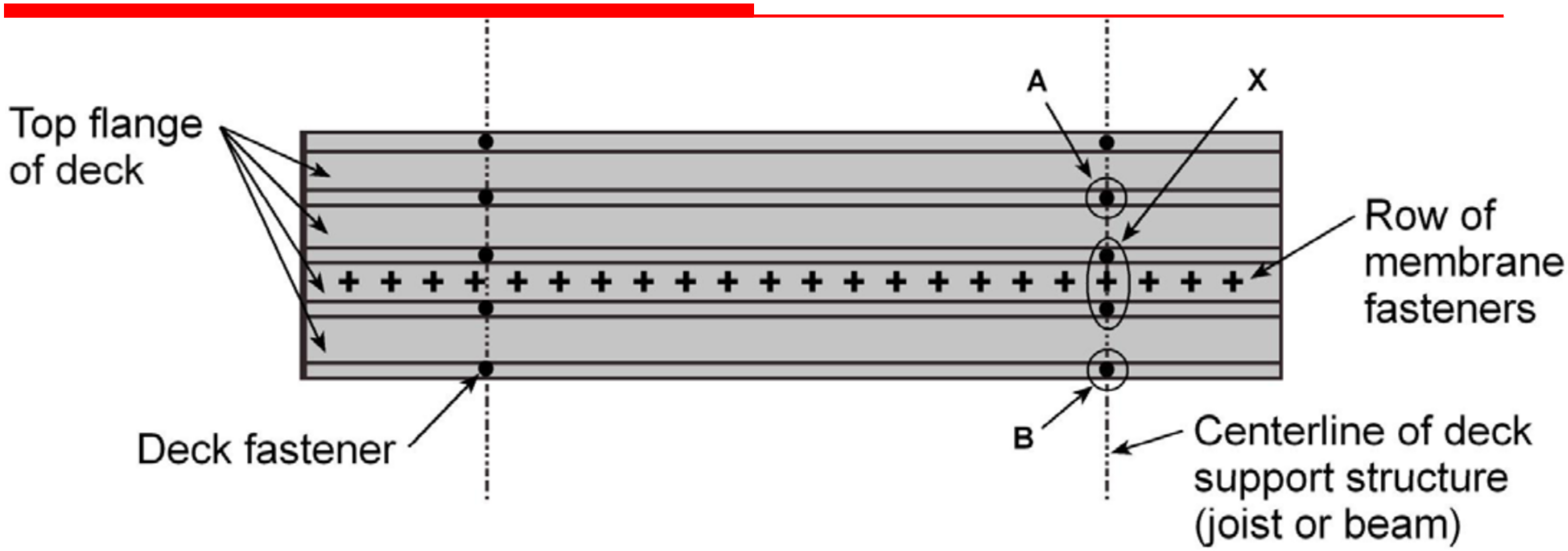
Fasteners Parallel to Deck Flutes



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

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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[1]	0.95	1.24[2]	0.23
East-West Perimeter	4.32[1]	0.81	2.31[2]	0.43
North-South Perimeter	11.9[2]	**2.22**	2.22[2]	0.41
Corner	11.55[2]	**2.16**	3.48[2]	0.65

1. Positive moment controls, 2. Negative moment controls
ϕM_p = 5.088 kip-in., ϕM_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)(6psf) - (0.6)(33.3psf)](6ft) = 98 plf

Concentrated Net Uplift: (0.6)(6psf)(6ft.) - (0.6)(33.3psf)(12ft) = 218 plf

Joist Weight (Normal Case):

Uniform Case Gravity Load = (6psf)(6ft) + (20psf)(6ft) = 156 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

Are your roof members overstressed?

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

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Roof Design

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

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Standing Seam Roofs

- Spans: 5 ft
- Slope: ¼ in./ft minimum
  - ½ in./ft used
- Diaphragm: Limited
- Anchorage: One end
- Snow drift areas: Add secondary members

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Standing Seam Roofs

- **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

- **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

- Joists
- Girts
- Rake Beams
- Wind Columns

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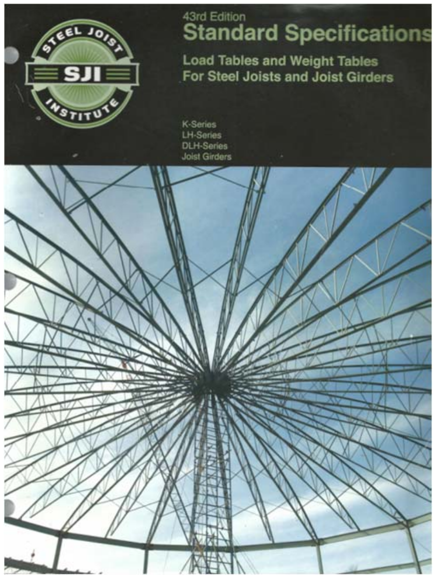
Joist Design

- Gravity Loads
- Ponding
- Seat Depths
- Wind Uplift



Free download from steeljoist.org

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Joist Design- Gravity Loads

30 ft joists
Dead Load = 10 psf, Live Load = 20 psf, w = (10 psf + 20 psf)(5 ft) = 150 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

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

L = 30 ft -0.33 ft = 29.67 ft

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

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

Roof slope = (0.50in./ft)(5ft) = 2.50 in.

1.21 in. ≤ 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:

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

w = (10 psf + 20 psf)(30 ft)/1000 = 0.90 kips/ft, x = 5 ft, L_g = 60 ft

From Lesson 4: I_g = 13,167 in^4

$$\Delta = \frac{wx}{24EI}\left(L_g^3 - 2L_g x^2 + x^3\right)$$

$$= \frac{(0.90)(5)(1728)}{(24)(29000)(13{,}167)}\left[(60)^3 - 2(60)(5)^2 + (5)^3\right] = 0.18 \text{ in.}$$

Total joist deflection = 1.21 in. + 0.18 in. = 1.39 in. < 2.50 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}) in.^{4}

L = 60 ft -0.33 ft = 59.67 ft

I = (26.767)(109 plf)(59.67 ft)^3(10^{-6}) = 620 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 Δ, w = (30 psf)(45 ft) / 1000 = 1.35 kips/ft

Δ = (0.18)(1.35/0.9) = 0.27 in.

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

Solving for w_{LL} = 134 plf

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

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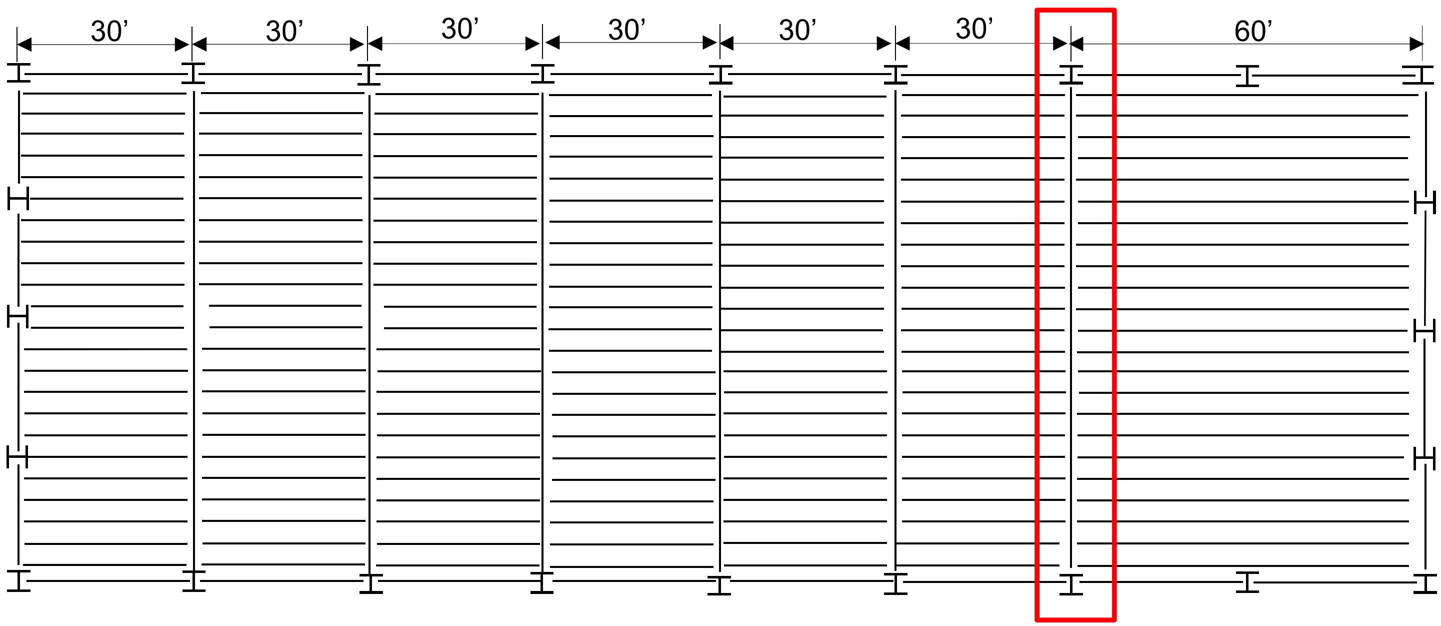
Roof Plan-Seat Depths



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 & θ ≤ 7˚)								
	Tributary Area, ft.^2							
Location	**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	**-35.3**	-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 ft^2

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Girder Roof Suctions

Using Spreadsheet:

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

			WW	LW	Total	Side	Roof WL			Int.
Height	Kz	q	WL	WL	WL	WL	0 to h	h to 2h	> 2h	WL
0-15	0.85	24.43	16.6							
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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Joist Calculations

Uplift requirements (Gross)

Joists = (0.6)(35.3 psf) = 21.2 psf

Girders = (0.6)(25 psf + 5.9 psf) = 18.5 psf

Uplift requirements (Net)

Joists = 21.2 psf – (0.6)(7 psf) = 17.0 psf

Girders = 18.5 psf – (0.6)(10 psf) = 12.5 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
Total	**19.0 psf**

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Girt Design

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

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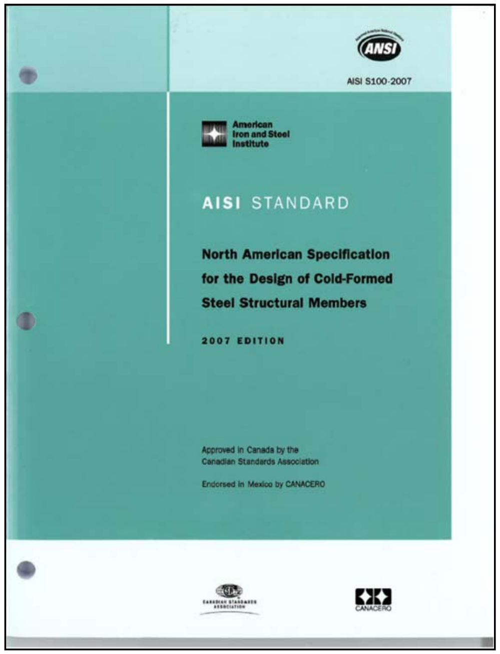

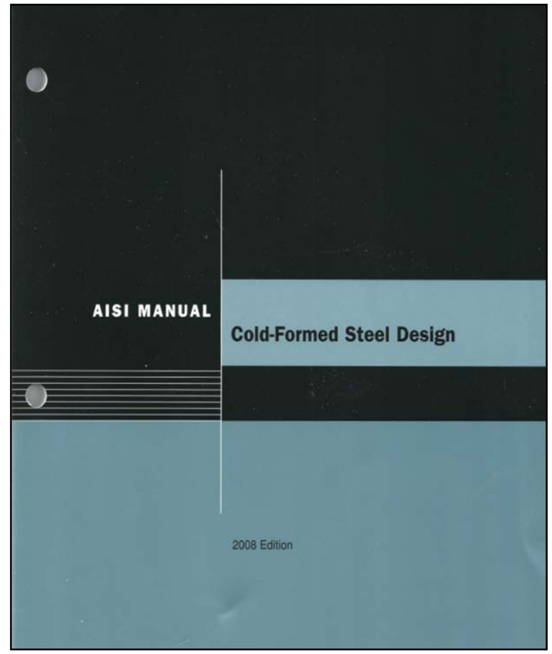

1. Download all the AISI new standards from the link (www.AISIStandards.org)
2. Order the AISI Cold-Formed Steel Design Manual from the AISI online store or via the link (https://shop.steel.org/p/312/cold-formed-steel-design-manual-2013-edition-electronic-version-includes-aisi-s1-12-specification-and-commentary).

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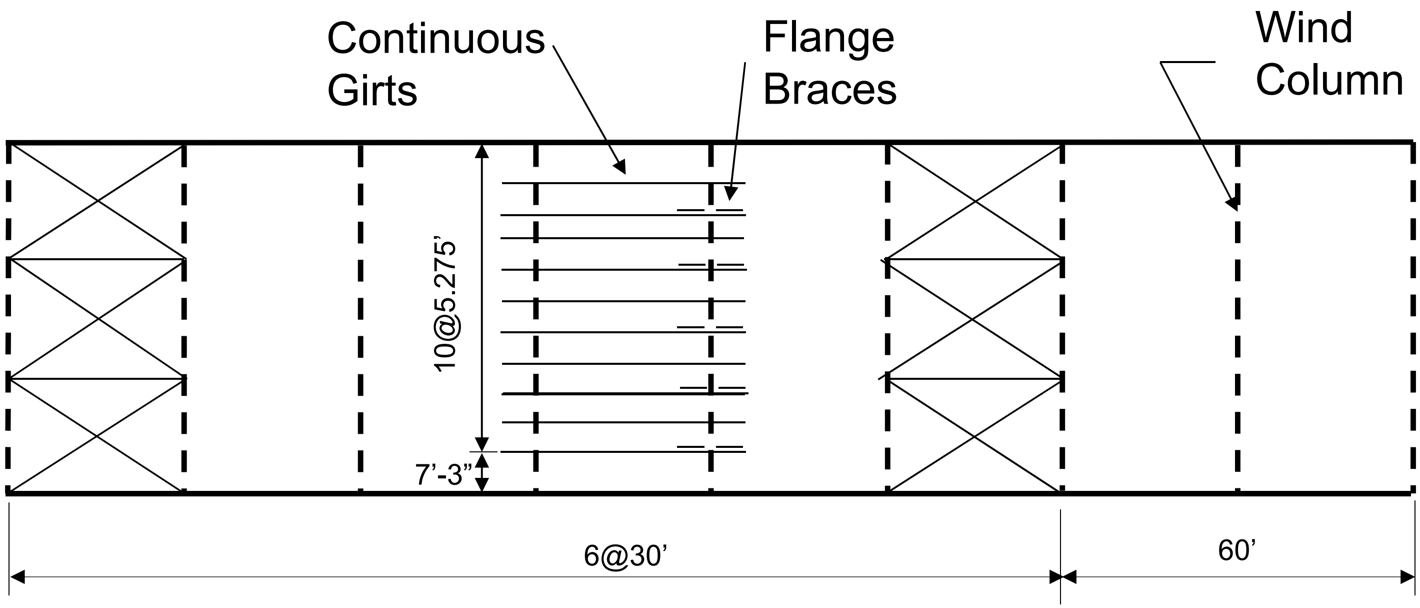
Girt Design- Sidewalls



Sidewall Elevation

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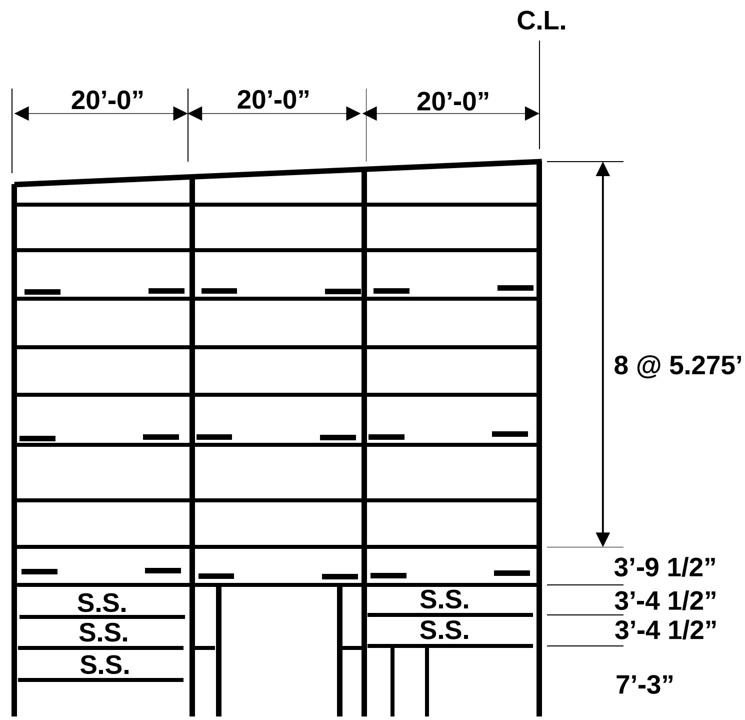
Girt Design- Endwalls



Endwall

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Girt Design

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

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Girt Design- AISI Specification

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

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

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Girt Design- Windloads

Sidewall values:

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

Endwall values:

- Tributary Area (Typical girt) = (5.275)(20 ft) = 106 sq. ft
- Need not be < (1/3)(20)(20) = 133 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.^2							
	133		10					
	GCp	P	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: Ω = 1.67, R = 0.5 for s.s. girt

Endwall: w = (5.275)(0.6)(35.4 psf) = 112 plf = 0.112 klf

M = (1/8)wL^2 = (1/8)(0.112 plf)(20)^2 = 5.60 kip-ft

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

Specify: 10Z3.25x105 S_x = 5.69 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

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

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

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

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

L = 20 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*Kzt*Kd*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
Encl. =	**E**	enclosure classification, Section 26.10
Ri=	**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
α =	9.5	3-sec gust speed power law exponent, Table 26.9-1

Pressure Coefficients per Table 26.11-1 & 27.4-1

			Wind Pressures, psf							
			WW WL	LW WL	Total WL	Side WL	Roof WL 0 to h	Roof WL h to 2h	Roof WL > 2h	Internal WL
Height	Kz	qz	**0.8**	**0.5**		**-0.7**	**-0.9**	**-0.5**	**-0.3**	**0.18**
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

Sidewall Wind Columns:

Use p = (0.6)(22.2 psf + 5.9 psf) = 16.9 psf

Pressure: w = (16.9 psf)(30 ft)/1000 =0.516 k/ft (Use for suction also)

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

Lateral bracing at ≈ 10 ft centers, Try W18x55, I_x = 890 in.^{4}

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

Serviceability (10 year wind), H/120 = (60 ft)(12)/120 = 6.0 in. > 5.83 in.

5.83 in. was for a 50 year wind.

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

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Wind Columns

Endwall: Consider MWFRS

p = (16.9 psf)(20) = 338 plf = 0.338 klf

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

Use a W21x44, I_x = 843 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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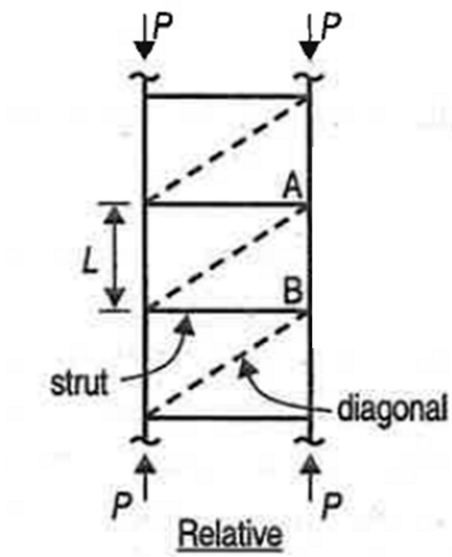
Relative Brace

- 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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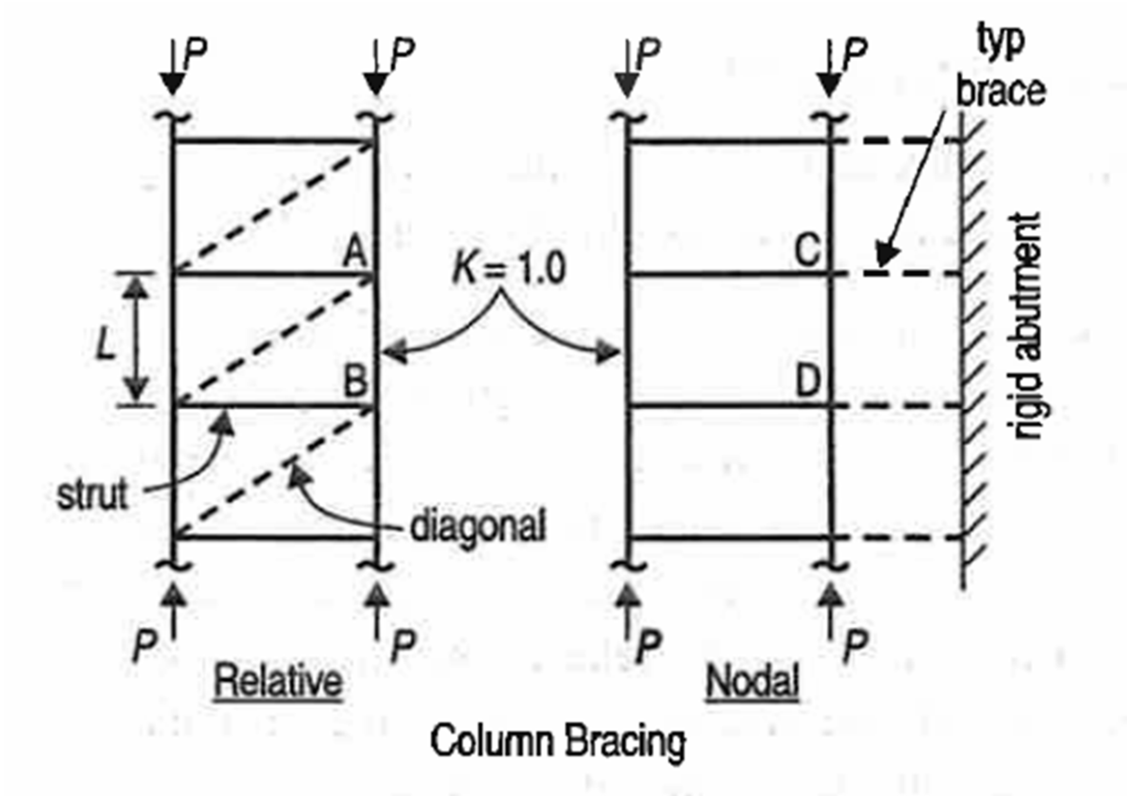
Relative and Nodal Braces



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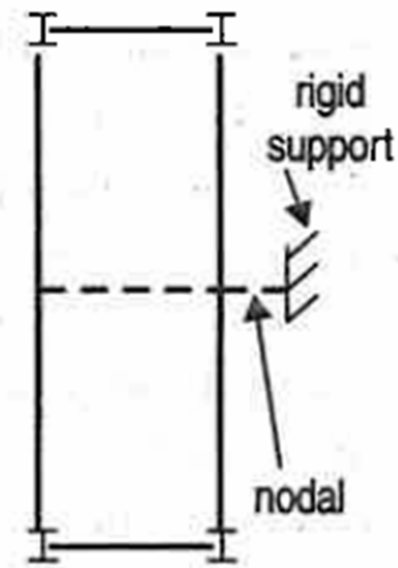
Nodal Brace

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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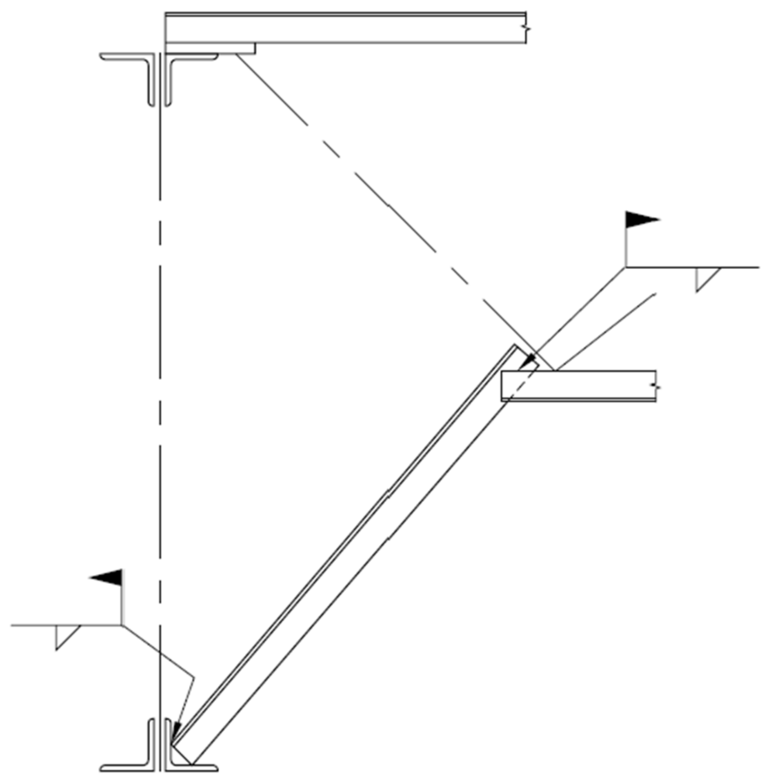
Girder Bracing

Typical girder bracing for stability and wind uplift



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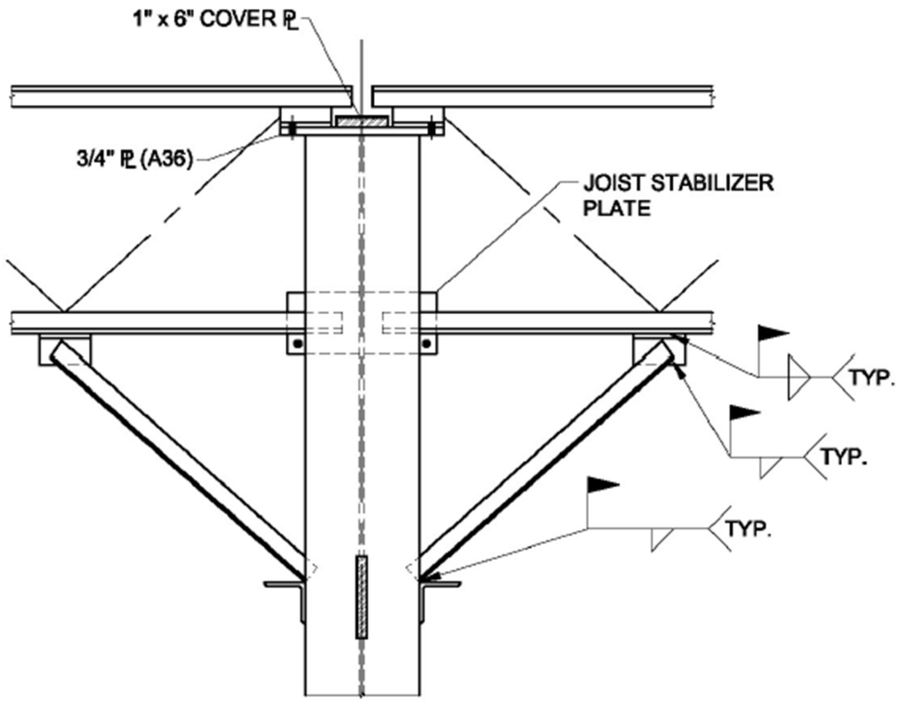
Girder Bracing

Typical girder bracing at plastic hinge locations



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Bracing of Sidewall Columns

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

Relative bracing equations are appropriate.

Strength: $P_{br} = 0.008M_rC_d/h_o$

Stiffness: $$\beta_{br} = \Omega\frac{4M_rC_d}{L_bh_o}(ASD) = \frac{4M_rC_d}{\phi L_bh_o}(LRFD)$$

$\Omega = 2.0$, $\phi = 0.75$

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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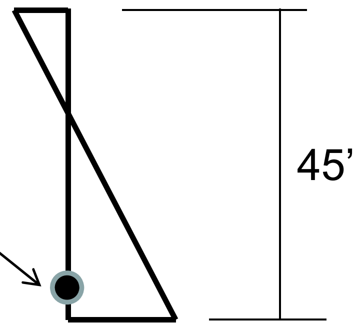
Bracing of Sidewall Columns

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.

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

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

Typical R panel shear value = 140 lbs/ft ok

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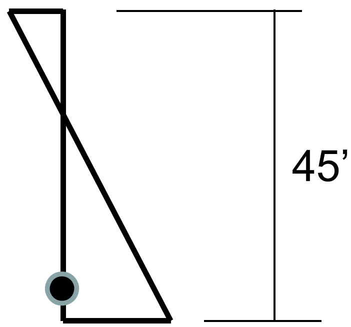
Bracing of Sidewall Columns

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 in.^{2}

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

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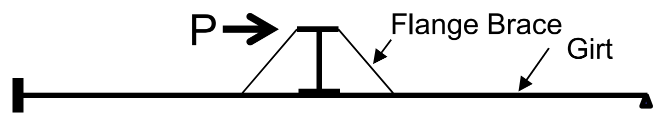
Bracing of Sidewall Columns

From analysis:

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



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

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

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

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Bracing of Sidewall Columns

Determine Flange Brace Angle Strength:

Length, $L = (30\text{ in.})(1.414) = 42.4\text{ in.}$ (column depth = 30 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: $F_{cr}/\Omega = 9.59$ ksi

$P_a = (9.59)(0.722) = 6.92$ kips > $(1.414)(0.78) = 1.10$ kips ok

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The End

End of Lesson 8 and Night School 13

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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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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 - 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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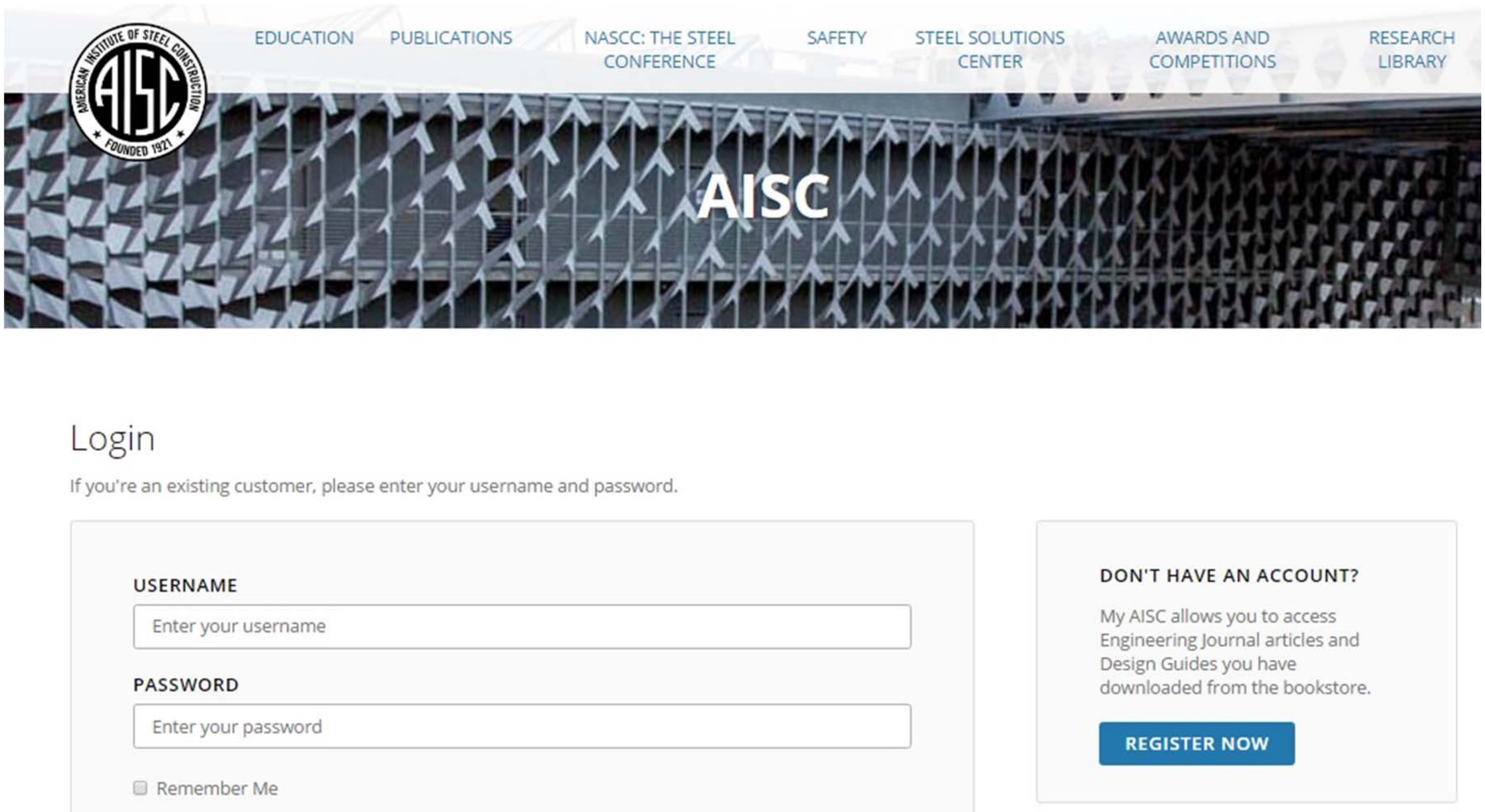

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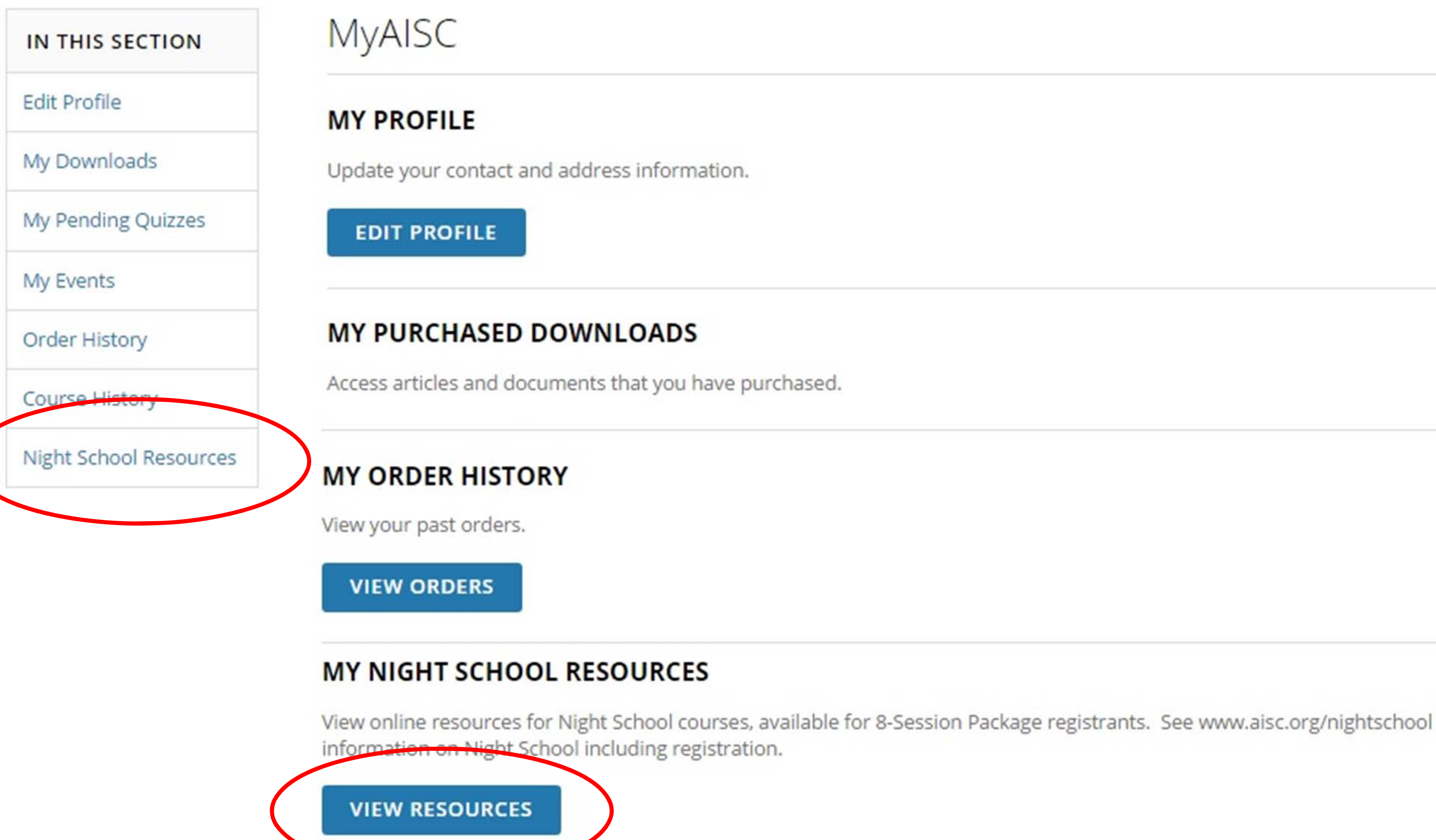

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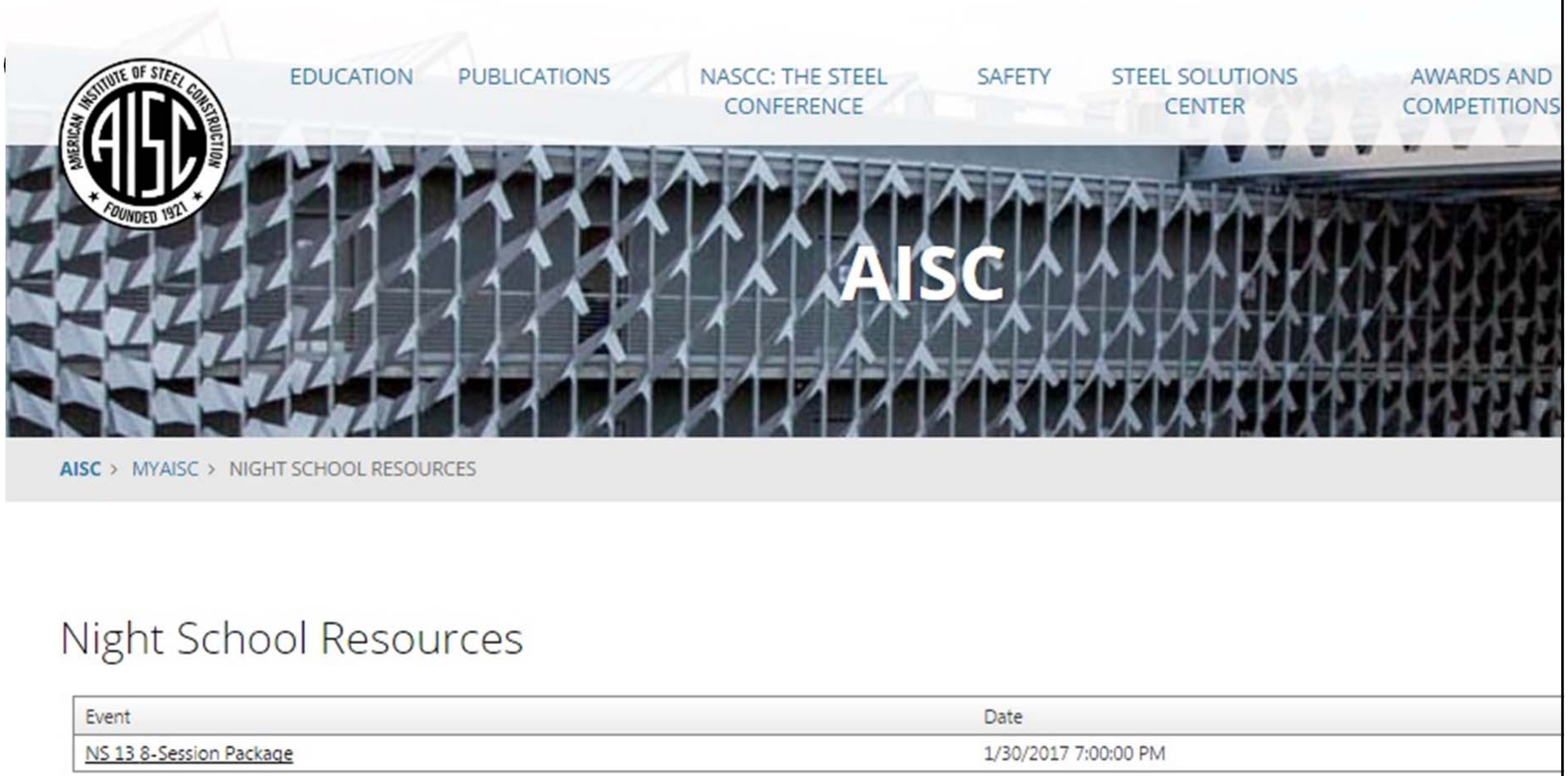

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AISC > MYAISC > NIGHT SCHOOL RESOURCES > NS13 8-SESSION PACKAGE RESOURCES

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	Handouts	View Passcode: NS13DSN	Pass Score: 80	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	Handouts	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	Handouts	Available 02/15/2017 5pm EST	Available 02/15/2017 5pm EST	Pending
NS13 - Preliminary Design Procedures	2/27/2017 7:00:00 PM	Handouts	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	Handouts	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	Handouts	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	Handouts	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	Handouts	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	

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

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Thank You

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Survey at conclusion of webinar.

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