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Course Description

Session 3: Lateral Load Systems and Details
February 13, 2017

Lesson 3 deals with lateral load resisting systems which includes, roof horizontal bracing systems, braced frames, and rigid frames. Economical choices for the lateral load system are discussed. Moment connections details for connecting Joist Girders to columns are provided along with practical suggestions for the maximum moment resistance for each detail. Three examples are given, these include a braced frame using LRFD and ASD, and a rigid frame using ASD. The braced frame examples include the design of the roof diaphragm. The rigid frame example includes a "hand calculation" of a moment connection as well as a demonstration of a Spreadsheet solution.

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Learning Objectives

- Explain the advantages and disadvantages of various lateral load systems.
- Describe girder to column moment connection details and how to maximize moment resistance in each.
- Determine (calculate) the forces that must be carried by the braced frame.
- List the steps for a designing a moment connection.

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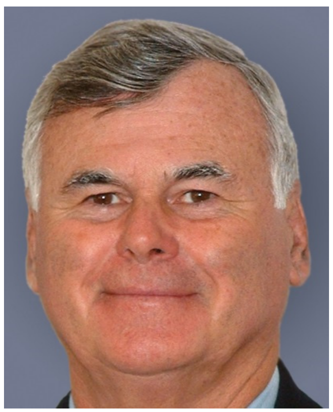
Design of Industrial Buildings

Session 3: Lateral Load Systems and Details

February 13, 2017



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

structural STEEL

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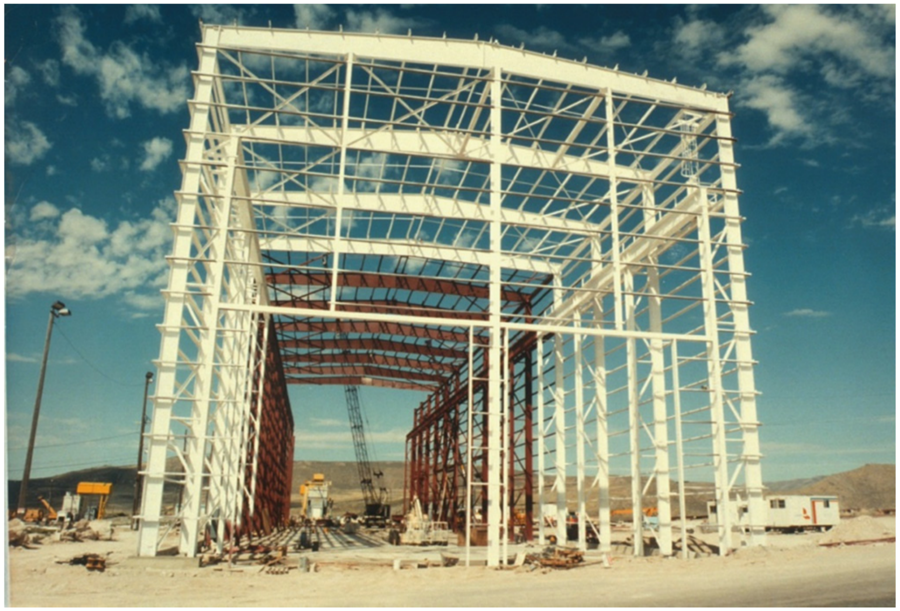
Design of Industrial Buildings
Lesson 3



Presenter:
Jim Fisher

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Buildings w/o Overhead Cranes

- Lesson 3
  - Horizontal Bracing
  - Braced Frames
  - Rigid Frames
  - Selection of the Lateral Load System
  - Examples

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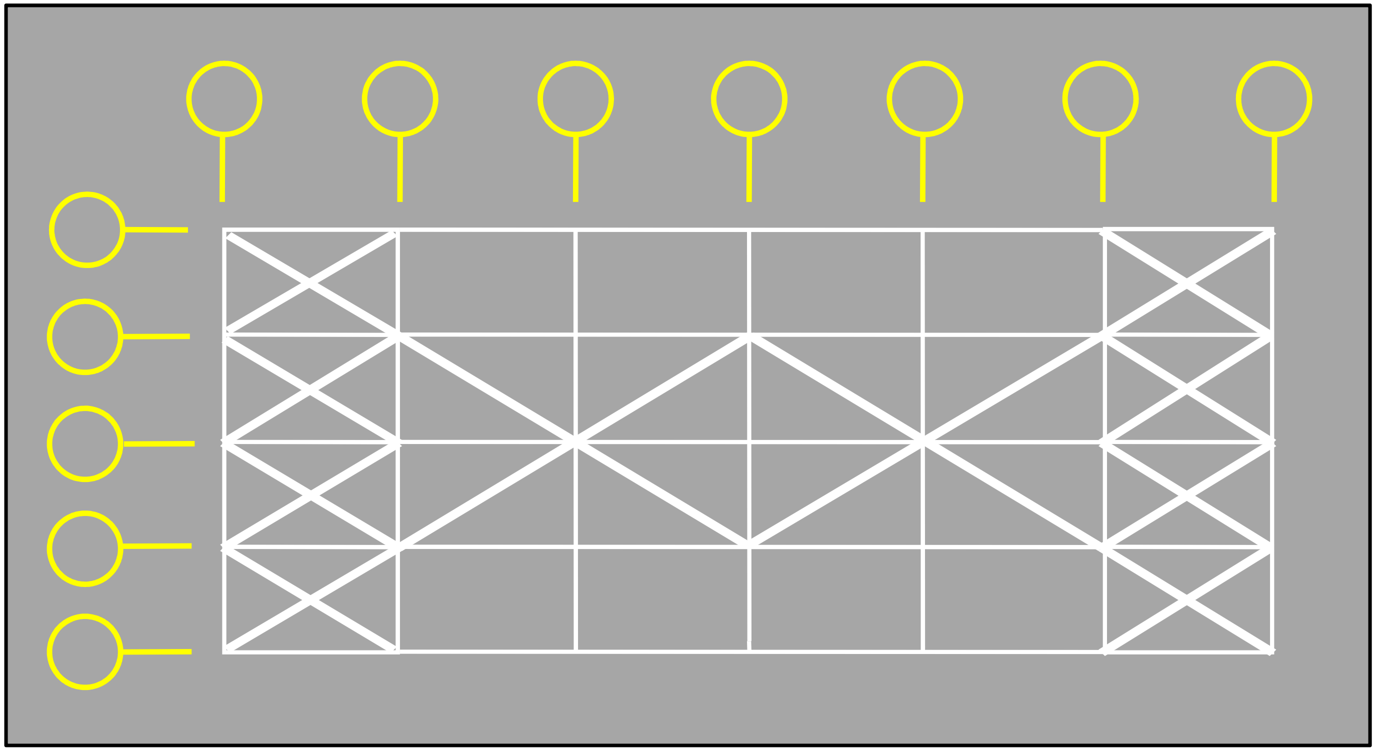
Horizontal Bracing



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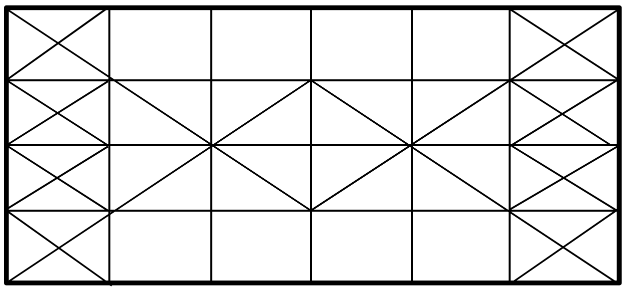
Horizontal Bracing

- Required in One or Two Directions
- Bracing Forms
- Analysis Assumptions:
  1. Divide the Lateral Loads Equally (based on stiffness)
  2. Design for Pressure or Suction
  3. Model the Full Roof System



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Horizontal Truss

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Rigid Frames

- Determine if Rigid Frames are Required
  - Use W Shapes, Trusses or Joist Girders ?
- Consider Connection Types
  - Field welded or bolted?
  - End Plates?

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Rigid Frames

- The specifying professional shall provide the nominal loads and load combinations as stipulated by the applicable code under which the structure is designed and shall provide the design basis (ASD or LRFD).
- Type and magnitude of end moments and/or axial forces at the joist and Joist Girder end supports shall be shown on the structural drawings.
- A note shall be provided on the structural drawings stating that all moment resisting joists shall have all dead loads applied to the joist before the bottom chord struts are welded to the supporting connection whenever the moments provided do not include dead load.
- The top and bottom chord moment connection details shall be designed by the specifying professional. The joist designer shall furnish the specifying professional with the joist detail information if requested.

SJI Code of Standard Practice 6.1(a)

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Rigid Frames

GIRDER SCHEDULE USING ASD							
Mark	Designation	Live Load Moments, ft.-kip (+ Sign Convention +)		Wind or Seismic Moments, ft.-kip (+ Sign Convention +)		Applied Chord Forces, kips (+ Sign Convention +)	
		Left	Right	Left	Right	Top Chord	Bottom Chord
G1	48 inch depth	50	150	± 100	∓100	± 50	± 50

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Rigid Frames

GIRDER SCHEDULE USING LRFD							
Mark	Designation	Live Load Moments, ft.-kip (+ Sign Convention +)		Wind or Seismic Moments, ft.-kip (+ Sign Convention +)		Applied Chord Forces, kips (+ Sign Convention +)	
		Left	Right	Left	Right	Top Chord	Bottom Chord
G1	48 inch depth	50	150	± 100	∓100	± 50	± 50

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Moment Connections

Joist Girders

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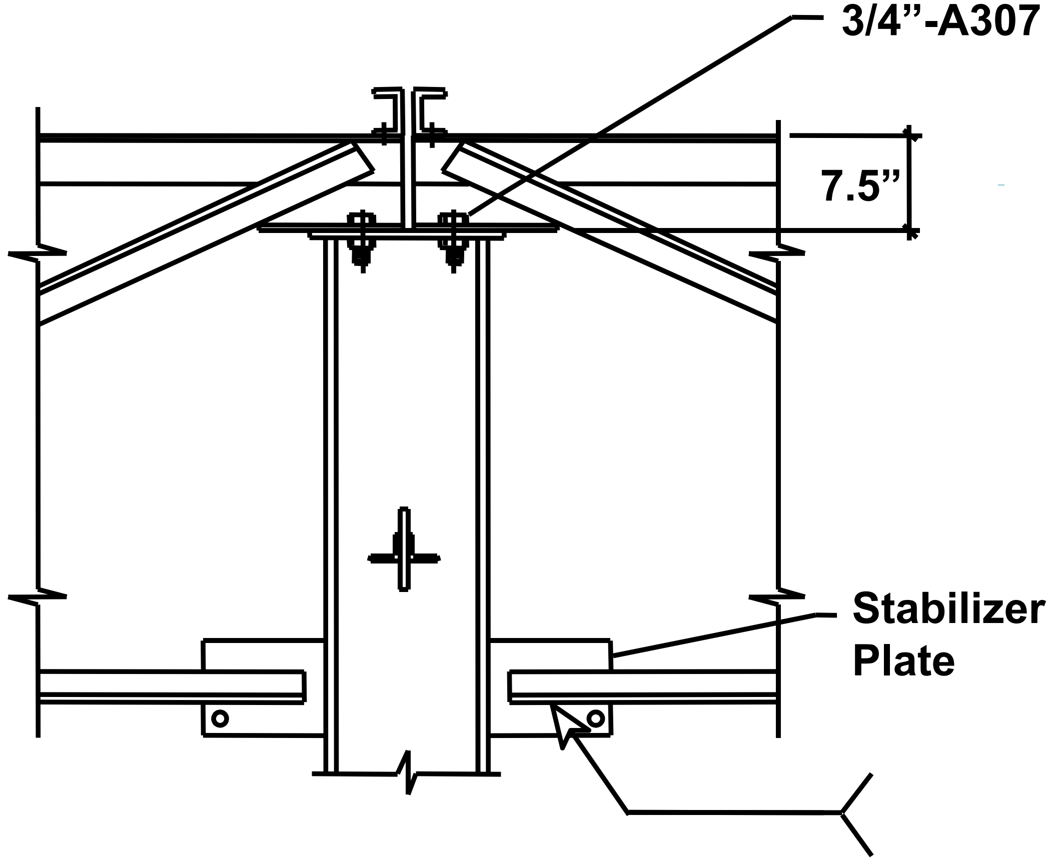
The Basic Connection



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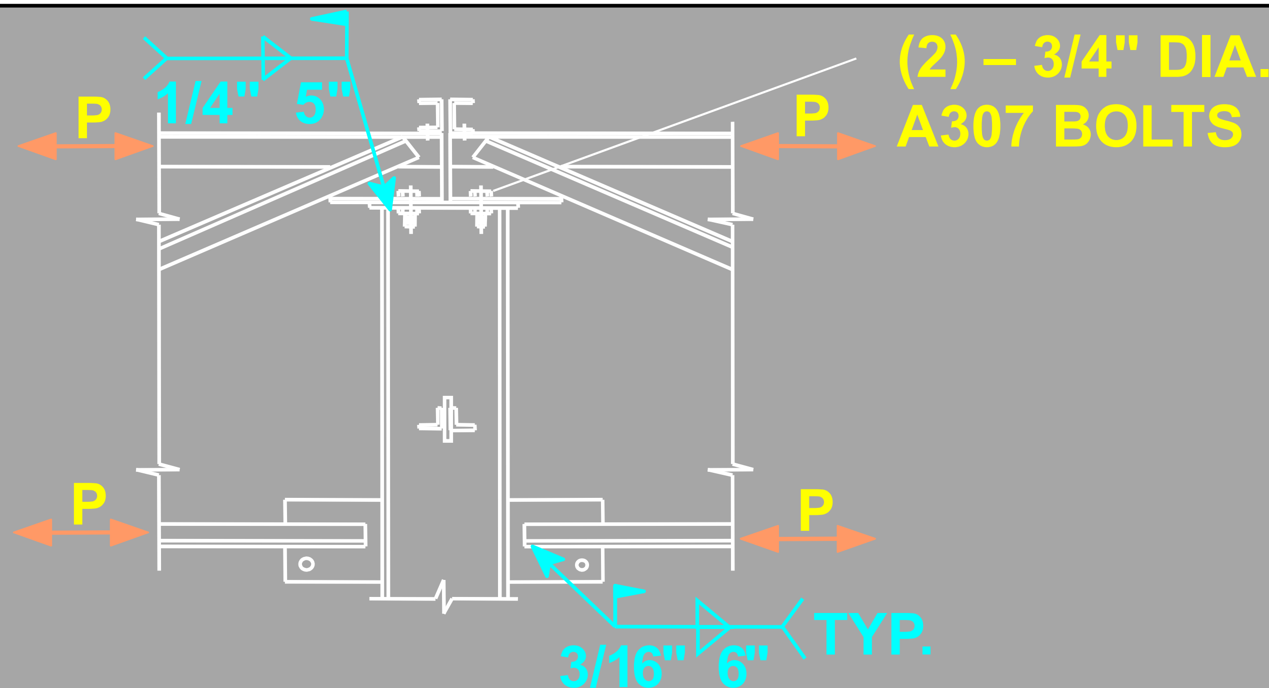
The Welded Basic Connection



Note: Bottom Chord to be Welded only after Dead Load is Applied

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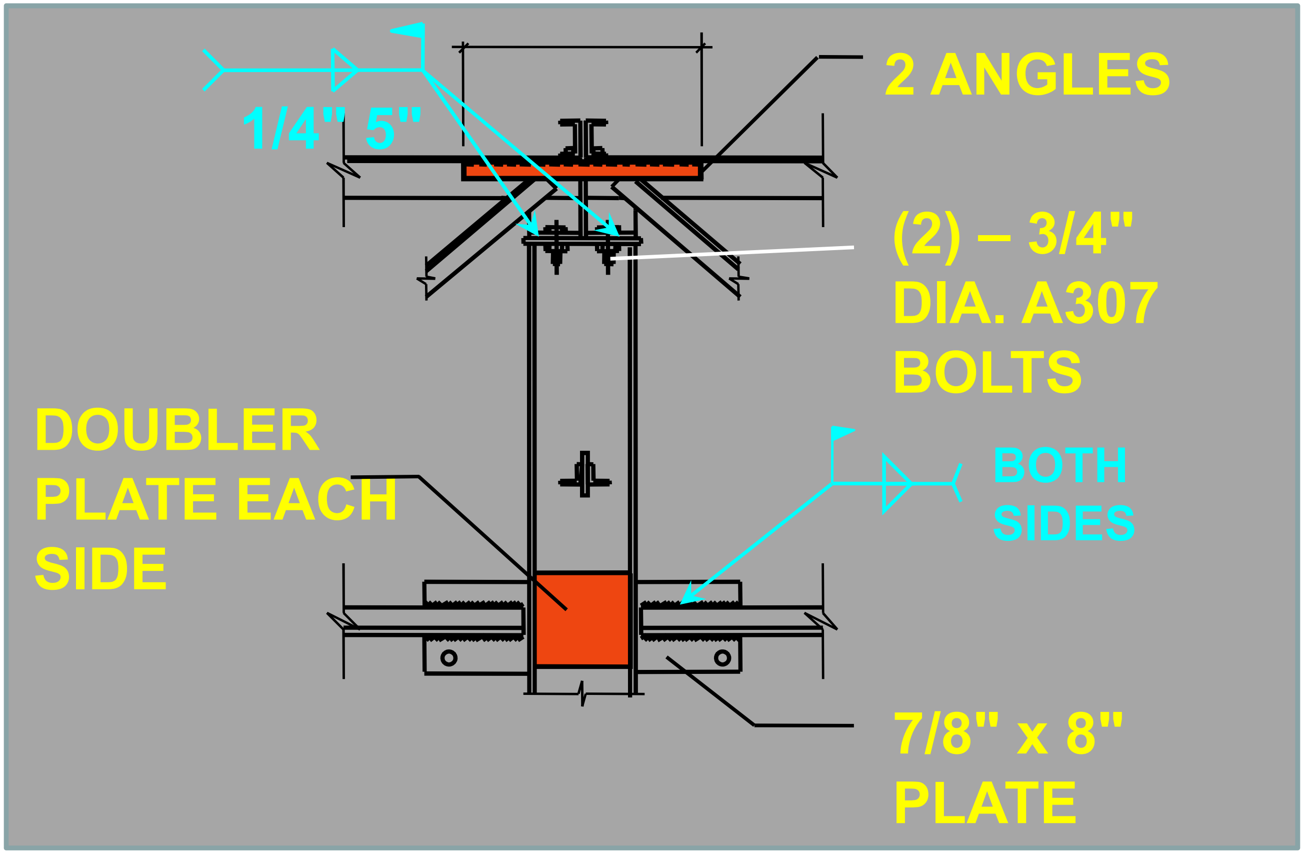
The Welded Basic Connection

Joist Girder (7.5″ seat) Top Chord Leg Size	ASD P_a	LRFD ϕP_n
2.5″	4^K	6^K
3.0″	8^K	12^K
3.5″and larger	10^K	15^K

The Welded Basic Connection withTies



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Top Chord Transfer Angles

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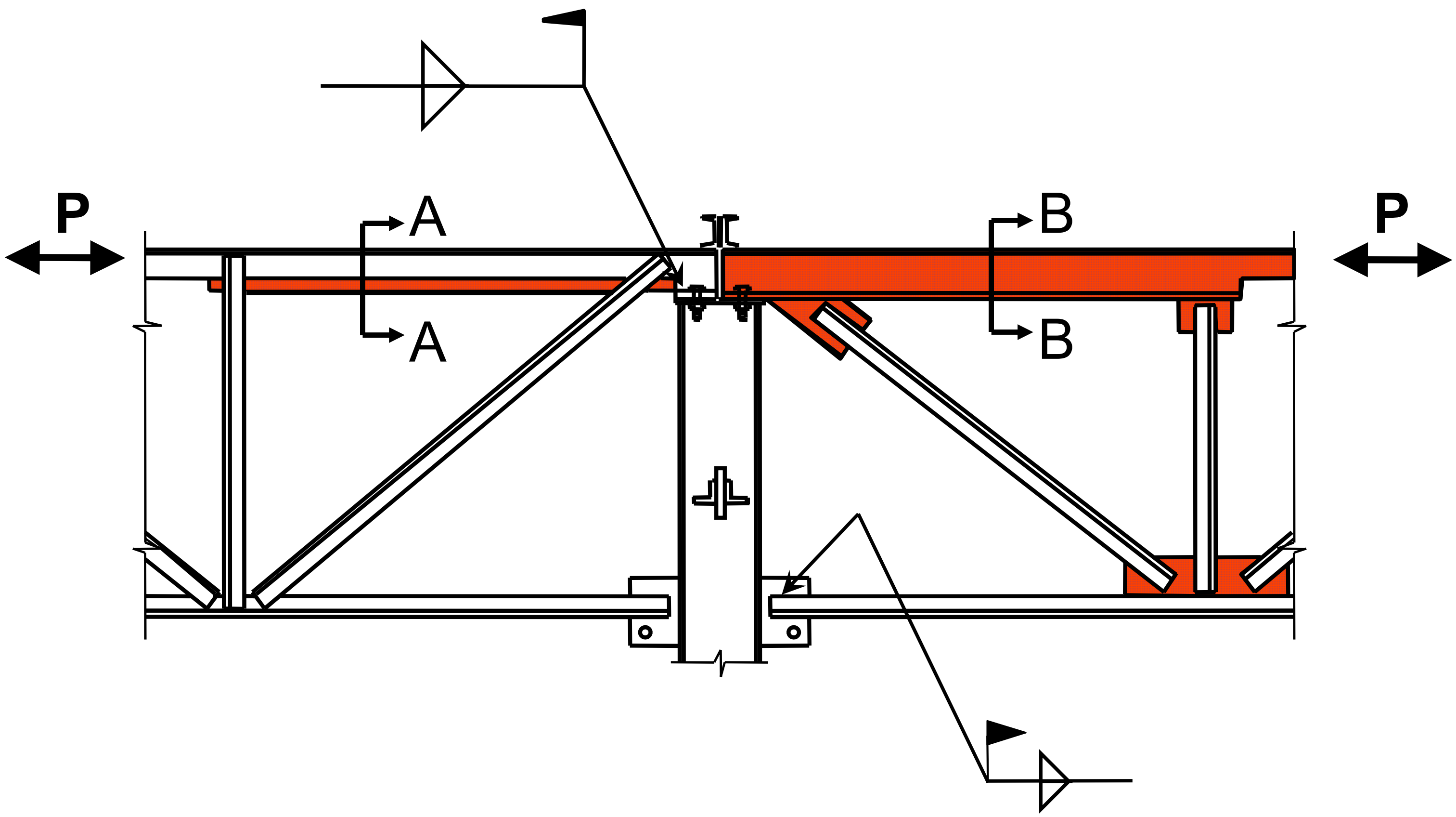
Reinforced Chords



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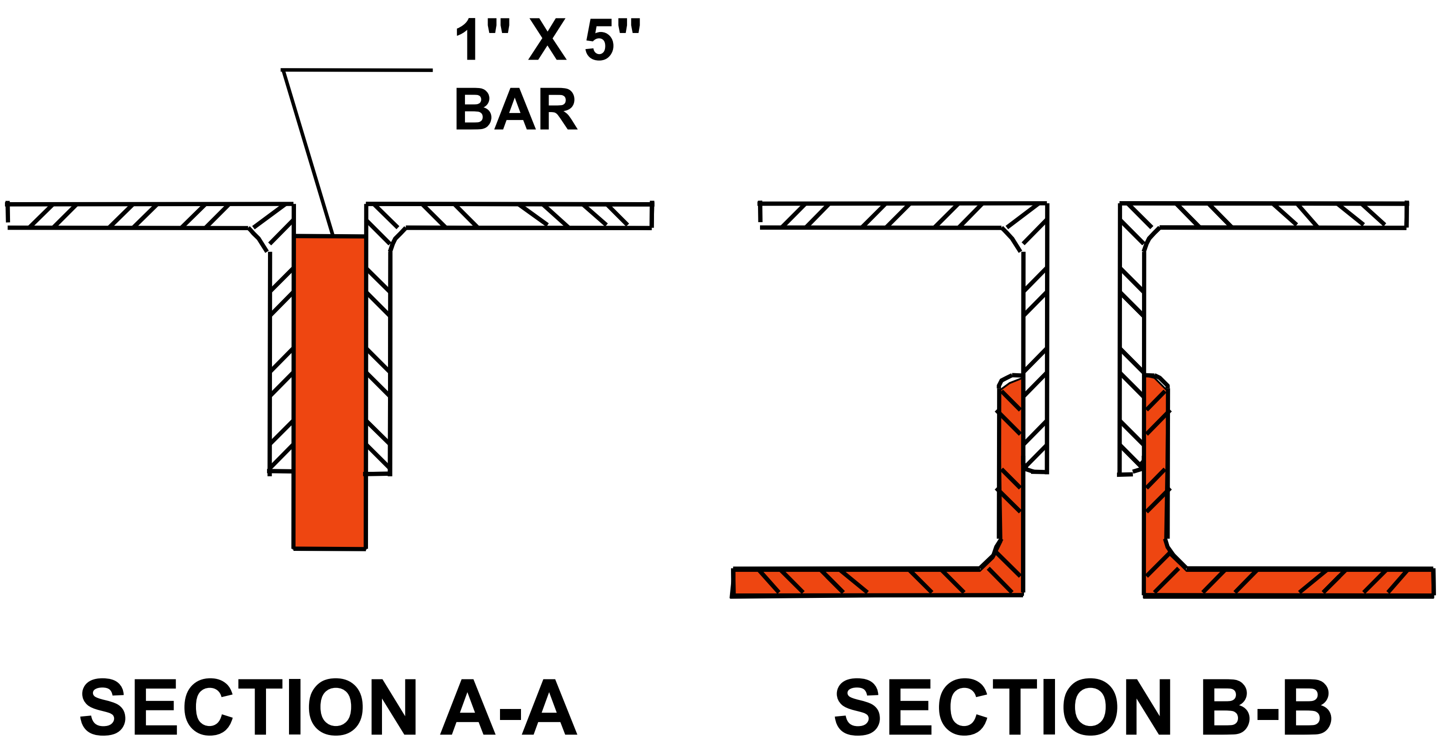
Reinforced Chords



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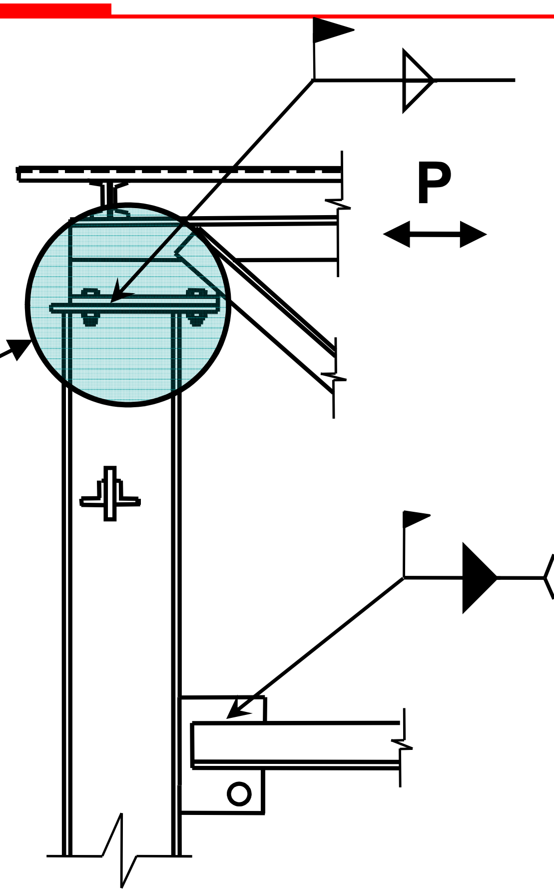
Rigid Seat Connection

P_{max} = 50^K ASD

P_{max} = 75^K LRFD

Special design required



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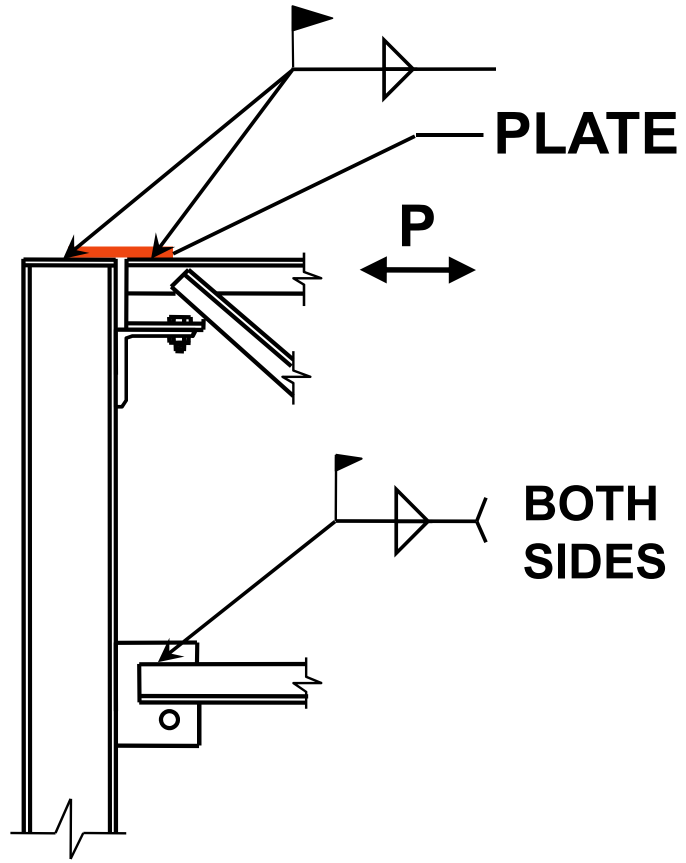
Moment Plate Connection

P_{max} = 200^K ASD

P_{max} = 300^K LRFD



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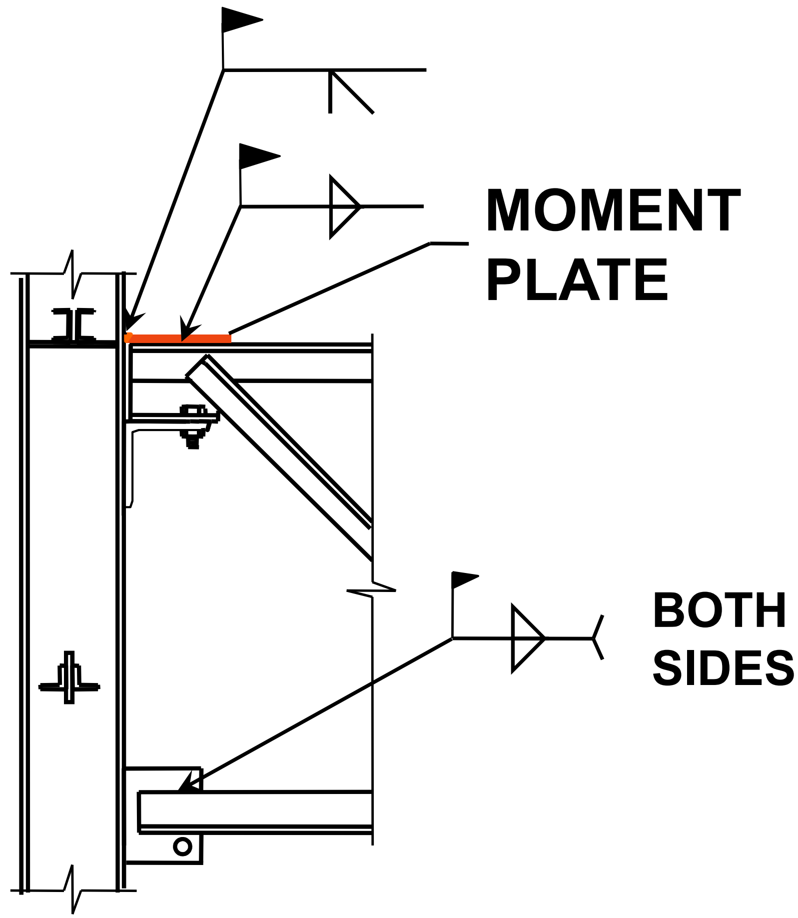
Floor Moment Plate Connection

$P_{max} = 200^K$ ASD

$P_{max} = 300^K$ LRFD



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Bottom Chord Extensions

Consider:

- Magnitude of Bottom Chord Force
- Geometrical Requirements
  - Chord Gap
  - Welding
- Column Type

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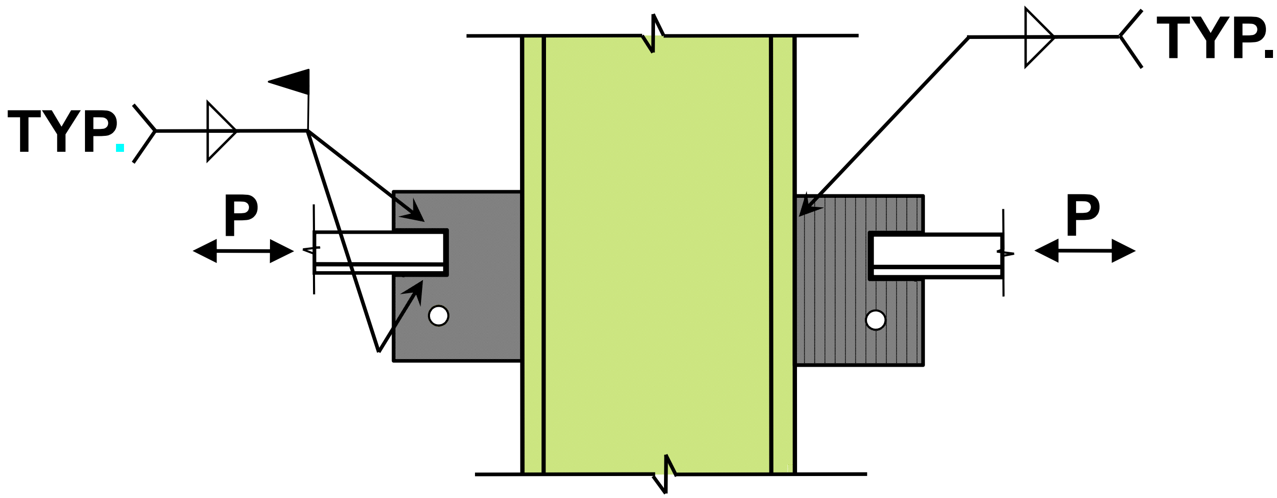
Bottom Chord Attachment to Flanges



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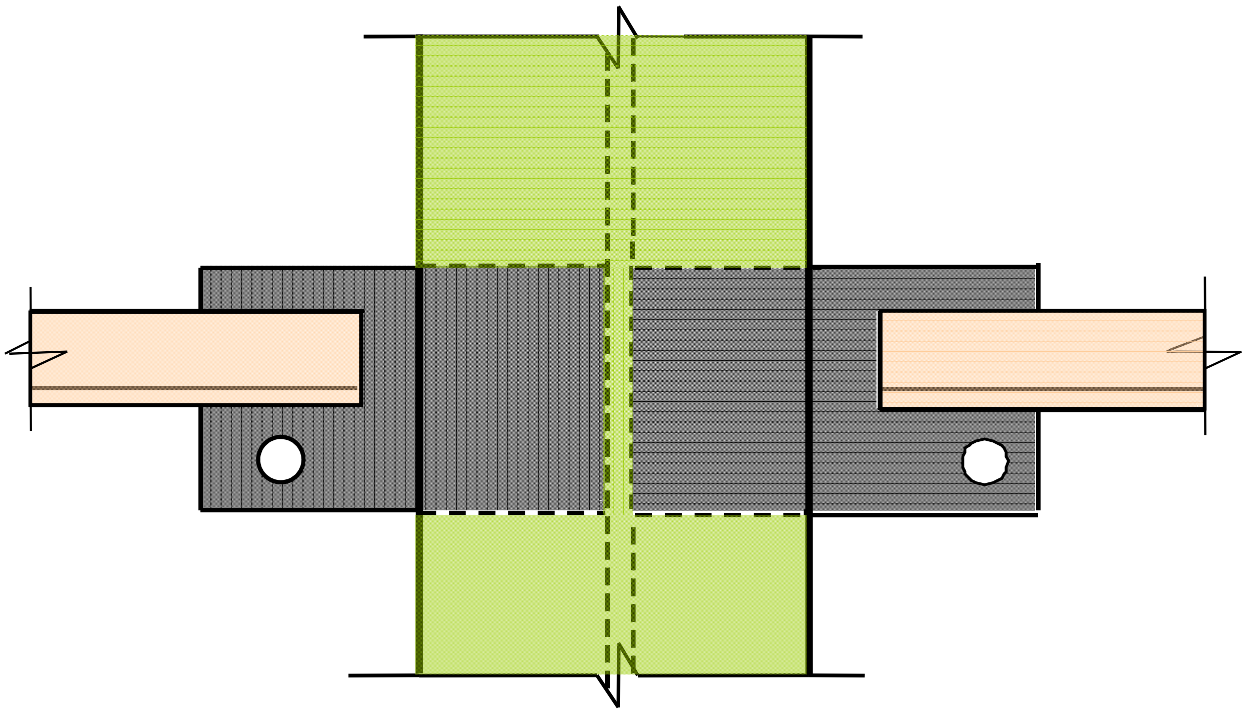
Framing to Column Web



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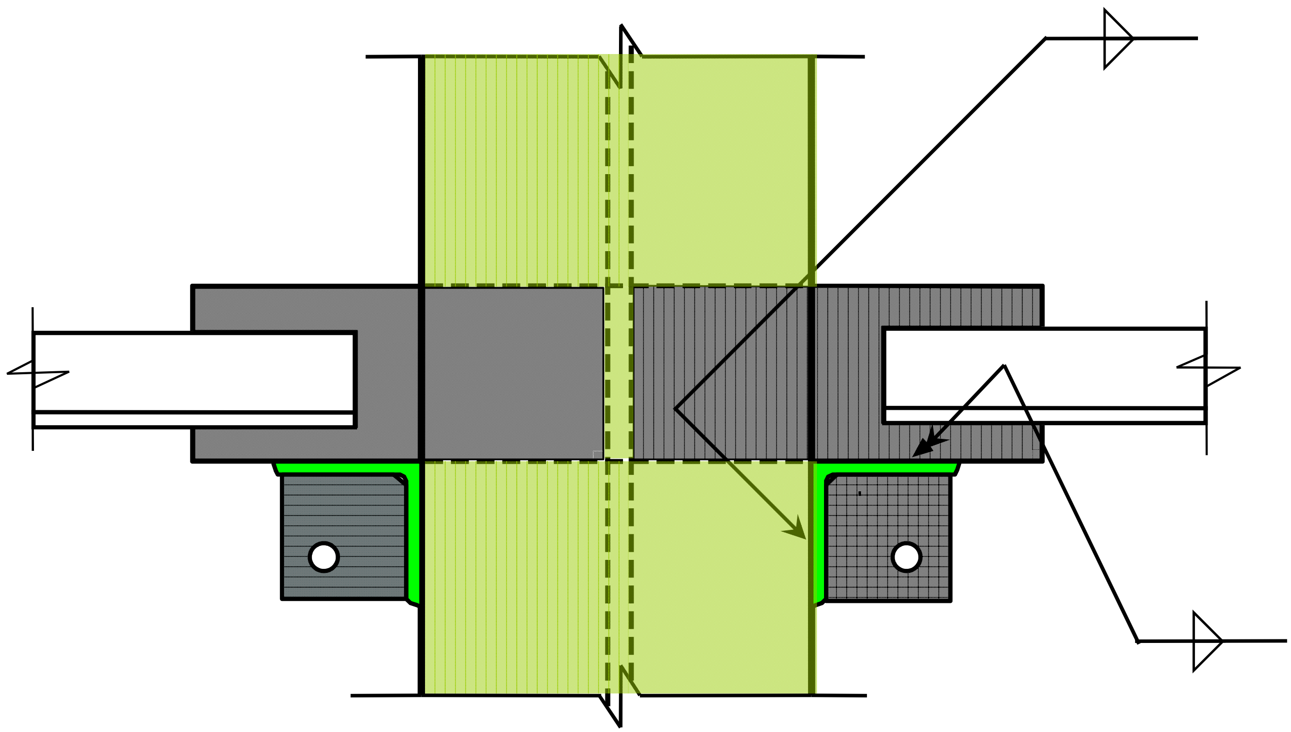
Angle Reinforcement to Stabilizer Plates



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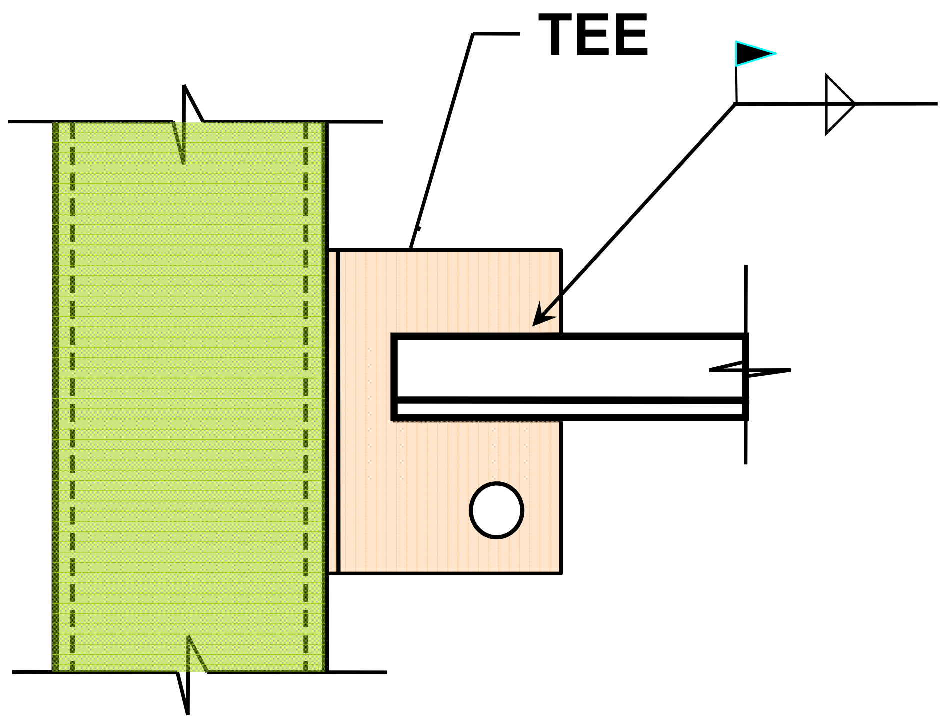
HSS Column



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Rigid Frame Design

$$I_J = 26.767(W_{LL})(L^3)(10^{-6}) / 1.15$$

$$I_{JG\ approx} = 0.027NPLd \text{ using ASD}$$

$$I_{JG\ approx} = 0.018NPLd \text{ using LRFD}$$

where:
N = number of joist spaces
P = Panel point load, kips
L = Joist Girder length, ft.
d = effective depth of the Joist Girder, in. (total depth of the Joist Girder minus y_{bar})

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Examples

- EXAMPLE 1A

  Building Braced at Walls using **ASD**
- EXAMPLE 1B

  Building Braced at Walls using **LRFD**
- EXAMPLE 2

  Rigid Frame Building using **ASD**

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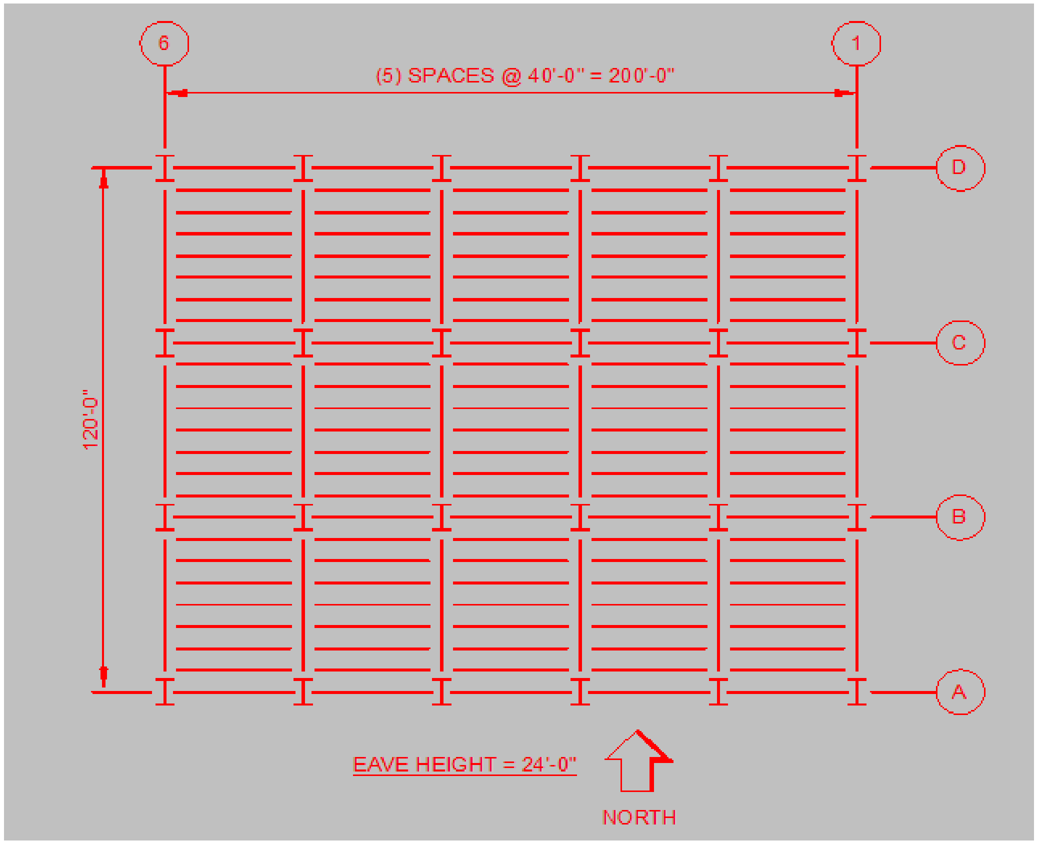
Example 1A: Building Braced at Walls (ASD)



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Example 1A: Building Braced at Walls

Live Load	20 psf
Collateral	5 psf
Dead Load	17 psf
Joist	3 psf
Joist Girders	1 psf
Wind Load	= 32 psf

Joists @ 5' - 0", o/c

Joist Girders at 40 ft. spacing

22 gage Wide Rib Deck

Load Combinations:

$D + L_r$

$D + 0.6W$

Factored Dead Load = 5 psf + 17 psf + 3 psf + 1 psf = 26 psf

Factored L_r Load = 1.0(20 psf) =20 psf

Factored Wind Load = 0.6(32 psf) ≈ 20.0 psf

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Example 1A: Building Braced at Walls

Joist Load = (25 psf + 20 psf)(5 ft.) = 225 plf

From **ASD** Load Table for 40 ft. span,

Select 22K7 with Total load = 231 plf

Joist Girder Load - Interior = (0.230 klf)(40 ft.) = 9.2 kips

- Exterior = (0.230 klf)(40 ft.)/2 = 4.6 kips

Select Joist Girders:

JG1	42G8N4.6K	(Exterior)
JG2	42G8N9.2K	(Interior)

Live Load for Deflection (unfactored)

LL = (20 psf)(5 ft.)(40 ft.) = 4 kips Interior; 2 kips Exterior

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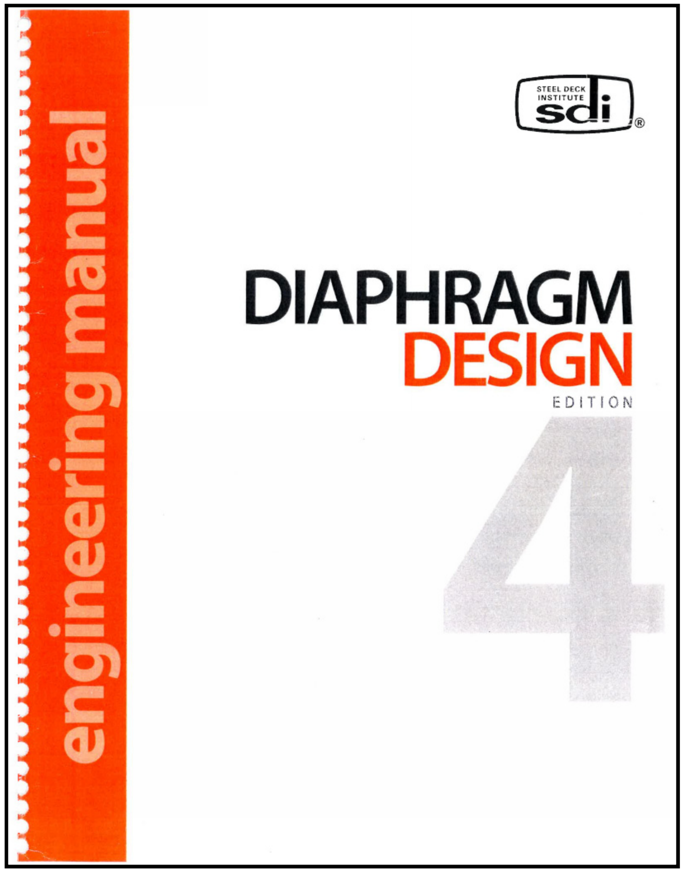
SDI Diaphragm Design Manual (4th Edition)



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Diaphragm Design

Determine the shear at each wall.

The shear wall force equals the eave force per foot times one half of the length of the wind loaded wall.

LINES 1 and 6:

$V_1 = V_6 = (20\text{ psf})(24/2\text{ ft})(200/2\text{ ft}) = 24{,}000\text{ lbs}$

LINES A and D:

$V_A = V_D = (20\text{ psf})(24/2\text{ ft})(120/2\text{ ft}) = 14{,}400\text{ lbs}$

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Diaphragm Design (cont'd)

Determine the maximum shear force per foot in the diaphragm.

The diaphragm shear equals the wall shear divided by the wall length.

LINES 1 and 6:

$$V_1 = V_6 = 24{,}000/120 = 200 \text{ plf}$$

LINES A and D:

$$V_A = V_D = 14{,}400/200 = 72 \text{ plf}$$

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Diaphragm Design (cont'd)

Select Fastening Pattern from SDI Diaphragm Design Manual 4th Ed. (2015)

For welded support fasteners and screwed side laps with 5' joist spacing (5/8 in. arc spot welds):

(Welded side laps are not recommended for 22 ga. decks)

36/4 weld pattern with (1) #10 Tek side lap screw.

v_{allow} = 555/2.35 = 236 plf where Ω (wind) = 2.35

36/5 weld pattern without side lap screws.

v_{allow} = 625/2.35 = 266 plf where Ω (wind) = 2.35

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Diaphragm Design Manual

ROOF DECK

1.5(WR, IR, NR)22
Design thickness = 0.0295 in.
Support fastening: 5/8 in. arc spot welds or equivalent
Side-lap fastening: #10 screws

F_u = 45 ksi
F_y = 33 ksi
F_{xx} = 60 ksi

Loading	ϕ_{df}	Ω_{df}
Seismic	0.55	3.00
Wind	0.70	2.35
Other	0.60	2.65

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span, ft. 3	3.5	4	4.5	5	5.5	6	6.5	7	K_1 1/ft
36/9	0	1775	1560	1390	1230	1100					0.324
	1	1875	1650	1475	1325	1180	1065	970			0.272
	2	1970	1740	1560	1405	1265	1140	1040	955	880	0.234
	3	2060	1830	1640	1480	1350	1220	1110	1015	940	0.206
	4	2145	1910	1715	1555	1420	1295	1180	1080	1000	0.183
	5	2230	1990	1795	1630	1490	1370	1250	1145	1060	0.165
	6	2310	2070	1870	1700	1555	1430	1320	1210	1115	0.151
36/7	0	1135	985	855	755	675					0.486
	1	1250	1090	960	850	760	685	625			0.377
	2	1360	1190	1055	940	845	765	695	640	590	0.308
	3	1465	1285	1145	1030	925	840	765	705	650	0.261
	4	1565	1380	1230	1110	1005	915	835	765	710	0.226
	5	1665	1470	1315	1185	1080	990	905	830	770	0.199
	6	1755	1560	1395	1265	1150	1055	975	895	830	0.178
36/5	0	1005	880	785	700	625					0.583
	1	1100	975	870	785	710	640	585			0.433
	2	1195	1060	950	860	785	715	655	600	555	0.345
	3	1275	1140	1030	935	855	785	720	665	615	0.286
	4	1355	1215	1100	1005	920	845	785	725	670	0.245
	5	1425	1290	1170	1070	980	905	840	785	730	0.214
	6	1490	1355	1235	1130	1045	965	895	835	785	0.190
36/4	0	770	675	600	530	470					0.728
	1	865	765	685	620	555	500	455			0.509
	2	950	850	765	690	635	575	525	480	445	0.391
	3	1025	925	835	760	700	645	595	545	500	0.318
	4	1095	995	905	825	760	705	655	610	560	0.267
	5	1155	1055	965	890	820	760	705	660	620	0.231
	6	1210	1110	1025	945	875	815	760	710	665	0.203
30/6	0	1030	885	765	675	605					0.647
	1	1150	1000	870	770	690	620	565			0.491
	2	1265	1105	975	860	770	695	635	585	540	0.395
	3	1375	1205	1070	955	855	775	705	645	600	0.331
	4	1480	1300	1155	1040	940	850	775	710	660	0.284
	5	1580	1395	1245	1120	1015	925	845	775	715	0.249
	6	1675	1485	1325	1195	1090	1000	915	840	775	0.222
30/4	0	935	825	735	655	585					0.728
	1	1030	915	820	740	670	605	550			0.536
	2	1120	1000	895	815	745	680	620	570	525	0.424
	3	1200	1075	970	885	810	745	690	635	585	0.350
	4	1270	1150	1040	950	875	805	750	695	645	0.299
	5	1335	1215	1105	1015	935	865	805	750	700	0.260
	6	1395	1275	1170	1075	990	920	855	800	750	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See table below.

	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00

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Diaphragm Design Manual

1.5(WR, IR, NR)22
Design thickness = 0.0295 in.
Support fastening: 5/8 in. arc spot welds or equivalent
Side-lap fastening: #10 screws

F_u = 45 ksi
F_y = 33 ksi
F_{xx} = 60 ksi

Loading	ϕ_{df}	Ω_{df}
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Wind	0.70	2.35
Other	0.60	2.65

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span, ft. 3	3.5	4	4.5	5	5.5	6	6.5	7	K_1 1/ft
	0	1775	1560	1390	1230	1100					0.324
	1	1875	1650	1475	1325	1180	1065	970			0.272
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	0	770	675	600	530	[illegible]					0.728
	1	865	765	685	620	555	500	455			0.509
36/4	2	950	850	765	690	635	575	525	480	445	0.391
	3	1025	925	835	760	700	645	595	545	500	0.318
	4	1095	995	905	825	760	705	655	610	560	0.267
	5	1155	1055	965	890	820	760	705	660	620	0.231
	6	1210	1110	1025	945	875	815	760	710	665	0.203
	0	1030	885	765	675	605					0.647

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Diaphragm Design (cont'd)

For power driven fasteners:

36/4 Pneutek SDK63 drive pins with (1) #10 Tek side lap screw – support thickness 0.155" to 0.25".

v_{allow} = 590/2.35 = 251 plf where Ω(wind) = 2.35

36/4 Hilti ENP2K v_{allow} = 530/2.35 = 225 plf

36/5 Pneutek SDK63 drive pins with no side lap screws – support thickness 0.155" to 0.25".

v_{allow} = 615/2.35 = 261 plf where Ω(wind) = 2.35

36/5 Hilti ENP2K v_{allow} = 535/2.35 = 227 plf

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Diaphragm Design (cont'd)

Select Fastening Pattern from SDI Diaphragm Design Manual 4th Ed. (2015)

CHOOSE:

36/4 Weld Pattern

with (1) #10 Tek side lap screw.

NOTE: One side lap fastener is required at a 5 ft joist spacing to meet FM Global and SDI requirements.

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Diaphragm Design (cont'd)

Check the weld strength for combined uplift and shear. Uplift and shear forces are based on the MWFRS values.

Assume the uplift is 60 psf.
The diaphragm shear is 200 plf.

Determine the combined shear and uplift requirement Using the AISI Specification and Tables from the SJI Diaphragm Manual. ASD design.

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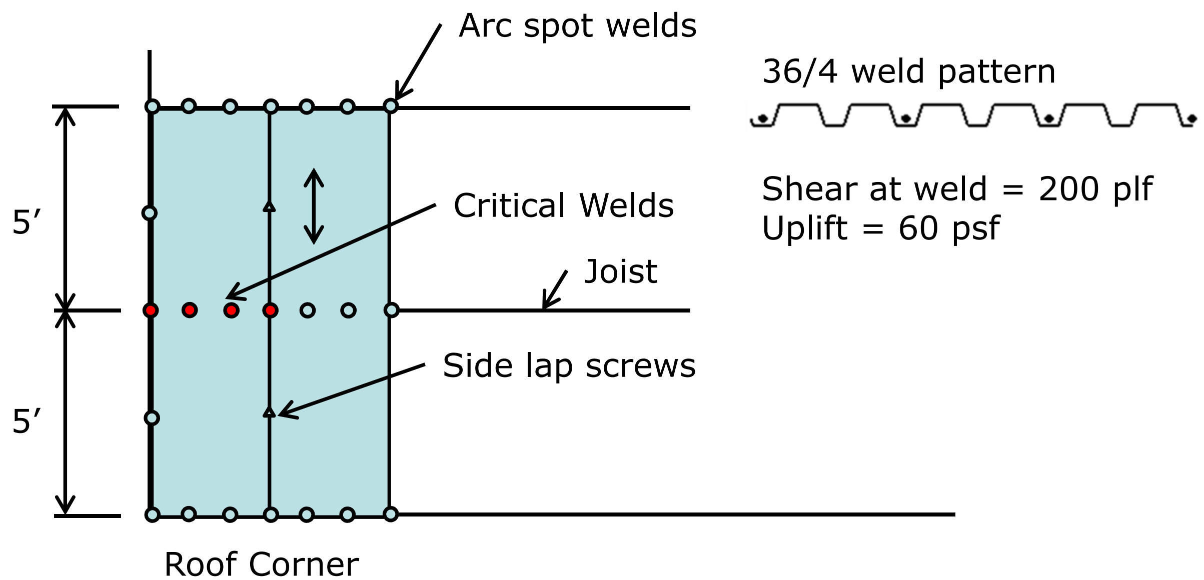
Combined Weld Shear and Uplift



Shear at weld = 200 plf
Uplift = 60 psf

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Combined Weld Shear and Uplift

Determine tension force per weld.

36/4 pattern
Number of effective welds across the sheet width, per SDI = 2.7 (edge welds are only 70% effective 2 + 0.7 = 2.7).

Tension per fastener, T= (60 psf)(5 ft)(3ft)/(2.7)
= 333 lbs/weld

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Combined Weld Shear and Uplift

Table 9-8 Support Fastener Nominal Shear Strength (lbs)

Support Fastener	P_{nf}, lbs / Deck Gage										
	Form Deck			Roof Deck				Composite Deck			
	26	24	22	22	20	18	16	22	20	18	16
5/8" arc spot weld - E60XX	--	--	2147	1739	2088	2710	3346	1943	2413	3132	3867
3/4" arc spot weld - E60XX	--	--	--	[illegible]	2531	3297	4086	1997	2857	3810	4722
16 gage weld washer with 3/8" hole - E60XX	1455	1503	--	--	--	--	--	--	--	--	--
#12 screw	647	864	1067	774	940	1244	1569	895	1086	1437	1814

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Combined Weld Shear and Uplift

Table 9-15 Nominal Arc Spot Weld Uplift Strength (Concentric Loading)

Gage	Concentric Uplift Strength - lbs (Nominal)			
	Visable Diameter (Inches)			
	0.5	**0.625**	**0.75**	**1**
22	867	1098	1328	1789
20	1039	[illegible]	1598	2157
18	1341	[illegible]	2081	2822
16	1645	2112	2580	3514

Assumes F_{exx} = 60 ksi

Assumes F_y = 40 ksi; F_u = 50 ksi; which will give lower weld strength than 33/45 ksi

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Combined Weld Shear and Uplift

$\Omega_t = 2.5,\ \Omega_f = 2.55$

$P_{nt} = 1098$ lbs (available tensile strength) P_{nf} = 1739 lbs (available shear strength)

$\Omega_t T = (2.5)(333) = 833$ lbs, (required tension)

If $\left(\frac{\Omega_t T}{P_{nt}}\right)^{1.5} \leq 0.15$ then $P_{nft} = P_{nt}$; $\left(\frac{\Omega_t T}{P_{nt}}\right)^{1.5} = \left[\frac{833}{1098}\right]^{1.5} = 0.660 > 0.15$ (AISI S310)

$P_{nft} \neq P_{nt}$

Else $\left(\frac{P_{nft}}{P_{nf}}\right)^{1.5} + \left(\frac{\Omega_t T}{P_{nt}}\right)^{1.5} \leq 1.0$ (AISI 310- Modified)

$$P_{nft} = P_{nf}\left[1-\left(\frac{\Omega_t T}{P_{nt}}\right)^{1.5}\right]^{2/3} = P_{nf}\left[1-\left(\frac{\Omega_t T}{P_{nt}}\right)^{1.5}\right]^{2/3} = 1739(1-0.660)^{2/3} = 847 \text{ lbs} \leq 1098 \text{ lbs n.g.}$$

The SDI Diaphragm Manual outlines a method of checking the diaphragm shear to comply with the combined uplift and shear.

There's always a solution in steel!

Check Diaphragm Chord Forces

The edge joists are assumed to resist the diaphragm chord force. Determine diaphragm chord force in edge joist due to **wind** loadings:

$M_{A\&D}$ = $wL^2/8 = (0.020)(24/2)(200)^2 /8 = 1200$ ft.-kips

P_{chord} = M / diaphragm depth = 1200 ft-kips/120 ft = 10 kips

There's always a solution in steel!

Check Diaphragm Chord Forces (cont'd)

Determine top chord force in the joist due to **dead** load:

M_D = $wL^2/8$ = (17 psf + 3 psf +5 psf)(5 ft /2)$(40 \text{ ft})^2$ /8

= 12,500 ft.- lbs

P_D = M/depth = (12,500)(12) /(22 in.-1.0 in.)

= 7,140 lbs

P_r = 7.14 kips + 10 kips = 17.1 kips

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Check Diaphragm Chord Forces (cont'd)

Determine allowable joist chord force for typical edge joist (22K7)

For the 22K7 the allowable gravity load = 231 plf

M_{allow} = (0.231)(40)^2 / 8 = 46.2 ft.-kips

P_{chord} = (46.2 ft-kips)(12) / 21 in. = 26.4 kips

26.4 kips > 17.1 kips ok

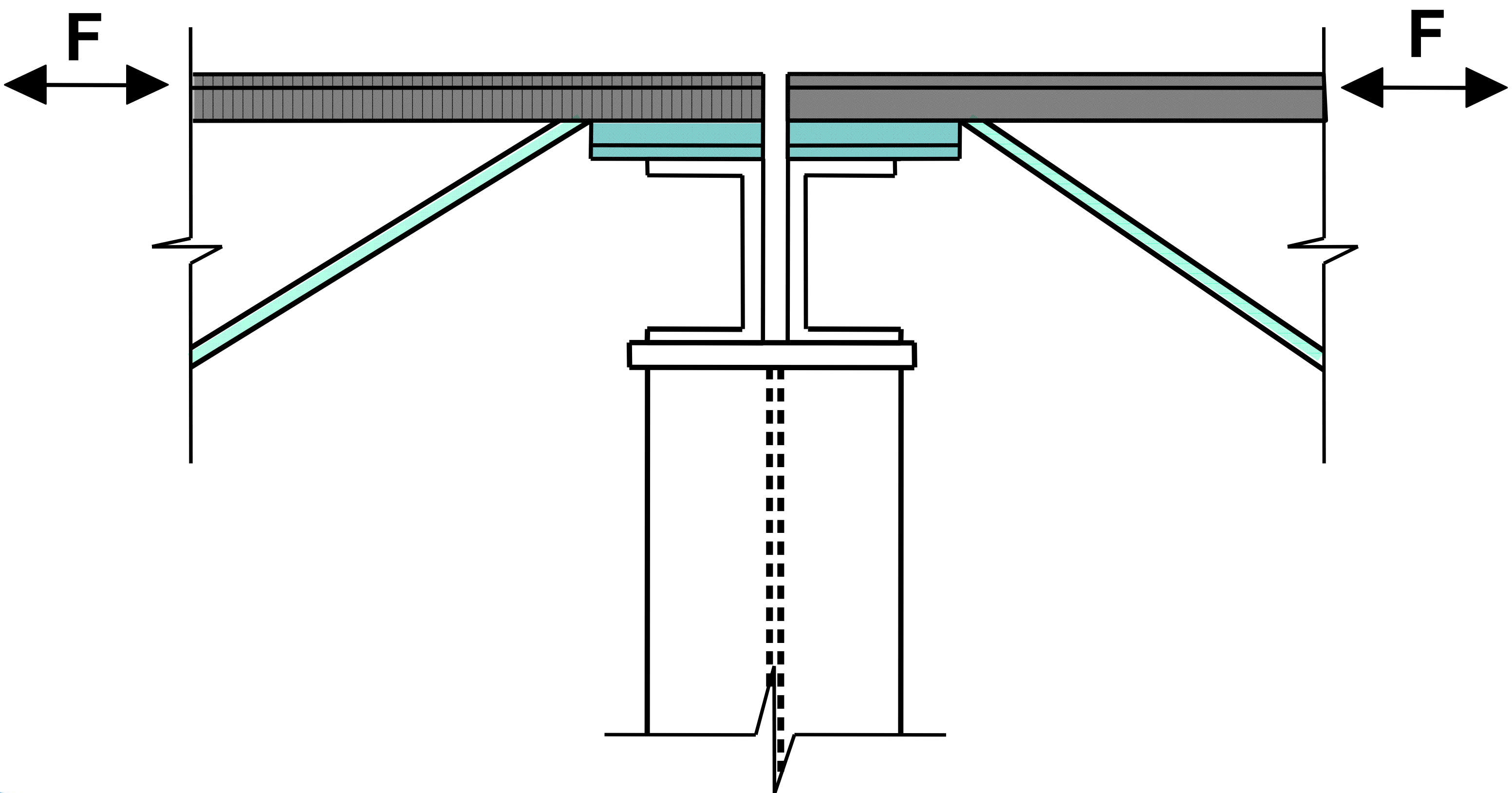
There's always a solution in steel!

Check Diaphragm Chord Forces (cont'd)

- The 22K7 joist is adequate as the diaphragm chord at A and D, provided eccentric chord bending is eliminated.
- Don't try to pass the chord force through the joist seat.

There's always a solution in steel!

Diaphragm Chord Force

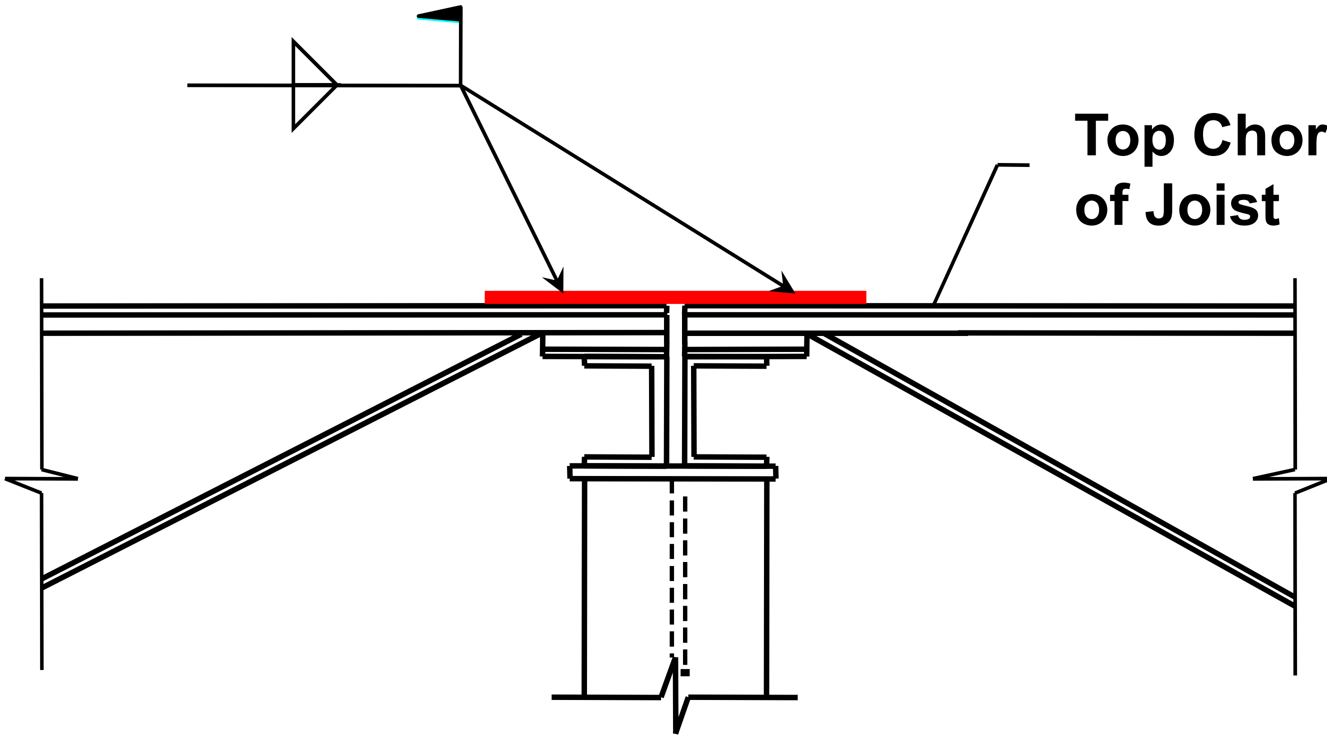


There's always a solution in steel!

Check Diaphragm Chord Forces (cont'd)

To eliminate bending stresses in the joist top chord use a tie plate between adjacent joists to transfer the chord force.

Joist Tie Plate



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Size Top Plate and Weld

The chords may act in tension or compression, therefore design for compression case.

Try a 4” wide x 1/4” thick, A36 tie plate.

Chord Force P_{chord} = 17.1 kips

Assume unwelded (unsupported) plate length of 4”, check the buckling capacity of the plate:

Plate Properties:

A = 1.0 in.^{2}

r_x = d/(12)$^{.5}$ =0.072 in.

L / r_x = 56

F_{cr}/Ω = 18.3 ksi (AISC Manual Table 4-22)

P_a = (18.3)(1.0) = 18.3 kips

18.3 >17.1 ok

There’s always a solution in steel!

Size Top Plate and Weld (cont'd)

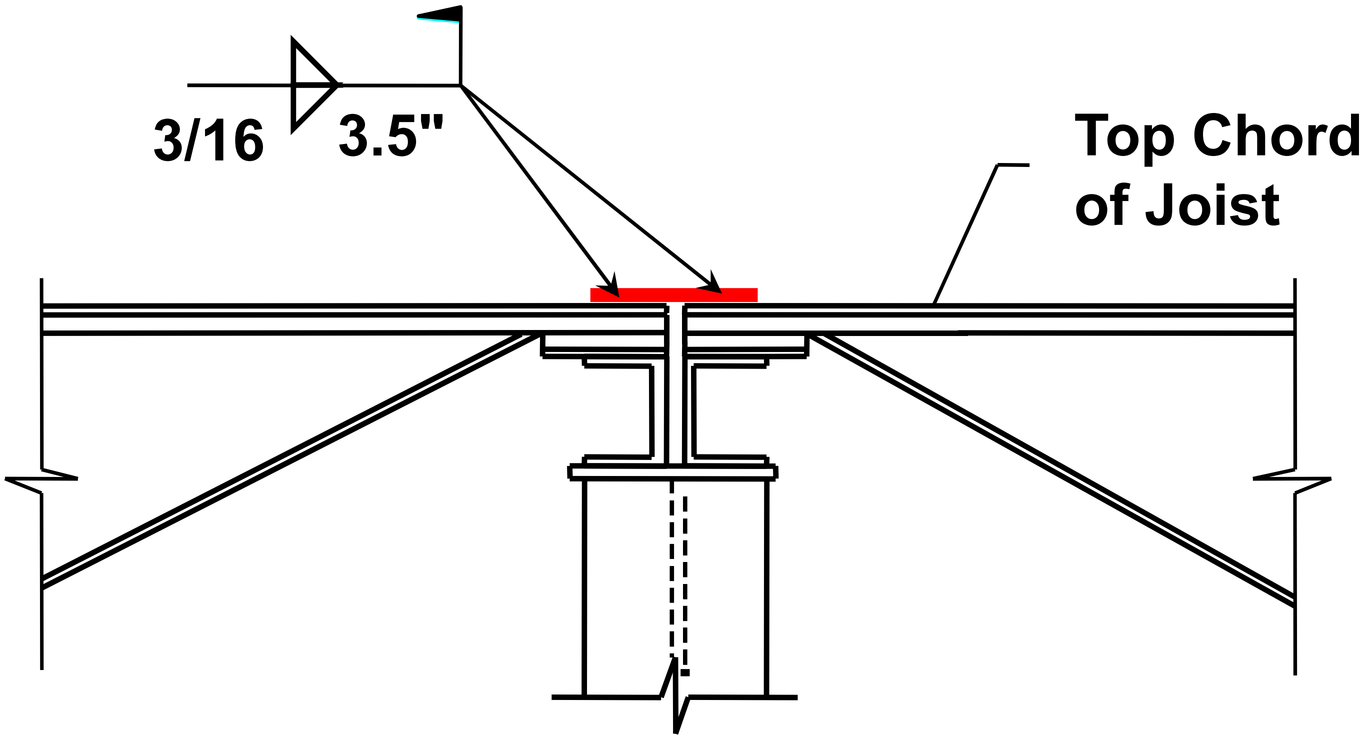
Use a 3/16 inch E70 fillet weld:

The allowable weld strength = 2.78 kips / in.

Weld length req'd = 17.1 / 2.78 = 6.2 inches

Therefore, use 7 inches of 3/16 inch fillet on each joist end.

Joist Tie Plate



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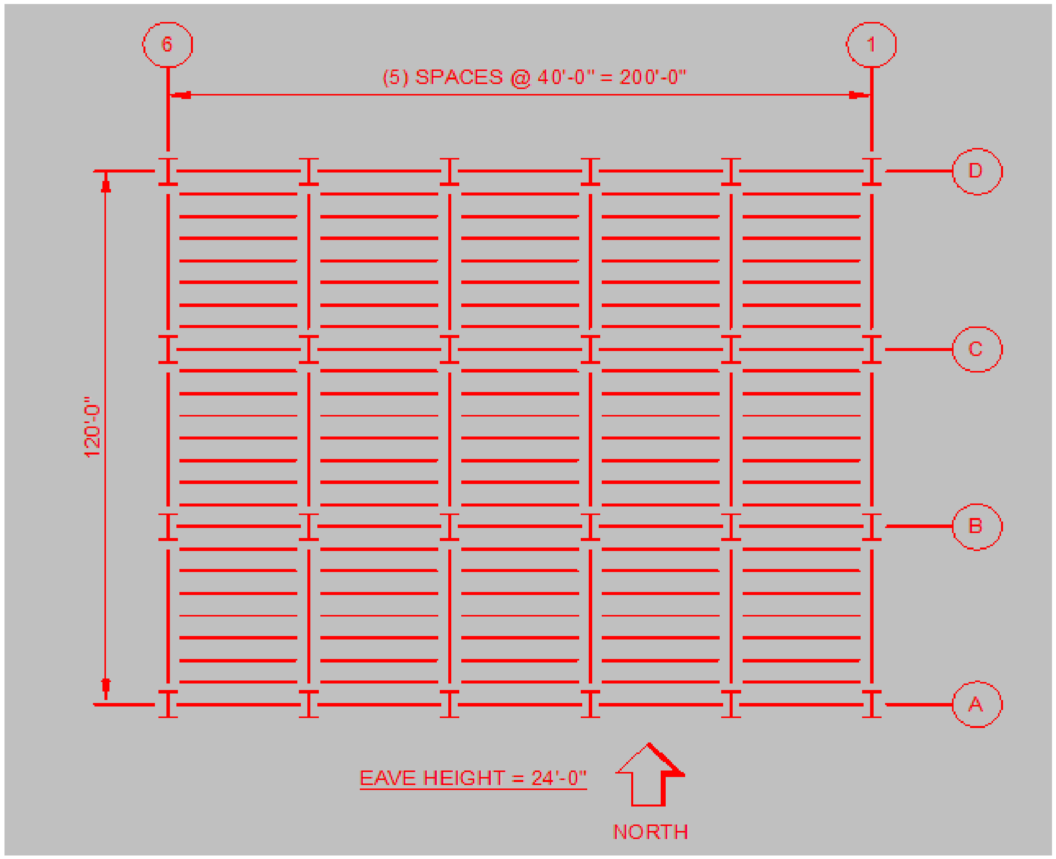
Example 1B: Building Braced at Walls (LRFD)



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Example 1B: Building Braced at Walls

Live Load	20 psf
Collateral	5 psf
Dead Load	17 psf
Joist	3 psf
Joist Girders	1 psf
Wind Load	= 32 psf

Joists @ 5' - 0", o/c

Joist Girders at 40' spacing

22 gage Wide Rib Deck

Load Combinations:

$1.2D + 1.6L_r$

$1.2D + 1.0W + 0.5L_r$

Factored Dead Load = 1.2 (5 psf + 17 psf + 3 psf + 1 psf)

= 31.2 psf

Factored Roof Live Load = 1.6(20) = 32.0 psf

Factored Wind Load = 1.0 (32 psf) = 32.0 psf

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Example 1B: Building Braced at Walls

Joist Load = (31.2 psf + 32 psf)(5 ft.) = 316 plf
From **LRFD** Load Table for 40 ft. span,
Select 22K7 with Total load = 346 plf

Joist Girder Load - Interior = (0.316 klf)(40 ft.) = 12.6 kips
- Exterior = (0.316 klf)(40 ft.)/2) = 6.3 kips

Select Joist Girders:

JG1	42G8N6.3F	(Exterior)
JG2	42G8N12.6F	(Interior)

Live Load for Deflection (unfactored)
LL = (20 psf)(5 ft.)(40 ft.) = 4 kips Interior; 2 kips Exterior

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Diaphragm Design

Determine the shear at each wall:

The shear wall force equals the eave force per foot times one half of the length of the wind loaded wall.

LINES 1 and 6:

$V_1 = V_6 = (32\ \text{psf})(24\ \text{ft}/2)(200\ \text{ft}/2) = 38{,}400\ \text{lbs}$

LINES A and D:

$V_A = V_D = (32\ \text{psf})(24\ \text{ft}/2)(120\ \text{ft}/2) = 23{,}000\ \text{lbs}$

Diaphragm Design (cont'd)

Determine the maximum shear force per foot in the diaphragm.

The diaphragm shear equals the wall shear divided by the wall length.

LINES 1 and 6:

$$V_1 = V_6 = (38{,}400 \text{ lbs})/(120 \text{ ft}) = 320 \text{ plf}$$

LINES A and D:

$$V_A = V_D = (23{,}000 \text{ lbs})/(200 \text{ ft}) = 115 \text{ plf}$$

Diaphragm Design (cont'd)

Select Fastening Pattern from SDI Diaphragm Design Manual 4rd Ed. (2015)

For welded support fasteners and screwed side laps with 5' joist spacing:

(Welded side laps are not recommended for 22 ga. decks)

36/4 weld pattern with (1) #10 Tek side lap screw.

$v_{available}$ = (555 plf)(0.7) = 389 plf (ϕ = 0.7)

36/5 weld pattern without side lap screws.

$v_{available}$ = (626 plf)(0.7) = 438 plf

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Diaphragm Design (cont'd)

Select Fastening Pattern from SDI Diaphragm Design Manual 4th Ed. (2015)

CHOOSE: 36/4 Weld Pattern with (1) #10 Tek side lap screw.

NOTE: One side lap fastener is required at a 5 ft joist spacing to meet Factory Mutual and SDI requirements.

Check Diaphragm Chord Forces

The edge joists are assumed to resist the diaphragm chord force. Determine diaphragm chord force in edge joist due to **wind** loadings:

$M_{A\&D}$ = $wL^2/8$ = (32 psf)(24 ft)/2)$(200)^2$/8/1000

= 1,920 ft.-kips

P_{chord} = M/diaphragm depth = 1,929 kips/120 ft = 16.0 kips

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Check Diaphragm Chord Forces (cont'd)

From $1.2D + 1.0W + 0.5L_r$

M_D = (1.2)(25 psf)(2.5 ft.)(40 ft.)^2/8 = 15,000 ft.-lbs

M_L = (0.5)(20 psf)(2.5 ft.)(40 ft.)^2/8 = 5,000 ft.-lbs

$P_{chord\ D}$ = M_D /depth = (15,000 ft.- lbs)(12)/(22 in.- 1.0 in.)

= 8,570 lbs = 8.6 kips

$P_{chord\ L}$ = M_L /depth = (5,000 ft.-lbs)(12)/(22 in.- 1.0 in.)

2,860 lbs = 2.9 kips

$P_{chord\ W}$ = 16 kips; P_r = 8.6 kips + 16 kips+ 2.9 kips = 21.5 kips

There's always a solution in steel!

Check Diaphragm Chord Forces (cont'd)

P_{22K7} = [wL²/8] / depth = [0.346 klf (40)² /8)](12)/(21)

= 39.5 kips

Evaluate joist top chord forces:

$P_{1.2D+1.5L+W} < P_{22K7}$

21.5 kips < 39.5 kips ok

There's always a solution in steel!

Size Top Plate and Weld

Try a 4" wide x 1/4" thick, A36 tie plate.

Factored Chord Force = 21.5 kips

Assume unwelded (unsupported) plate length of 4 in., check the buckling capacity of the plate:

Plate Properties:

$A = 1.0\text{ in.}^2$

$r_x = d/(12)^{.5} = 0.076\text{ in.}$

$L / r_x = 56$

$\phi F_{cr} = 27.5\text{ ksi}$

Pa = (27.5)(1.0) = 27.5 kips

27.5 >21.5 ok

There's always a solution in steel!

Size Top Plate and Weld (cont'd)

Use a 3/16 inch E70 fillet weld:

The design weld strength = 4.18 kips / in.

Weld length req'd = 21.5 kips/4.18 = 5.14 inches

Therefore, use 6 inches of 3/16 inch fillet on each joist end.

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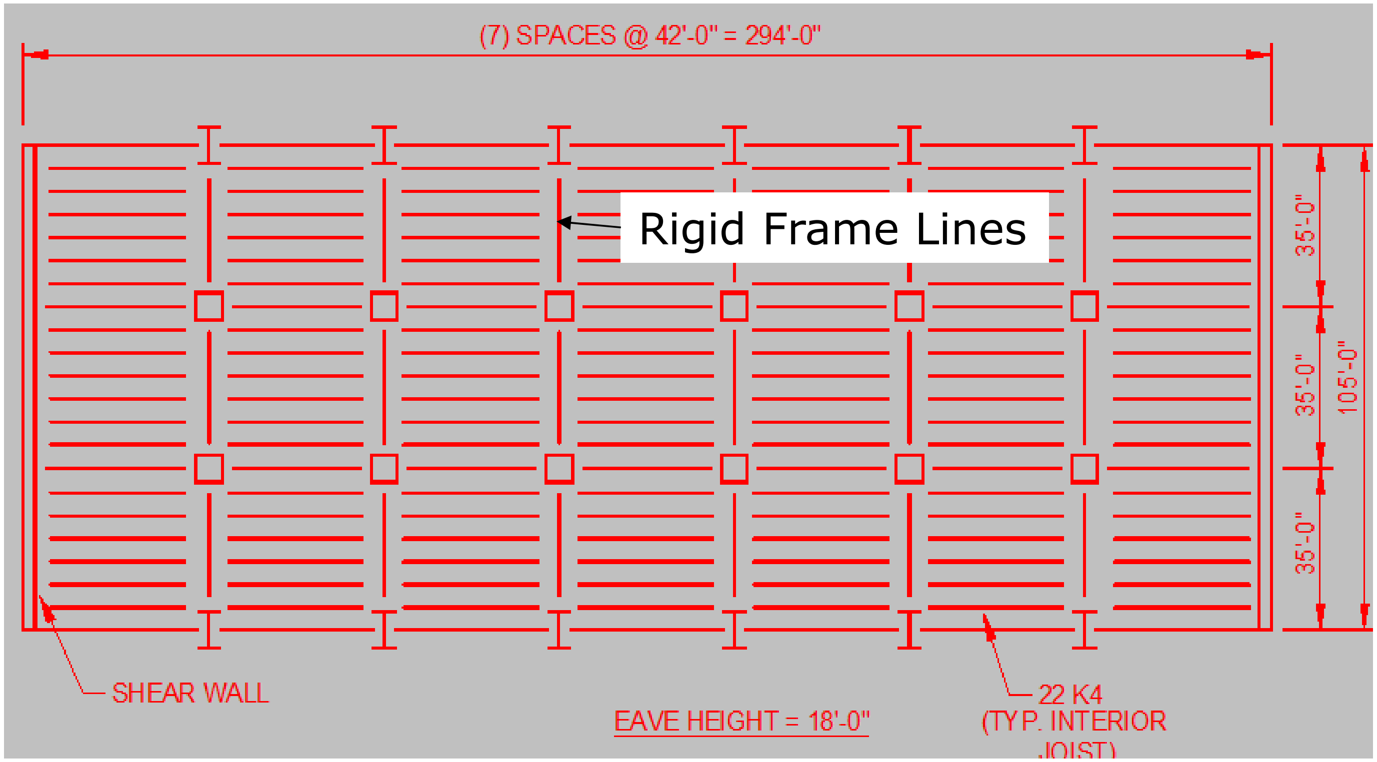
Example 2: Rigid Frame Building (ASD)



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Example 2: Rigid Frame Building

Dead Load:	5 psf (Standing Seam Roof)
Collateral Load:	4 psf
Joist Girder:	1 psf
Live Load:	20 psf (Live load reduction allowed)
	12 psf – Reduced Live Load on Joist Girders only

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Example 2: Rigid Frame Building

Structure is Category II. From ASCE 7-10 Figure 26.5-1A the basic wind speed is 115 mph (3 second gust). For the 18 ft high structure with Exposure C the lateral load equals 32.7 psf. Roof Uplift: 27.4 psf (Suction).

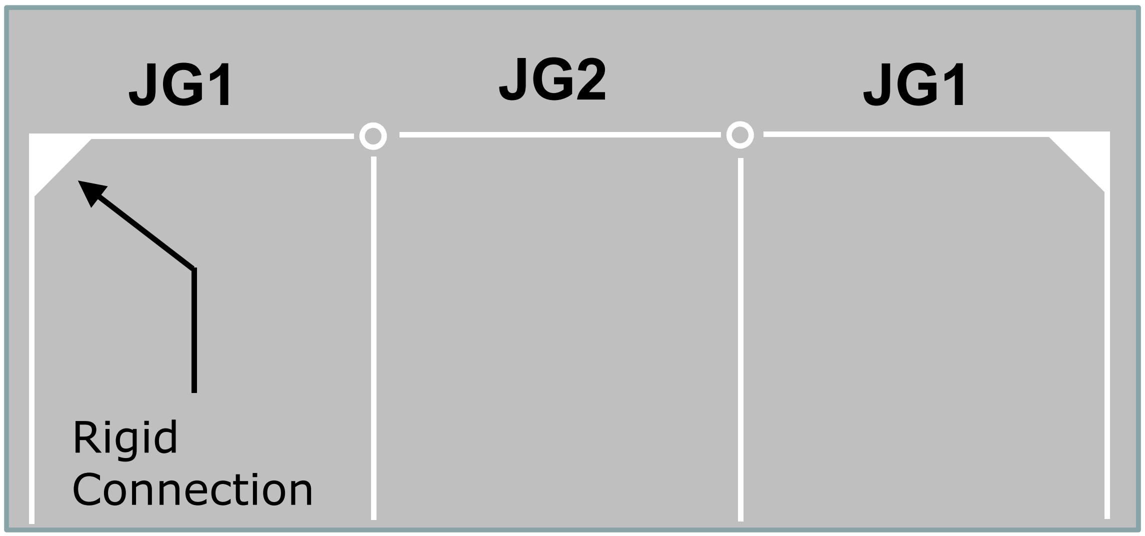
By inspection the following load combination controls:

D + 0.6W; D = (10 psf)(42 ft) = 420 plf

Eave strut load = (0.6)(32.7)(42 ft)(18 ft/2) = 7,420 lbs

Since this is an ASD design, loads must be multiplied by 1.6 and after a second order analysis divided by 1.6.

Design Joist Girder Rigid Frame (cont'd)



JG1 and JG2 are 36" deep, with 7 panel spaces

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Design Joist Girder Rigid Frame

Joist Girder Panel Point Load:

$P = (DL + Coll. + LL_{req'd})(5)(42) = 4.6 \text{ kips}$

Use a 36G7N4.6K Joist Girder

Determine the joist girder moment of inertia:

$I_{JG} = 0.027\ PNLd = (0.027)(4.6)(7)(35)(36 - 2 \times 0.5^*) = 1065 \text{ in.}^4$

*(0.5 in. is the estimated centroid distance for the chord angles)

Provide a rigid connection between the perimeter column and the Joist Girder. The connection is to be made after all dead loads are applied.

The Basic Connection is used for the Joist Girders at the interior columns.

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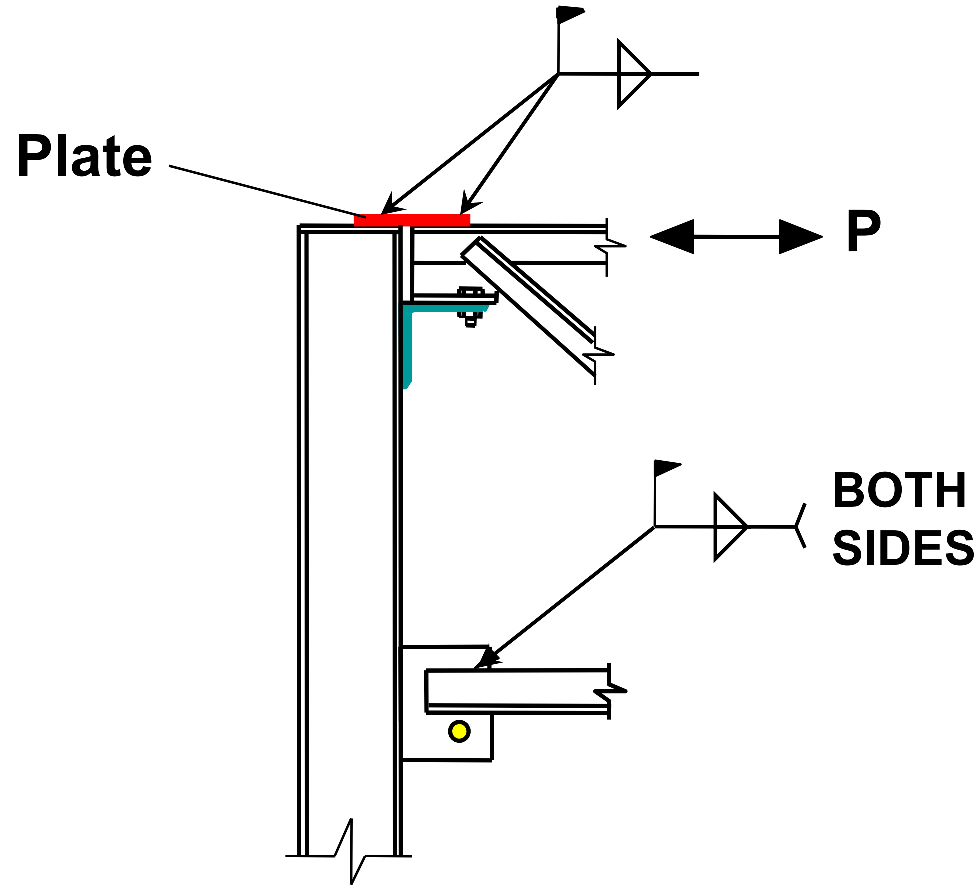
Moment Plate Connection



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Design Joist Girder Rigid Frame (cont'd)

Exterior Columns:

Try W14x38 (F_y = 50 ksi)

A = 11.2 in.^{2}

I_x = 385 in.^{4}

r_x = 5.87 in.

Interior Columns:

Try HSS 8x8x1/4 (F_y = 46 ksi)

A = 7.10 in.^{2}

I_x = 70.7 in.^{4}

r_x = 3.15 in.

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Design Joist Girder Rigid Frame (cont'd)

Joist Girder End Moments and Column Forces

Based on frame analysis:

D + (.6)(W) Controls

M_r = -1005 in. kips = 83.8 ft-kips

P_r = 9.8 kips

From the analysis, the eave deflection at full wind load is 1.9 inches. H/100 = 2.2 in. For a 10 year wind (using 75% of full wind) the deflection = 1.4 in.

= H/150

Example 2: Rigid Frame Building

Use AISC Manual Table 6 - 1 to determine column size.

Try the W 14x38.

$p \times 10^3 = 11.5$, $b_x \times 10^3 = 10.9$

$pP_r = (0.0115)(9.8 \text{ kips}) = 0.11 \leq .2$

$1/2pP_r + 9/8(b_x M_{rx}) = (1/2)(0.11) + (9/8)(0.0109)(83.8)$

$= 1.08$ ng

Brace at mid-height: $p = 4.24 \times 10^3$, $b_x = 6.61 \times 10^3$

$1/2pP_r + 9/8(b_x M_{rx}) = (1/2)(0.00424)(9.8 \text{ kips})$

$+ (9/8)(0.00661)(83.8) = 0.64$ ok

Use W14x38

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Example 2: Rigid Frame Building

Support the girder on column bracket and use a top plate welded to Joist Girder and to the column cap plate to transfer the force.

Top Chord: M_{end} = 1005 in.-kips

$P_{chord} = M_{end}/d$ = 1005 in.-kips/(36 in. – 1.5 in.) = 29.13 kips

d is the distance from the top of the top chord to the half depth of the bottom chord leg.

$A_{req'd} = P_{chord}/F_t$ (where $F_t = F_y/\Omega$)

= 29.13/22 = 1.32 in.^{2}

Use a 4 in. x 1/2 in. plate; A =2.00 in.^{2}

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Design Joist Girder End Connection (cont'd)

Check Plate for compression:

Controlling load case: D + 0.6W

P_{chord} = -29.13 kips

L_x = 3 in. (unbraced length)

r_x = 0.144 in.

$L/r_x = 20.8 \leq 25$

F_{cr}/Ω_c = 21.6 ksi

$P_a = (F_{cr}/\Omega)(A) = (21.6\text{ ksi})(2.0\text{ in.}^2)$

$= 43.2\text{ kips} > 29.13\text{ kips}$ ok

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Design Joist Girder End Connection (cont'd)

Determine top plate weld:

P_r = 29.13 kips

3/16 in. weld: v allow = 2.78 kips/in.

Required weld length: L = 29.13 kips/2.78 kips/in. = 10.5 in.

To eliminate shear lag effects on the plate use a weld equal to twice the with of the plate.

Use 10 in. of 3/16" weld each side an at each end of plate

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Design Joist Girder End Connection (cont'd)

Determine bottom chord connection:

Try a 6" x 3/4" plate

$A = 4.5$ in., $P_r = 36.5$ kips

$P_a = (F_y)(A)/\Omega = (50)(4.5\text{ ksi})/\Omega = 135$ kips

$P_a < P_r$ ok

(Note: 1/4" minimum per AISC Sect. J)

$v_{allow} = 3.71$ kips/in.

Required weld length, $L_w = 29.13$ kips / (3.71) = 7.85 in.

Use 4 inches of 1/4 in. fillet weld each side of stabilizer

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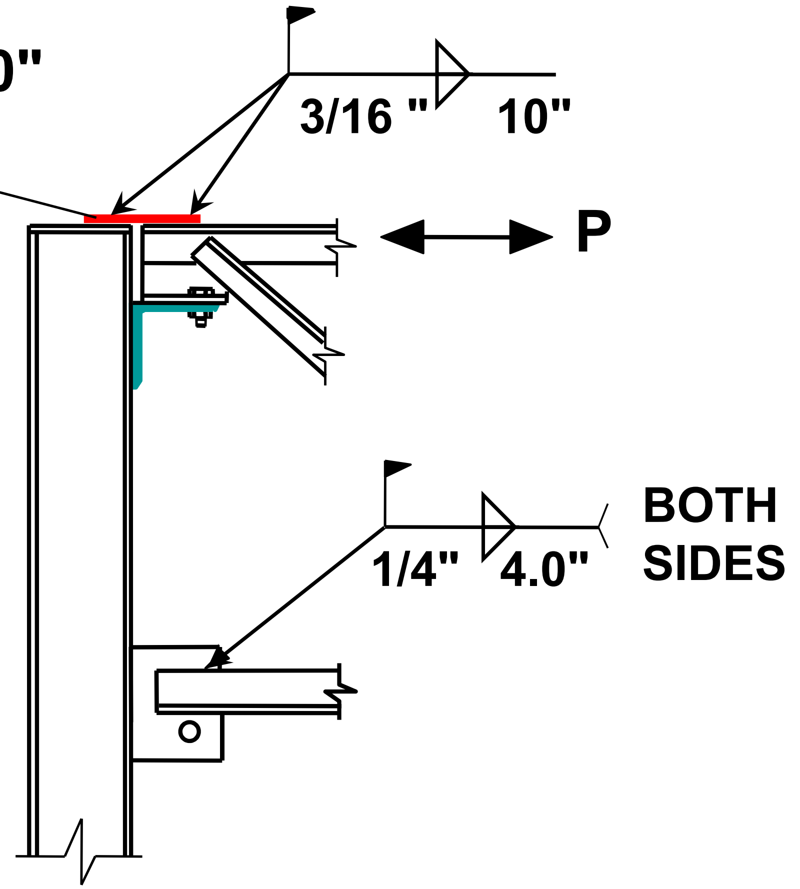
Moment Plate Connection



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Design Joist Girder End Connection (cont'd)

Check column web for web yielding & web crippling:

(Requirements per AISC Sections J10.2 and J10.3)

Web Local Yielding: $\Omega = 1.5$

$R_n/\Omega = F_{yw}t_w(5k + l_b)/\Omega$

= (50 ksi)(0.31 in.)[(5)(0.915 in.) + 6 in.]/1.5 = 109.3 kips

= 109.3 kips > 29.13 kips ok

There's always a solution in steel!

Design Joist Girder End Connection (cont'd)

Web Local Crippling:

$$R_n = 0.80t_w^2\left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$R_n = (0.80)(0.31)^2\left[1+3\left(\frac{6}{14.1}\right)\left(\frac{0.31}{0.515}\right)^{1.5}\right]\sqrt{\frac{(29000)(50)(0.515)}{0.31}}$$

$$R_n = 190 \text{ kips} \quad \frac{R_n}{\Omega} = \frac{190}{2.00} = 95 \text{ kips} > 29.13 \text{ ok}$$

There's always a solution in steel!

SJI Spreadsheet Solutions

- Joist Girder Moment Connections to the Strong Axis of Wide Flange Columns
- Joist Girder Moment Connections to the Strong Axis of Wide Flange Columns- Intermediate Levels
- Joist Girder Moment Connections to the Weak Axis of Wide Flange Columns
- Joist Girder Moment Connections to HSS Columns- Top Plate
- Joist Girder Moment Connections to HSS Columns- Knife Plates
- Joist Girder Moment Connections to Strong Axis of WF Columns- Knife Plates

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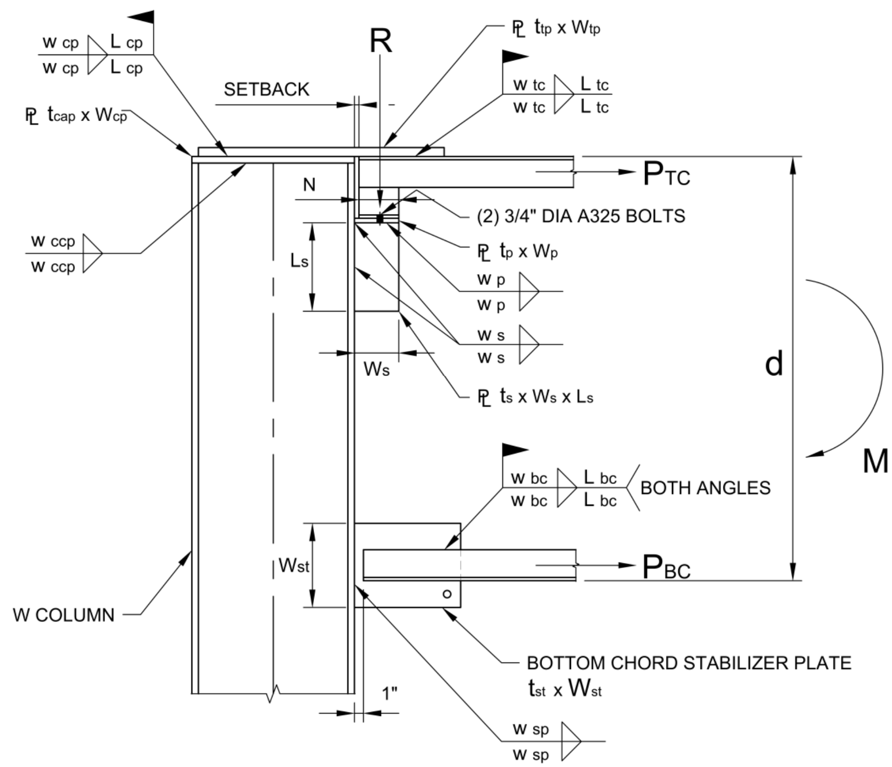
SJI Spreadsheet Solution



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SJI Spreadsheet Solutions

All Spreadsheets are available for free download from:

Steeljoist.org.

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End of Lesson 3

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Individual Webinar Registrants

PDH Certificates

Within 2 business days…

- You will receive an email on how to report attendance from: 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

PDH Certificates

Within 2 business days…

- New reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
- Password: Same as AISC website password.

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8-Session Registrants

PDH Certificates

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

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8-Session Registrants

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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8-Session Registrants

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

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

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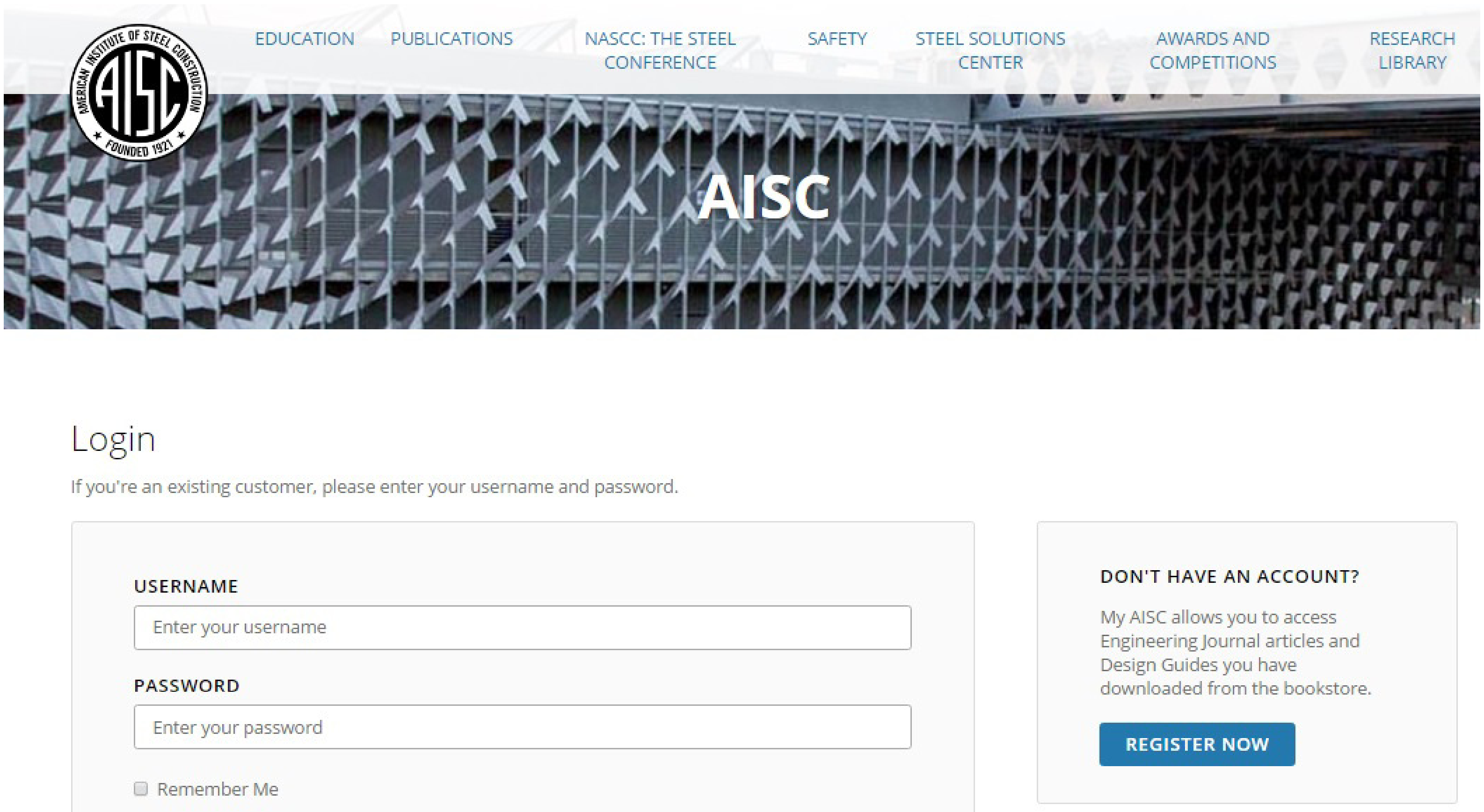
Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.



Night School Resources for 8-session package Registrants

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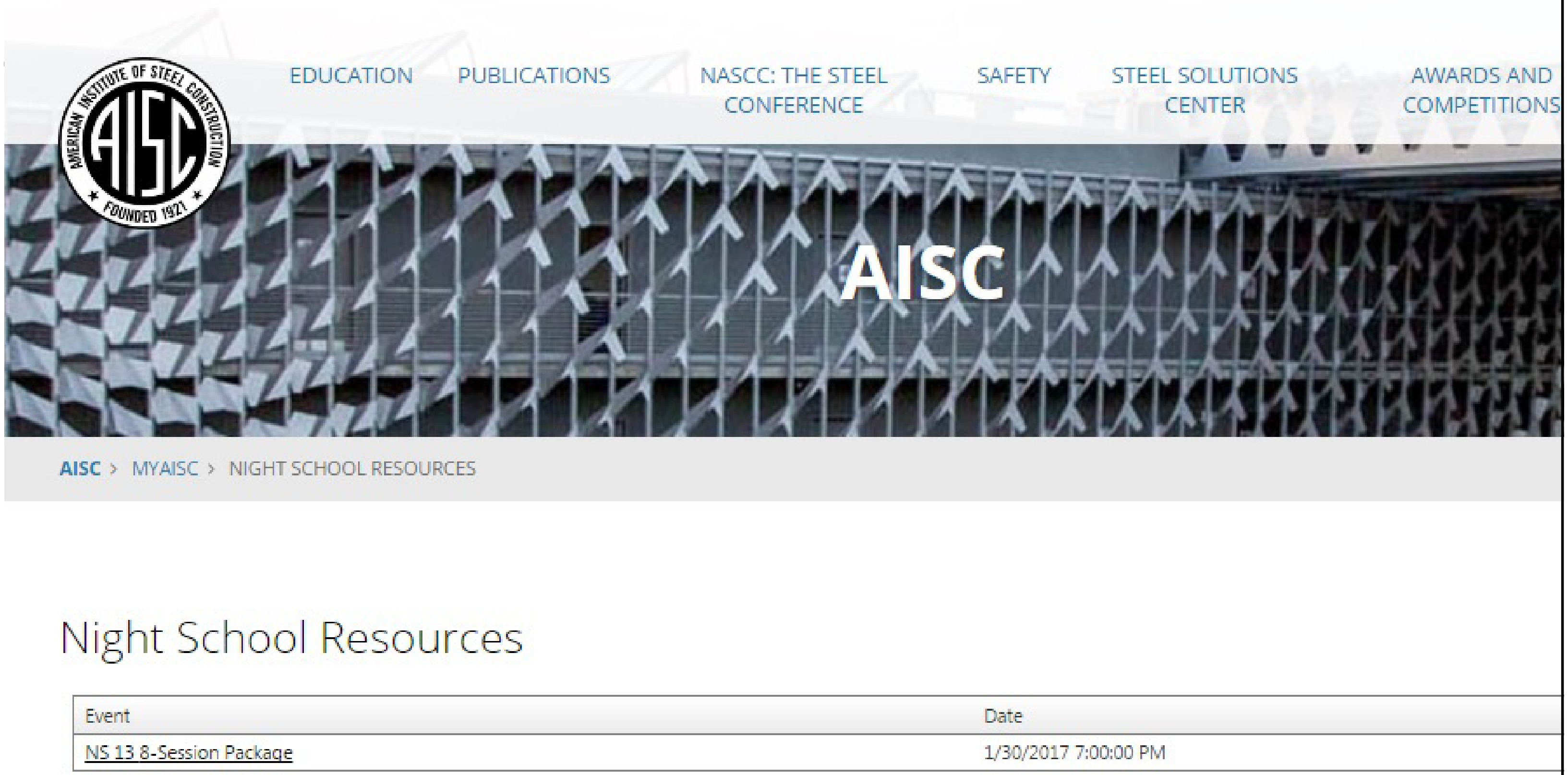
View online resources for Night School courses, available for 8-Session Package registrants. See www.aisc.org/nightschool f
information on Night School including registration.

VIEW RESOURCES

Night School Resources for 8-session package Registrants



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Night School Resources for 8-session package Registrants

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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	

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. 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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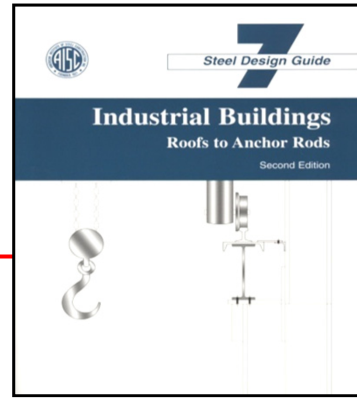
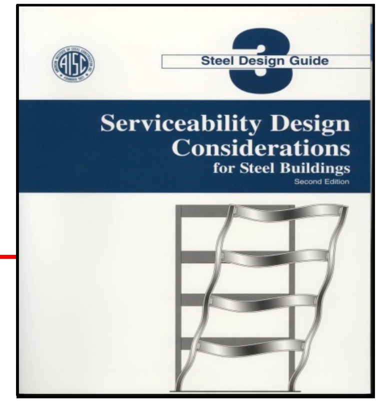
Design Guides





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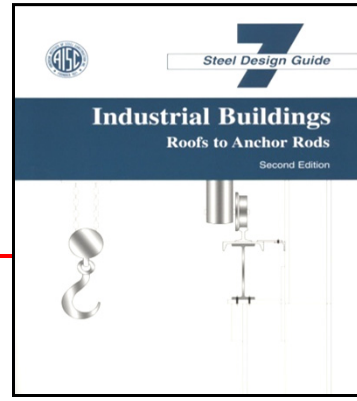

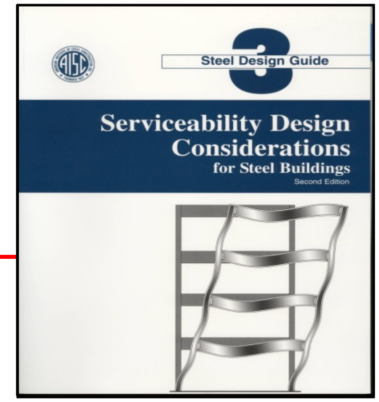

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